HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS
TCEQ PERMIT NO. MSW-2214B

MAJOR PERMIT AMENDMENT APPLICATION

VOLUME 2 OF 3

Prepared for

BFI Waste Systems of North America, LLC

March 2017
Revised August 2017

Revised December 2017

NEVZAT TURAN
84059

12-29-2017

Prepared by

Weaver Consultants Group, LLC
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WCG Project No. 0120-758-11-02

This document is intended for permitting purposes only.

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PART III – SITE DEVELOPMENT PLAN
APPENDIX IIID
LINER QUALITY CONTROL PLAN

Prepared for

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STATE OF TEXAS
NEVZAT TURAN
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LICENSED
PROFESSIONAL ENGINEER

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CONTENTS

STATE OF TEXAS
NEVZAT TURAN
84059
LICENSED PROFESSIONAL ENGINEER

12-5-2017

CONTENTS (Continued)

STATE OF TEXAS
NEVZAT TURAN
84059
LICENSED
PROFESSIONAL ENGINEER

12-5-2017

CONTENTS (Continued)

NEVZAT TURAN
84059

12-5-2017

TABLES

1 INTRODUCTION

1.1 Purpose

This appendix addresses §330.63(d)(4)(G), §330.337, §330.339, and §330.341.

This Liner Quality Control Plan (LQCP) has been prepared for the Hardin County Landfill (landfill) to provide the operator, design engineer, construction quality assurance professional of record, and the contractor the means to govern the construction quality and to satisfy the environmental protection requirements under current Texas Commission on Environmental Quality (TCEQ) Municipal Solid Waste Rules (MSWR). More specifically, the LQCP addresses the soil and geosynthetic components of the liner system. The provisions of this LQCP were developed based on the latest technical guidelines of the TCEQ, including quality control of construction, testing frequencies and procedures, and quality assurance of sampling and testing procedures.

This LQCP is divided into the following parts:

- Section 1 – Introduction
- Section 2 – Construction Quality Assurance for Earthwork and Drainage Aggregates
- Section 3 – Construction Quality Assurance for Geosynthetics
- Section 4 – Construction Quality Assurance for Piping
- Section 5 – Liners Constructed Below the Highest Groundwater Level
- Section 6 – Documentation

1.2 Definitions

Whenever the terms listed below are used, the intent and meaning will be interpreted as indicated.

ASTM

The American Society for Testing and Materials

Atterberg Limits

Atterberg limits testing is a measure of a soil's physical boundaries dealing with its liquidity and plasticity characteristics. Atterberg limits also provide the engineer with preliminary information regarding swell capacity and shear strength, specifically for clay soils. The Atterberg limits used most frequently in geotechnical engineering include the Plastic Limit (PL) and Liquid Limit (LL), and the index value (Plasticity Index, or PI), as defined below:

Liquid Limit (LL) – The percentage of moisture in a soil, subjected to a prescribed test, that defines the upper point at which the soil's consistency changes from the plastic to the liquid state, and the soil being susceptible to viscous flow when jarred or impacted.

Plastic Limit (PL) – The percentage of moisture in a soil, subjected to a prescribed test, that defines the lower point at which the soil's consistency changes from the plastic to the semi-solid state. For soils, the plastic phase is defined as a soil being malleable or moldable, having the ability to be shaped (by rolling for instance) with the soil retaining the shape.

Plasticity Index (PI) – The numerical difference between the LL and the PL of a fine-grained soil that denotes the soils plastic range. The larger the PI, the greater a soil's plasticity range.

Compactive Effort

The amount of compaction energy transferred into a soil sample with a compaction hammer device, used on soil samples in various laboratory test procedures, to establish a soil's moisture-density relationship. Test method ASTM D 698, referred to as the Standard Proctor test, is used to correlate laboratory and field compactive effort.

Construction Quality Assurance (CQA)

A planned system of activities that provides the operator and permitting agency assurance that the facility was constructed as specified in the design. Construction quality assurance includes observations and evaluations of materials, and workmanship necessary to determine and document the quality of the constructed facility. Construction quality assurance (CQA) refers to measures taken by the CQA organization to assess if the installer or contractor is in compliance with the construction plans and specifications for a project. CQA may also be referred to as Quality Assurance.

Construction Quality Assurance Professional of Record (POR)

The POR is an authorized representative of the operator and has overall responsibility for construction quality assurance that confirms the facility was constructed in general accordance with plans and specifications approved by the permitting agency. The POR must be registered as a professional engineer in Texas and experienced in geotechnical testing and its interpretations. Experience and education should include geotechnical engineering, engineering geology, soil mechanics, geotechnical laboratory testing, construction quality assurance and quality control testing, and hydrogeology. The POR must show competency and experience in certifying like installations, and be approved by the permitting agency, and be presently employed by or practicing as a geotechnical engineer in a recognized geotechnical/environmental engineering organization. The credentials of the POR must meet or exceed the minimum requirements of the permitting agency. Any references to monitoring, testing, or observations to be performed by the POR should be interpreted to mean the POR or CQA monitors working under the POR's direction and supervision. The POR or his designated representative will be on-site during all liner system construction.

The POR may also be known in applicable regulations and guidelines as the CQA Engineer, Resident Project Representative, or the Geotechnical Professional (GP).

Construction Quality Assurance (CQA) Monitors

These are representatives of the POR who work under direct supervision of the POR. The CQA monitor is responsible for quality assurance monitoring and performing onsite tests and observations. The CQA monitor is on site full-time during construction and reports directly to the POR. The CQA monitor performing daily QA/QC observation and testing will be NICET-certified in geotechnical engineering technology at level II or higher for soils testing; a CQA monitor with a minimum of four years of directly related experience; or a graduate engineer or geologist with one year of directly related experience. Field observations, testing, or other activities associated with CQA may be performed by the CQA monitor(s) on behalf of the POR.

Additional CQA monitors may be used if they work under the direction of a qualified CQA monitor who is onsite full-time.

Construction Quality Control (CQC)

Construction Quality Control (CQC) provides a means to measure and regulate the characteristics of an item or service to comply with the requirements of the contract documents during construction. CQC will generally be performed by the contractor. CQC may also be referred to as Quality Control.

Contract Documents

These are the official set of documents issued by the operator. The documents include bidding requirements, contract forms, contract conditions, specifications, contract drawings, addenda, and contract modifications.

Contract Specifications

These are the qualitative requirements for products, materials, and workmanship upon which the contract is based.

Contractor

This is the person or persons, firm, partnership, corporation, or any combination, private or public, who, as an independent contractor, has entered into a contract with the operator, and who is referred to throughout the contract documents by singular number and masculine gender.

Design Engineer

These individuals or firms are responsible for the design and preparation of the project construction drawings and specifications. Also referred to as "designer" or "engineer."

Earthwork

This is a construction activity involving the use of soil materials as defined in the construction specifications and Section 2 of this plan.

Film Tear Bond (FTB)

A failure in the geomembrane sheet material on either side of the seam and not within the seam itself.

Fish Mouth

A semi-conical opening of the seam that is formed by an edge wrinkle in one sheet of the geomembrane.

Geomembrane Liner (GM)

This is a synthetic lining material, also referred to as geomembrane, membrane liner, or sheet. The term Flexible Membrane Liner (FML) is also used for GM.

Geomembrane Liner Evaluation Report (GLER)

Certification report for the geomembrane liner, prepared and sealed by the POR that is submitted to the TCEQ for approval, and will include all the documentation necessary for certification of the liner. Also referred to as flexible membrane liner evaluation report (FMLER).

Geosynthetics Contractor

This individual is also referred to as the "contractor" or "installer" and is the person or firm responsible for geosynthetic construction. This definition applies to any person installing FML or geotextile, even if not his primary function.

Independent Testing Laboratory

A laboratory that is independent of ownership or control by the permittee or any party to the construction of the liner system or the manufacturer of the liner system products used.

Manufacturing Quality Assurance (MQA)

A planned system of activities that provides assurance that the raw materials were constructed (manufactured) as specified.

Manufacturing Quality Control (MQC)

A planned system of inspection that is used to directly monitor and control the manufacture of a material.

Nonconformance

This is a deficiency in characteristic, documentation, or procedure that renders the quality of an item or activity unacceptable or indeterminate. Examples of nonconformances include, but are not limited to, physical defects, test failures, and inadequate documentation.

Operator

The organization that will operate the disposal unit. For this LQCP, the term "Operator" also designates the party legally responsible for the construction of a project in accordance with the permit and the permitted designs, and may mean the Operator, Owner, or the Owner/Operator.

Operator's Representative

This is the person that is an official representative of the permittee responsible for planning, organizing, and controlling the design and construction activities.

Organics

Organic matter is material that may be capable of decay (e.g., plant material), the product of decay, or both.

Panel

This is a unit area of the GL or FML, which will be seamed in the field.

Representative Sample

A representative sample of FML material consists of 1 or more specimens (commonly referred to as coupons) from the same rectangular portion of FML material, oriented along a seam, that is removed for field or laboratory testing purposes.

Slip Direction

The direction in which translational slip will occur (i.e., the direction oriented parallel to the sideslope or top slope of the landfill).

Soil Borrow Source

Soils in which the Liquid Limit (LL) and Plasticity Index (PI) do not vary by 10 points. A soil that varies by 10 or more points from the originally established LL or PI is considered as a separate soil source for the purpose of this LQCP and requires a separate soils test series. Minor variations in this requirement may be accepted at the POR's discretion, based on the physical characteristics (color, texture, visual sand or silt content), proximity, or other observed characteristics of the borrow soils.

Soil Liner Evaluation Report (SLER)

Construction report for the soil liner prepared and sealed by the POR and submitted to the TCEQ.

Soil Test Series

Tests performed to determine a soil's physical characteristics and to document its ability to satisfy the soil infiltration layer regulatory requirements. These tests include sieve analysis (gradation), Atterberg Limits, moisture/density, and coefficient of permeability (also referred to in this plan as hydraulic conductivity).

Specimen

(With respect to FML destructive testing) – A specimen is the individual test strip (sometimes called coupon) from a sample location. A sample location usually consists of many specimens.

Quality Assurance

See Construction Quality Assurance, above.

Quality Control

See Construction Quality Control, above.

2 CONSTRUCTION QUALITY ASSURANCE FOR EARTHWORK AND DRAINAGE AGGREGATES

2.1 Introduction

This section of the LQCP addresses the construction of the soil and drainage components of the liner system and outlines the LQCP program to be implemented with regard to materials selection and evaluation, laboratory test requirements, field test requirements, and addressing problems during construction.

The scope of earthwork and related construction quality assurance includes the following elements:

- Subgrade preparation
- Soil liner stockpile
- Soil liner placement
- General fill
- Drainage aggregates
- Anchor trench backfill
- Excavation dewatering

2.2 Composite Liner

The landfill is designed to include a Subtitle D composite liner for the Type I MSW disposal cells will be comprised of 2-foot-thick compacted clay liner overlain by a 60-mil-thick high density polyethylene (HDPE) FML. These liner systems are detailed in Appendix IIIA – Landfill Unit Design Information. The areas of MSW are identified on Drawing A.1 in Appendix IIIA.

A structural stability analysis for the liner system, including calculations for anchor trench runout lengths, stress on the liner components, and an interface slope stability analysis, is included in Appendix IIIE – Geotechnical Report.

2.3 Earthwork Construction

The following paragraphs describe general construction procedures to be used for various earthwork components within the landfill. The earthwork construction specifications will be developed based on the material and construction procedures outlined in this section of the LQCP for each specific liner construction. The earthwork construction specifications will include details for compaction of soils, cross sections showing typical slopes, widths, and thicknesses for compacted lifts.

2.3.1 Subgrade

Subgrade refers to a surface which is exposed after stripping topsoil or excavating to establish the base grades directly beneath the composite liner. The prepared subgrade must conform to the Excavation Grades included in Appendix IIIA – Landfill Unit Design Information. Construction plans and specifications will be developed for each cell construction event, consistent with the liner plans and sector designs set forth in the approved Part III – Site Development Plan.

Prior to beginning liner construction, the subgrade area will be stripped to a depth sufficient to remove all loose surface soils or soft zones within the exposed excavation. The liner subgrade area will be proof-rolled with heavy, rubber-tired construction equipment to detect unstable, soft, or pumping areas. Unstable areas will be undercut to firm material and refilled with suitable compacted general fill. Soil used for backfill will meet the same material requirements as the soil liner and will be installed in accordance with the soil liner installation procedures. The backfill soil will be free of organics, foreign objects, and other deleterious matter, compacted sufficiently to provide a firm base for composite liner placement.

Subgrade voids and cracks are expected to be minor. However, the subgrade will be re-worked as necessary to provide a foundation suitable for soil liner placement. Visual examination of the subgrade preparation by the CQA monitor will generally be sufficient to evaluate its suitability as a foundation for the subgrade. The CQA monitor may find that physical testing is necessary to evaluate the prepared subgrade or fill placed in large voids. The subgrade will be scarified prior to placement of the first lift of clay liner.

The POR will approve the prepared subgrade prior to the placement of soil liner or structural fill. Approval will be based on a review of test information, if applicable, and CQA monitoring of the subgrade preparation. Additionally, as an element of subgrade acceptance, the POR will verify that the underlying material and backfill soils are consistent with the geotechnical design assumptions included in Appendix IIIE (refer to Section 2.4.3).

Surveying will be performed to verify that the finished subgrade is to the lines and grades specified in design with a vertical tolerance of -0.2 feet to +0.0 feet to ensure that the clay liner will achieve a 2-foot thickness. The surface slope of the top layer of clay liner will conform to the slope requirements of the leachate collection layer set forth in the design.

2.4 Soil Liner

The soil liner will consist of a minimum 2-foot-thick compacted clay liner (measured perpendicular to the subgrade surface) that will extend along the floor and side slopes of the landfill cell or sector. The soil liner will be constructed in continuous, single, compacted lifts (6-inch-thick compacted lift thickness) parallel to the floor and sideslope subgrades. Details depicting the liner system are included in Appendix IIIA – Landfill Unit Design Information.

2.4.1 Soil Borrow Material

Adequate clayey soil liner material should be available from proposed landfill excavations and/or onsite borrow sources. The CQA monitor, earthwork contractor, and/or operator will identify the clay material during excavation, and the clay material will be stockpiled separately, if stockpiling is required. If deemed necessary, an off-site borrow source can be used to obtain clay soils for liner and protective cover construction.

The liner soil will be free of debris, rock greater than 1 inch in diameter, vegetative matter, frozen materials, foreign objects, and organics. Laboratory tests will verify that materials are adequate to meet the compacted clay liner requirements listed in 30 TAC §330.339(c)(5) prior to liner construction.

Soils used in soil liner construction will have the following minimum properties as verified by testing in a geotechnical laboratory prior to liner construction.

Table 2-1
Required Borrow Soil Properties

Test[1]	Specification
Coefficient of Permeability (Remolded Sample)[2]	1.0×10^{-7} cm/s or less
Plasticity Index, percent	15 minimum
Liquid Limit, percent	30 minimum
Percent Passing No. 200 Mesh Sieve	30 minimum
Percent Passing 1-inch Sieve	100

[1] Testing will be performed in accordance with the test methods included in Section 2.4.
[2] The coefficient of permeability for remolded sample is run at a minimum of 95% of the maximum dry density at or above the optimum moisture content.

Preliminary sampling and testing will be performed of representative stockpiled soils or off-site borrow sources to be used as liner material. Prior to construction of each new cell, the conformance tests listed in Table 2-1 will be performed for each borrow source proposed for each cell construction sequence. The coefficient of permeability test specimens will be prepared by laboratory compaction to a dry density of approximately 95 percent of the Standard Proctor maximum dry density at or above the optimum moisture content. One Standard Proctor moisture-density relationship and remolded coefficient of permeability test will be required for each borrow source.

The soil is considered as being from a separate borrow source if the liquid limit or plasticity index is determined to vary by more than 10 points. Minor deviations to this requirement for soils excavated from a single borrow source may be accepted at the POR's discretion, based on the physical characteristics (color, texture, visual sand or silt content) or other observed characteristics of the borrow source. Additional conformance tests will be conducted if there are visual changes (color, texture, etc.) in borrow material or as determined necessary by the POR. Atterberg limits testing will be performed on separate borrow sources as an initial determination. If the liquid limit or plasticity index varies by more than 10 points from previously analyzed borrow soils, or if other differences in physical characteristics of the soils are observed by the POR or geotechnical laboratory, then all remaining testing listed in Table 2-1 will be performed on the separate borrow source.

The physical characteristics of the liner materials will be evaluated through visual observation before and during construction. To adjust the moisture content of the material properly, any clod sizes will first be reduced into manageable sizes of 4 inches in diameter or less. Rocks and clods within the compacted liner must be less than 1 inch in diameter. Soil clod size will be reduced to the smallest size necessary to achieve the coefficient of permeability reported by the testing laboratory. Additionally, the rock content of the soil liner will not be more than 10 percent by weight.

The clay soil to be used for liner may require processing to achieve the required moisture content for compaction. Water will be applied as necessary to the material and worked into the material with the processing or compacting equipment. If necessary to achieve even moisture distribution or break down clods, the material will be watered and processed in the stockpile prior to placing in the liner to allow the soil adequate time to hydrate. Water used for the soil liner moisture adjustment must be clean and not contaminated by waste or any objectionable material. Stormwater collected onsite may be used if it has not come into contact with solid waste.

2.4.2 Soil Liner Construction

This LQCP has been developed in accordance with the TCEQ MSWR. The requirements for testing and evaluation of the soil liner during construction are included in this LQCP. The construction methods and test procedures documented in the SLER will be consistent with this LQCP and TCEQ regulations.

The soil liner material will be placed in maximum 8-inch-thick loose lifts to produce compacted lift thicknesses of approximately 6 inches. The soil liner will have elevations, slopes, thickness, and widths as depicted on Drawing A.1 – Excavation Plan, and Drawings A.3 through A.5 – Liner System Details in Appendix IIIA – Landfill Unit Design Information. A temporary hydrostatic pressure relief system will be installed as discussed in Appendix IIID-C.

The liner material will be compacted to a minimum of 95 percent of the maximum dry density at or above the optimum moisture content as determined by Standard Proctor (ASTM D 698). Field density testing will be performed by the CQA monitor or an independent geotechnical laboratory.

The soil liner must be compacted with a pad/tamping-foot (preferable) or prong-foot (sheepsfoot) roller. The lift thickness will be controlled so that there is total penetration through the loose lift under compaction into the top of the previously compacted lift; therefore, the lift thickness must not be greater than the pad or prong length. Use of pad/tamping-foot or prong-foot rollers will provide sufficient roughening of individual lift surfaces for bonding with subsequent lifts. These procedures are necessary to achieve adequate bonding between lifts and reduce seepage pathways. Adequate cleaning devices must be in place and maintained on the compaction roller so that the prongs or pad feet do not become clogged with clay soils to the point that they cannot achieve full lift penetration during compaction. The footed roller is necessary to achieve this bonding and to reduce the individual clods and achieve a blending of the soil matrix through its kneading action. In addition to the kneading action, the weight of the compaction equipment is important. The minimum weight of the compactor should be 40,000 pounds. The Caterpillar 815B and 825C are examples of equipment typically used to achieve satisfactory compaction results. The soil liner will not be compacted with a bulldozer or any track-mobilized equipment unless it is used to pull a pad-footed roller.

During the construction of continuous liners, the new liner segment will not be constructed by "butting" the entire thickness of the new liner directly against the edge of the old liner. The tie-in will be constructed by a sloped transition (typical 5 horizontal to 1 vertical) as shown in Appendix IIIA – Landfill Unit Design Information. The length of the tie-in must be at least 5 feet per foot of liner thickness. The tie-in will be scarified prior to placement of each subsequent lift.

The finished surface of the final lift of soil liner must be rolled with a smooth, steel-wheeled roller to obtain a hard, uniform, and smooth surface. The surface of the final lift of soil liner will then be inspected by the CQA monitor. All undesired materials will be removed from the liner surface, and any voids created by removing undesired materials will be backfilled with liner material to the density specifications outlined for liner construction and tested at the discretion of the CQA monitor.

Surveying will be performed to verify that the finished top of liner grade is to the lines and grades specified in construction plans for a particular cell. Top of soil liner surveying will be performed within a tolerance of -0.0 feet to +0.2 feet. The surface slope of the top layer will conform to the slope requirements of the leachate collection layer. Survey frequency is included in Table 2-2.

CQA testing of the soil liner will be performed as the liner is being constructed. Testing of the soil liner is addressed in Section 2.4. Sections of compacted soil liner which do not pass both the density and moisture requirements will be reworked by moisture conditioning (as required to meet the moisture content criterion), and with additional passes of the compactor until the section in question passes the acceptance criterion. All field density and moisture test results will be incorporated into the SLER.

Hydraulic conductivity (permeability) test samples will be obtained by pushing a tube sampler through the constructed clay liner. The sample from each test location will be sealed and transported to the laboratory. Two samples may be collected at each sample location and labeled the "A" and "B" sample. The sampling holes will be backfilled with bentonite or a bentonite/clay liner soil material mixture consisting of at least 20 percent bentonite.

If the integrity of the "A" sample appears to have been compromised during collection or transportation of the sample prior to testing, the "B" sample may be tested. In addition, if an "A" sample hydraulic conductivity test does not comply with the minimum allowable value, the "B" sample collected at the same location may be tested to determine compliance with the hydraulic conductivity requirements if during testing of the "A" sample, the ASTM D 5084 or EM 1110-2-1906 procedure was not followed or the permeameter malfunctioned. The POR will provide a detailed justification of the use of the "B" sample, if applicable, in the SLER.

If the "B" sample passes, the area will be considered in compliance. If the "B" sample fails (or sample "A" fails in such a way that there is not an option to use the "B" sample), the test interval will be considered unsatisfactory for the area bounded by passing test locations (but not extending past a satisfactory test location). Additional tests may be taken to further define the unsatisfactory area. The area defined unsatisfactory will be reworked and retested in accordance with this section.

Furthermore, if it is determined that the "B" sample may not be used to replace the "A" sample result, then the test interval will be considered unsatisfactory for the area bounded by passing test locations (but not extending past a satisfactory test location).

Once the exact area represented by the failing test is determined, the constructed liner lifts will be removed to the bottom of the lift that did not pass the hydraulic conductivity test, and reconstructed until all the samples obtained from the failed area meet the hydraulic conductivity requirements. At a minimum, one hydraulic conductivity test will be performed for each lift, given that the reconstructed liner area is not larger than 100,000 square feet (i.e., 4 hydraulic conductivity tests per 100,000 square feet of reconstructed MSW liner area). The reconstructed liner area will be tied into the currently constructed liner with a 5H:1V transition slope according to the tie-in detail included in Appendix IIIA – Landfill Unit Design Information. Reconstructed liner area is also subject to field density and moisture content testing per Table 2-2 (at least one field density and one moisture content test is required for each lift regardless of the size of the area that is reconstructed).

Each lift of the reconstructed liner area will be tested for hydraulic conductivity. Reconstruction activities, including additional testing and surveying, will be incorporated into the SLER.

Soil liner construction and testing will be conducted in a systematic and timely fashion on each lift. Delays will be avoided in liner construction. Construction and testing of the soil liner will generally not exceed 60 working days from beginning of liner installation to completion. The TCEQ will be notified during construction if delays in excess of 60 days are anticipated. Reasons for liner construction taking more than 60 days to complete will be fully explained in the SLER submittal.

The POR will submit to the TCEQ a SLER for approval of each soil liner area.

The soil liner will be prevented from losing moisture during the SLER approval process. Preserving the moisture content of the installed soil liner will be dependent on the earthwork contractor's means and methods, and is subject to POR approval.

Upon completion of soil liner construction (but no later than the approval of the SLER by the TCEQ), SLER markers will be installed to clearly indicate the limits of constructed and approved liner areas in accordance with Section 4.7 – Landfill Markers and Benchmark of the approved Site Operating Plan. SLER markers will be located so that they are not destroyed during operations. Any damaged SLER marker will be replaced and/or re-installed immediately.

2.4.3 General Fill/Structural Fill

General fill material placed below the composite liner (e.g., over-excavated areas within the liner footprint) will be placed in uniform lifts which do not exceed 8 inches in loose thickness similar to the compacted clay liner. General structural fill (e.g., detention pond berms) will be placed in uniform lifts which do not exceed 12 inches in loose thickness. The fill placed below the bottom of liner grades will be compacted to at least 95 percent of Standard Proctor maximum dry density (ASTM D 698) at a moisture content at or above the optimum moisture content.

2.4.4 Drainage Aggregate Around Pipes

The coarse aggregate selected for placement around the leachate collection pipes used in the leachate collection system (LCS) for the composite liner and for the temporary hydrostatic pressure relief system discussed in Section 5 will consist of normal (i.e., unit weight of 90 to 110 pcf) or lightweight (i.e., unit weight not to exceed 70 pcf) materials that comply with the following criteria. The LCS aggregate will have a calcium carbonate content less than 15 percent. Either the J&L Testing method or the ASTM D 3042 method, modified to use a solution of hydrochloric acid having a pH of 5, can be used to determine calcium carbonate content. The drainage aggregate will meet the following gradation for ASTM D 448, size number 467 (nominal aggregate size 1.5 inches to No. 4 sieve).

Sieve Size Square Opening	Percent Passing
2 inches	100
1½ inches	95 - 100
¾ inch	35 - 70
3/8 inch	10 - 30
No. 4 (3/16 inch)	0 - 5

However, if approved by the POR, coarse aggregates not complying with the size number 467 gradation may also be used if demonstrated to have a hydraulic conductivity of at least 1.0 cm/s and meet the filter gradation requirements given below. In no case will the maximum stone size be more than 2 inches for the specific leachate collection pipe perforation design.

For circular holes in the leachate collection pipe:

$$\frac{\text{85 Percent Size of Filter Material}}{\text{Hole Diameter}} > 1.7$$

For slots in the leachate collection pipe:

$$\frac{\text{85 Percent Size of Filter Material}}{\text{Slot Width}} > 2.0$$

The coarse aggregate will be sampled for gradation testing (ASTM D 448) at the supply source or from the on-site stockpile prior to acceptance. Gradation testing will be conducted at a minimum frequency of 1 test per 3,000 cubic yards of coarse aggregate or 1 test per liner construction event if less than 3,000 cubic yards of coarse aggregate is required for the specific construction. The aggregate will be free of organics, angular rocks, foreign objects, or other deleterious materials. The physical characteristics of the aggregate will be evaluated through visual observation and laboratory classification testing before construction and visual observation during construction. The coarse aggregate may be tested during construction at the discretion of the CQA monitor. The test results for the coarse aggregate will be included in the GLER.

2.4.5 Protective Cover

Protective cover will be placed over the drainage layer in accordance with this section and approved Top of Liner Plan (Appendix IIIA, Appendix IIIA-A, Drawing IIIA.1) for each cell construction. The geosynthetics of the composite liner system will be covered with a minimum of 2 feet of protective cover for the composite liner. The protective cover will consist of soil materials that have not previously come in contact with solid waste or other deleterious materials, and do not contain materials detrimental to the underlying geosynthetics. The protective cover will be free of organics, foreign objects, or other deleterious materials. The physical characteristics of the protective cover will be evaluated through visual observation (and laboratory testing if the POR deems it necessary) before construction, and visual observation during construction. Additional testing during construction will be at the discretion of the CQA monitor. The protective cover will have passageways (i.e., chimney drains) to allow leachate to drain to the leachate collection system.

The protective cover layer will be placed using any low ground pressure equipment as outlined in Section 3.6. The protective cover will be placed by spreading in front of the spreading equipment with a minimum of 12 inches of soil between the spreading equipment and the installed geosynthetics. Under no circumstances will the construction equipment come in direct contact with the installed geosynthetics.

The thickness of the protective cover layer placed over the composite liner will be verified with surveying procedures at a minimum of 1 survey point per 5,000 square feet of constructed area by a licensed Texas land surveyor with a minimum 2 reference points. The survey results for the protective cover will be included in the GLER.

During construction the CQA monitor will:

- Verify that grade control is performed prior to work.
- Verify that underlying geosynthetic installations are not damaged during placement operations or by survey grade controls. Mark damaged geosynthetics and verify that damage is repaired.
- Verify that the cover soil for sideslopes is pushed from the toe up the slope.
- Monitor haul road thickness over geosynthetic installations and verify that equipment hauling and materials placement meet equipment specifications. (See Section 3.6)
- The POR will coordinate with the project surveyor to perform a thickness verification survey of the protective cover materials upon completion of

placement operations. Verify corrective action measures as determined by the verification survey.

2.4.6 Anchor Trench Backfill

The anchor trench backfill material for geosynthetic anchoring will be placed in uniform lifts which do not exceed 12 inches in loose thickness and will be compacted to at least 95 percent of Standard Proctor maximum dry density (ASTM D 698) at a moisture content at or above the optimum moisture content. In-place moisture/density tests may be taken at the discretion of the CQA monitor to evaluate the quality of the backfill placement and compaction. The test results will not be required as part of the GLER.

2.4.7 Surface Water Removal

Excavations may encounter water from storm events. Soil liner will not be placed in standing water. The excavation area will therefore have a temporary sump area to collect water entering the excavation and will be graded to facilitate drainage to the sump. Portable pumps will be on site to dewater the sumps. Temporary earthen berms will be constructed to divert surface flow away from the excavation. Surface water that accumulates on the constructed liner or geosynthetics surface will be removed promptly after the end of each rainfall event. POR will inspect and approve the constructed area that received rainfall prior to placement of the overlying liner system component. The criteria for approval of the finished surface of the soil liner for geomembrane placement will follow the requirements of Section 3.3.3, and for geocomposite placement on top of geomembrane will follow the requirements of Section 3.5.3. Surface water from the site will be discharged in compliance with the site's TPDES permit.

2.4.8 Excavations Below Groundwater

Construction of liners below groundwater is discussed in Section 5 of this document.

2.4.9 Liner Tie-In Construction

Newly constructed liners will be tied in with any adjoining existing liners. Additionally, terminations will be constructed for future tie-ins along edges where the liner will be extended in the future. The tie-ins with existing clay liners will be constructed utilizing a sloped transition a minimum of 10 feet wide for the 2-foot-thick clay liner (5H:1V). Terminations for future tie-ins will be constructed by extending the clay liner approximately 10 feet past the limits for the cell under construction. The liner tie-in details are shown in Appendix IIIA – Landfill Unit Design Information. Waste and intermediate cover will not be deposited closer than 10 feet to the edge of any cell or 20 feet from the leading edge of a constructed clay liner (whichever is greater) where a future tie-in will be constructed. Red-colored markers (i.e., SLER markers) will be placed along the limits of the cells with constructed clay liners and tied to the site grid system in accordance with 30 TAC §330.143(b)(1).

2.5 Construction Testing

2.5.1 Standard Operating Procedures

CQA monitors with qualified professional experience in geotechnical engineering and/or engineering geology will perform field and laboratory tests in accordance with applicable standards specified in this LQCP. All quality control testing and evaluation of soil liners will be performed during construction of the liner and must be complete before placement of the leachate collection system, except for the testing required for the final constructed lift, verification of liner thickness, or cover material thickness. Standard operating and test procedures will be utilized per the POR's direction. Sampling from the constructed soil liner lifts will be performed in accordance with ASTM D 1587. The sampling holes (e.g., samples for coefficient of permeability testing) will be backfilled with bentonite or bentonite/liner soil material mixture with a minimum 20 percent bentonite. Prior written approval from the TCEQ by permit modification will be obtained if any changes will be made to material requirements or procedures set forth on this LQCP.

The following test standards apply as called out in this LQCP and in the project specifications:

Standard Test Method	Test Description
ASTM D 698	Moisture-density relations of soils and soil-aggregate mixtures, using 5½-lb hammer and 12-inch drop
ASTM D 422	Particle size analysis of soils
ASTM D 6938	Standard test method for in-place density and water content of soil and soil aggregate by nuclear methods (shallow depth)
ASTM D 1587	Thin-walled tube sampling of soils for geotechnical purposes
ASTM D 2167	Density and unit weight of a soil in place by the rubber balloon method
ASTM D 2216	Laboratory determination of water (moisture) content of soil, rock, and soil-aggregate mixtures
ASTM D 2434	Method of test for permeability of porous granular material
ASTM D 5084	Method of test for permeability of fine-grained soils

Standard Test Method	Test Description
ASTM D 4318	Atterberg limits
ASTM D 1140	Amount of material in soils finer than the No. 200 sieve
ASTM D 2487	Classification of soils for engineering purposes
ASTM D 2488	Description and identification of soils (visual-manual procedure)
EM 1110-2-1906	U.S. Army Corps of Engineers permeability test
ASTM D 448	Standard classification for sizes of aggregate for road and bridge construction
ASTM D 3042	Test method for insoluble residue in carbonate aggregates

2.5.2 Test Frequencies

This LQCP establishes the minimum test frequencies for the soil liner construction quality assurance. The test frequencies are listed in Table 2-2. Additional testing must be conducted at the direction of the POR whenever work or materials are suspect, marginal, or of poor quality. Additional testing may also be performed to provide additional data for engineering evaluation. The minimum number of tests is interpreted to mean minimum number of passing tests, and any tests that do not meet the requirements will not contribute to the total number of tests performed to satisfy the minimum test frequency.

Table 2-2
Required Tests and Observations on Compacted Clay Liner

Parameter	Frequency	Test Method	Passing Criteria
Field Density and Moisture	1 each 8,000 SF per 6-inch parallel lift	ASTM D 6938 and ASTM D 2216[1]	95% of Standard Proctor maximum dry density. Standard Proctor optimum moisture content or greater as determined during preconstruction testing
Sieve Analysis (passing no. 200)	1 test per 100,000 square feet per 6-inch parallel lift, with a minimum of 1 test per 6-inch lift	ASTM D 1140	30 percent minimum
Atterberg Limits (liquid and plastic limit)	1 test per 100,000 square feet per 6-inch parallel lift, with a minimum of 1 test per 6-inch lift	ASTM D 4318	PI = 15 percent minimum LL = 30 percent minimum
Coefficient Permeability (Hydraulic Conductivity)[2]	1 test per 100,000 square feet per 6-inch parallel lift, with a minimum of 1 test per 6-inch lift	ASTM D 5084 (Falling head, flex wall) Corps of Engineers EM 1110-2-1906 (Falling head permeameter)	1.0×10^{-7} cm/s or less
Thickness Verification	1 each 5,000 square feet with a minimum of 2 reference points by a licensed Texas land surveyor	Surveys performed of liner subgrade, top of clay liner, and top of protective cover layer	2 feet minimum compacted clay liner thickness and 2 feet minimum protective cover thickness (all areas)

[1] This method is not applicable if the field nuclear gauge reads both density and moisture.

[2] Field permeability testing by Boutwell STEI performed in accordance with 30 TAC §330.339(c)(7), may be performed to augment this testing program if a permit modification is submitted and approved by the TCEQ.

Liner construction may require general fill (e.g., for establishing excavation grades by means of backfilling as necessary, as discussed in Section 2.3.1–Subgrade) and structural fill (e.g., for construction of berms around the excavation perimeter). Testing of general/structural fill soils will be limited to one moisture density relationship (ASTM D 698) per borrow source (as defined in Section 2.3.2.1) per project.

2.5.3 Material Strength Requirements

The geotechnical analysis is included in Appendix IIIE – Geotechnical Report which includes slope stability, foundation heave, and settlement analyses for the proposed excavation. Soil parameters used in the geotechnical analysis were obtained from the previous subsurface investigations and geotechnical reports, as well as from geotechnical testing performed on soil samples recovered at the site. POR will verify that the proposed liner material meets the minimum soil properties used in the geotechnical analysis included in Appendix IIIE (refer to Table 6-1 in Appendix IIIE) prior to liner construction, as applicable. The POR will verify that the underlying subgrade material below the liner is consistent with design assumptions. If the POR finds the underlying subgrade material is not consistent with design assumptions, the appropriate geotechnical analysis (e.g., slope stability) in Appendix IIIE will be updated to ensure that the minimum required factor of safety is met. The updated analysis will be incorporated into the SLER.

2.6 Reporting

The POR will submit to the TCEQ a SLER for approval of each soil liner area. Section 6 describes the documentation requirements.

3 CONSTRUCTION QUALITY ASSURANCE FOR GEOSYNTHETICS

3.1 Introduction

Section 3 describes CQA (as defined in Section 1.2, above) procedures for the installation of geosynthetic components.

The scope of geosynthetic related CQA includes the following elements:

- Geomembrane
  - 60-mil HDPE – smooth on slopes less than 7H:1V and textured on both sides for slopes greater than or equal to 7H:1V, or as determined by stability analysis
- Geotextiles
- Drainage Layer
  - Single-sided drainage geocomposite (on slopes less than 7H:1V)
  - Double-sided drainage geocomposite (on slopes greater than or equal to 7H:1V), or as determined by stability analysis, and for the sideslope underdrain

The overall goal of the geosynthetics CQA program is to assure that proper construction techniques and procedures are used, the geosynthetic contractor implements his CQC (as defined in Section 1.2, above) plan in accordance with this LQCP, and that the project is built in accordance with the project construction drawings and technical specifications that will be developed in accordance with this LQCP for each liner construction. The CQA program is intended to identify and define problems that may occur during construction and to observe that these problems are avoided and/or corrected before construction is complete. A construction report and GLER, prepared after project completion, will document that the constructed facility meets design intent and specifications outlined in this LQCP.

3.2 Geosynthetics Quality Assurance

3.2.1 General

The composite liner system provides the primary means for preventing leachate infiltration into groundwater. A geomembrane is a component of the composite liner. Proper geomembrane installation is a crucial work element, which greatly affects the performance of the composite liner system. CQC for the geomembrane installation will be performed by the geomembrane installation contractor. CQA for the geomembrane installation will be performed by the CQA monitor under the direction of the POR to assure the geomembrane is constructed as specified in the design. Construction must be conducted in accordance with the procedures outlined in this LQCP. To monitor compliance, the CQA program will include the following:

- Review of the manufacturer's quality control testing
- Material conformance testing by an independent third party laboratory
- Field and construction testing
- Construction monitoring

The manufacturer's quality control testing will include resin and geomembrane testing. The required tests for material properties are included in Section 3.3.

Conformance testing refers to material testing performed by an independent third party laboratory that takes place prior to material installation. Field and construction testing includes testing that occurs during geosynthetics installation.

CQA testing will be conducted in accordance with this LQCP. Field testing will be observed by the CQA monitor. Documentation must meet the requirements of this LQCP.

3.3 Geomembrane

The composite liner system geomembrane will consist of a 60-mil high density polyethelene (HDPE) geomembrane. The geomembrane will be smooth on both sides on slopes less than 7H:1V and textured on both sides on slopes greater than or equal to 7H:1V, or as determined from the results of stability analyses. Required manufacturer's quality control tests for the geomembrane are included in Table 3-1 and required material properties for the geomembrane are included in Table 3-2.

3.3.1 Delivery

Upon delivery of FML, the CQA monitor will observe that:

- The geomembrane is delivered in rolls and is not folded. Folded geomembrane is not acceptable because the highly crystalline structure of the geomembrane will be damaged if it is folded. Any evidence of folding (other than from the manufacturing process) or other shipping damage is cause for rejection of the material.
- Equipment used to unload and store the rolls or pallets does not damage the geomembrane. The geomembrane is stored in an acceptable location in accordance with the manufacturer's specifications and stacked not more than 5 rolls high. The geomembrane is protected from puncture, dirt, grease, water, moisture, mud, mechanical abrasions, excessive heat, or other damage.
- All manufacturing documentation required by the project specifications and outlined in this LQCP has been received and reviewed for compliance. This documentation will be included in the GLER.
- The geosynthetics receipt log form has been completed for all materials received.

Damaged geomembrane will be rejected and removed from the site or stored at a location separate from accepted geomembrane. Geomembrane that does not have proper manufacturer's documentation must be stored at a separate location until all documentation has been received, reviewed, and accepted.

3.3.2 Conformance Testing

Tests. Prior to shipment of geomembrane to the site, one geomembrane sample will be obtained by the manufacturer for every resin lot of material supplied and for each 50,000 square feet of geomembrane to be installed. The samples will be forwarded to the independent third-party laboratory for the following conformance tests:

- Specific gravity/Density (ASTM D 1505 or alternate ASTM D 792, Method A if approved by the POR)
- Carbon black content (ASTM D 1603)
- Carbon black dispersion (ASTM D 5596)
- Thickness (ASTM D 5199 for smooth FML and for textured FML use ASTM D 5994 or alternate ASTM D 1593 if approved by POR)
- Tensile properties (ASTM D 6693, ASTM D 638/Type IV specimen)

The density of the geomembrane must be greater than 0.94 g/cc; the carbon black content must be between 2 percent and 3 percent; and recycled or reclaimed material must not be used in the manufacturing process.

In addition to the third-party laboratory testing listed above, the manufacturer must certify the geomembrane to be installed at the site in accordance with Table 3-1 and certify that the minimum required properties in Table 3-2 are met.

The POR may require additional test procedures and will inform the third party laboratory in writing. The POR must review all test results and report any nonconformance to the design engineer prior to product installation.

In addition to the conformance thickness tests shown above, field thickness measurements must be taken at maximum 5-foot intervals along the leading edge of each geomembrane panel. For smooth geomembranes, no single measurement will be less than 10 percent below the required nominal thickness for the panel to be accepted (i.e., for 60-mil geomembrane a minimum thickness of 54 mils is required) and the average must be at least 60 mils. Additionally, field testing of seams will be performed in accordance with Section 3.3.4. Refer to Table 3-2 for a complete listing of the material requirements for both smooth and textured geomembranes that will be used for the composite liner.

Table 3-1
Required Testing for 60-mil-thick Smooth and Textured (Both Sides) HDPE Geomembranes[1]

Test	Type of Test	Standard Test Method	Frequency of Testing (Minimum)
Resin	Specific Gravity/Density	ASTM D 792, Method A or ASTM D 1505	Per 100,000 SF and every resin lot
	Melt Flow Index	ASTM D 1238	Per 100,000 SF and every resin lot
Manufacturer's Quality Control	Thickness	ASTM D 5199 (smooth) or ASTM D 5994[2] (textured)	Per roll of geomembrane
	Specific Gravity/Density	ASTM D 1505/D 792	Per 200,000 pounds
	Carbon Black Content	ASTM D 1603	Per 20,000 pounds
	Carbon Black Dispersion	ASTM D 5596	Per 45,000 pounds
	Tensile Properties	ASTM D 6693, ASTM D 638/ Type IV specimen	Per 20,000 pounds
	Tear	ASTM D 1004	Per 45,000 pounds
	Puncture	ASTM D 4833	Per 45,000 pounds
	Stress Crack Resistance	ASTM D 5397	Per GRI-GM 10
	Oxidative Induction Time	ASTM D 3895 or ASTM D 5885	Per 200,000 pounds
	Oven Aging @ 85°C, minimum average	ASTM D 5721	Per each formulation
	Oven Aging @ 85°C Standard OIT (min. avg.) - % retained after 90 days	ASTM D 3895	Per each formulation
	UV Resistance[3], minimum average	GRI GM11	Per each formulation
	High Pressure OIT (min. avg.) - % retained after 1,600 hours	ASTM D 5885	
	Asperity Height[4]	GRI GM12	Every 2nd roll

[1] All tests will conform to the current minimum requirements set forth by GRI testing standard GM13. Required values for the parameters are listed in Table 3-2.

[2] ASTM D 1593 may also be used for thickness of textured geomembrane at the option of the POR.

[3] 20 hours of UV cycle at 75°C followed by 4 hours condensation at 60°C.

[4] Measurement side will be alternated for double-sided textured sheet. This testing is specified for textured geomembrane only.

Sampling Procedure. Samples will be taken across the entire roll width. Unless otherwise specified, samples will be approximately 15 inches long by the roll width. The CQA monitor must mark the machine direction and the manufacturer's roll identification number on the sample. The CQA monitor must also assign a conformance test number to the sample and mark the sample with that number.

3.3.3 Geomembrane Installation

Surface Preparation. Prior to any geomembrane installation, the installed soil liner surface will be inspected by the CQA monitor and geosynthetics contractor. The POR or CQA monitor must observe the following:

- All lines and grades for the soil liner have been verified by the surveyor and accepted by the contractor for geosynthetic installation. The POR or his representative, the owner, and geomembrane installer will certify and accept in writing the finished final lift of the soil liner.
- The soil liner has been prepared in accordance with the earthwork construction plans and specifications as outlined in Section 2.
- The soil liner surface is free of surface irregularities and protrusions. The soil liner will be rolled and compacted to ensure a clean surface.
- The soil liner surface does not contain stones or other objects that could damage the geomembrane and underlying soil liner. The surface of the soil liner will be smooth and free of foreign and organic material, sharp objects, exposed soil or aggregate particles greater than 3/8 inches (or less if recommended by the geosynthetic manufacturer), or other deleterious material. There are no rocks, debris, or any other objects on the soil liner surface.
- The anchor trench dimensions have been checked, and the trenches are free of sharp objects and stones.
- There are no excessively soft areas in the soil liner that could result in geomembrane damage. The soil liner is not saturated, and no standing water is present above the soil liner.
- The geomembrane will not be placed over soil liner during inclement weather such as rain or high winds.
- The soil liner has not desiccated, and no areas with desiccation cracks are observed.
- All construction stakes and hubs have been removed and the resultant holes have been backfilled.
- The geosynthetics contractor, POR or his representative, and the permittee or his representative have certified in writing that the surface on which the geomembrane will be installed is acceptable.

Table 3-2
Minimum Required Properties of 60-mil-thick Smooth and Textured (Both Sides) HDPE Geomembranes

Property	Test Method	Minimum Required Property[8]	
		Smooth	Textured
Thickness, mils Minimum average Lowest individual reading Lowest individual of 8 of 10 readings	ASTM D 5199 (smooth) ASTM D 5994 (textured)	60 54 NA	57 51 54
Density, g/cc	ASTM D 1505/D 792	0.940	0.940
Asperity Height, mils	GRI GM12	N/A	10
Tensile Properties[1] 1. Yield Strength, lb/in 2. Break Strength, lb/in 3. Yield Elongation, % 4. Break Elongation, %	ASTM D 6693, ASTM D 638/ Type IV specimen	126 228 12 700	126 90 12 100
Tear Resistance, lb	ASTM D 1004	42	42
Puncture Resistance, lb	ASTM D 4833	108	90
Stress Crack Resistance[2], hrs	ASTM D 5397	300	300
Carbon Black Content[3], %	ASTM D 1603	2.0 – 3.0	2.0 – 3.0
Carbon Black Dispersion[4], Category	ASTM D 5596	1 or 2 and 3	1 or 2 and 3
Oxidative Induction Time (OIT)[5] (Minimum Average) Standard OIT, minutes High Pressure OIT, minutes	 ASTM D 3895 ASTM D 5885	 100 400	 100 400
Oven Aging at 85ºC Standard OIT – % retained after 90 days High Pressure OIT – % retained after 90 days	ASTM D 5721 ASTM D 3895 ASTM D 5885	 55 80	 55 80
UV Resistance[6] High Pressure OIT[7] – % retained after 1600 hrs	GRI GM 11 ASTM D 5885	 50	 50
Seam Properties (4 out of 5 specimens, 5th specimen can be as low as 80% per GRI-GM19) 1. Shear Strength, lb/in 2. Peel Strength, lb/in	ASTM D 6392	 120 91 & FTB (78, Extrusion Weld)	 120 91 & FTB (78, Extrusion Weld)

[1] Machine direction (MD) and cross machine direction (XMD) average values will be on the basis of 5 test specimens each direction. Yield elongation is calculated using a gauge length of 1.3 inches; break elongation is calculated using a gauge length of 2.0 inches.

[2] The yield stress used to calculate the applied load for the Single Point Notched Constant Tensile Load (SP-NCTL) test will be the mean value via MQC testing.

[3] Other methods such as ASTM D 4218 or microwave methods are acceptable if an appropriate correlation can be established.

[4] Carbon black dispersion for 10 different views in Categories 1 and 2 and 1 in Category 3.

[5] The manufacturer has the option to select either one of the OIT methods listed to evaluate the antioxidant content in the geomembrane.

[6] The condition of the test will be 20 hr UV cycle at 75ºC followed by 4 hr. condensation at 60ºC.

[7] UV resistance is based on percent retained value regardless of the original HP-OIT value.

[8] Minimum required properties are based on the current GRI-GM13, except for the seam properties which are based on the current GRI-GM19.

Panel Placement. The CQA monitor must maintain an up-to-date panel layout drawing showing panel numbers that are keyed to roll numbers on the placement log. The panel layout drawing will also include seam numbers and destructive test locations.

During panel placement, the CQA monitor must:

- Observe that geomembrane is placed in direct and uniform contact with the underlying compacted clay soil liner.
- Record roll numbers, panel numbers, and dimensions on the panel or seam logs.
- Observe the sheet surface as it is deployed and record all panel defects and repair of the defects (panel rejected, patch installed, extradite placed over the defect, etc.) on the repair sheet. All repairs must be made in accordance with the specifications as outlined in Section 3.3.5 and located on a repair drawing.
- Observe that support equipment is not allowed on the geomembrane during handling (see Section 3.6 also).
- Observe that the surface beneath the geomembrane has not deteriorated since previous acceptance.
- Observe that there are no stones, construction debris, or other items beneath the geomembrane that could cause damage to the geomembrane.
- Observe that the geomembrane is not dragged across a surface that could damage the material. If the geomembrane is dragged across an unprotected surface, the geomembrane must be inspected for scratches and repaired or rejected, as necessary.
- Record weather conditions including temperature, wind, and humidity. The geomembrane must not be deployed in the presence of excess moisture (e.g., fog, dew, mist, wind, etc.). In addition, geomembrane will not be placed when the air temperature is less than 41°F, or when standing water or frost is on the ground, unless this requirement is waived by the POR and TCEQ. Excessive wind is that which can lift and move the geomembrane panels.
- Observe that people working on the geomembrane do not smoke, wear shoes that could damage the liner, or engage in activities that could damage the liner.
- Observe that the method used to deploy the sheet minimizes wrinkles but does not cause bridging and that the sheets are anchored to prevent movement by the wind (the contractor is responsible for any damage to or from windblown geomembrane). Excessive wrinkles will be walked-out or removed at the discretion of the CQA monitor.

- Observe that no more panels are deployed than can be seamed on the same day.
- Observe that there are no horizontal seams on side slopes (no flatter than 45 degrees and that are no closer than 5 feet from toe of slope), and the textured material extends a minimum of approximately 5 feet beyond the toe of the slope where textured geomembrane is used.

The CQA monitor must inform both the contractor and the POR of the above conditions.

Field Seaming. The contractor must provide the POR with a seam and panel layout drawing and update this drawing daily as the job proceeds. A seam numbering system must provide a unique number for each seam and be agreed to by the POR and contractor prior to the start of seaming operations. One procedure is to identify the seam by adjacent panels. For example, the seam located between Panels 306 and 401 would be Seam No. 306/401.

Prior to geomembrane welding, each welder and welding apparatus (both wedge and extrusion welders), must be tested to determine if the equipment is functioning properly, at a minimum, at daily start-up and at midday break, or any break that the seaming machine is stopped more than 30 minutes. The GLER will include the names for each seamer and the time and the temperatures for each seaming apparatus used each day. One trial weld will be taken prior to the start of work. The trial weld sample must be 3 feet long and 12 inches wide, with the seam centered lengthwise. The minimum number of specimens per trial weld test must be two coupons for shear and two coupons for peel. Both the inner and outer welds of dual track fusion welds must be tested for each peel test coupon (or additional coupons will be required). Trial weld samples must comply with "Passing Criteria for Welds" included in Section 3.3.4 – Construction Testing. The CQA monitor will observe welding operations, quantitative testing of each trial weld for peel and shear, and recording of the results on the trial weld form. The trial weld be completed under conditions similar to those under which the panels will be welded. Regarding the locus-of-break patterns of the different seaming methods in shear and peel, the following are unacceptable break codes per their description in ASTM D 6392 and GRI-GM19:

Hot Wedge:	AD and AD-Brk>25%
Extrusion Fillet:	AD1, AD2, AD-WLD (unless strength is achieved)

Additionally, there will be no apparent weld separation (i.e., greater than 1/8 inch). The third party strength tests must meet the manufacturer's specifications for the sample sheets, or the percentage of the manufacturer's parent sheet strength as determined by the manufacturer. For dual-track fusion welds, both sides (the inner and outer weld) must meet the minimum requirements for a satisfactory peel test. If, at any time, the CQA monitor believes that a welding apparatus is not functioning properly, a weld test must be performed. If there are wide changes in temperature (±30° Fahrenheit), humidity, or wind speed, the test weld will be repeated. The test weld must be allowed to cool to ambient temperature before testing. If a welded seam fails the shear or peel test, the length of the non-passing weld will be identified at a 10-foot interval and the failed seam will be patched. Patching will performed by placing additional geomembrane over the failed seam or removing the failed seam geomembrane weld and patching it with additional geomembrane per the POR's direction. Welding for patches must comply with the extrusion weld acceptance criteria outlined in this section.

Construction quality assurance documentation of trial seam procedures will include, at a minimum, the following:

- Documentation that trial seams are performed by each welder and welding apparatus prior to commencement of welding and prior to commencement of the second half of the workday.
- The welder, the welding apparatus number, time, date, ambient air temperature, and welding machine temperatures.

During geomembrane welding operations, the CQA monitor must observe the following:

- The contractor has the number of welding apparatuses and spare parts necessary to perform the work.
- Equipment used for welding will not damage the geomembrane.
- The extrusion welder is purged prior to beginning a weld until all the heat-degraded extradite is removed (extrusion welding only).
- Seam grinding has been completed less than one hour before seam welding, and the upper sheet is beveled (extrusion welding only).
- The ambient temperature, measured 6 inches above the geomembrane surface, is between 41° and 104° Fahrenheit unless more stringent limits are required by the manufacturer.
- The end of old welds, more than five minutes old, are ground to expose new material before restarting a weld (extrusion welding only).
- The contact surfaces of the sheets are clean, free of dust, grease, dirt, debris, and moisture prior to welding.
- The weld is free of dust, rocks, and other debris.
- The seams are overlapped a minimum of 3 inches for extrusion and hot-wedge welding, or in accordance with manufacturer's recommendations, whichever is more stringent. Panels will be overlapped (shingled) in the downgrade direction.

- No solvents or adhesives are present in the seam area.
- The procedure used to temporarily hold the panels together does not damage the panels and does not preclude CQA testing.
- The panels are being welded in accordance with the plans and specifications that will be developed in accordance with this section for each liner construction.
- There is no free moisture in the weld area.
- Observe that at the end of each day or installation segment, all unseamed edges are anchored with sandbags or other approved device. Penetration anchors will not be used to secure the geomembrane.

3.3.4 Construction Testing

Nondestructive Seam Testing. The purpose of nondestructive testing is to detect discontinuities or holes in the seam. It also indicates whether a seam is continuous and non-leaking. Nondestructive tests for geomembrane include vacuum testing and air pressure testing. Nondestructive testing must be performed over the entire length of the seam.

Nondestructive testing is performed entirely by the contractor. The CQA monitor's responsibility is to document the date, time and location of seaming and testing, and to observe and document that testing was performed in compliance with this section and document any seam defects and their repairs.

Nondestructive testing procedures are described below.

- For welds tested by vacuum method, the weld is placed under suction utilizing a vacuum box made of rigid housing with a transparent viewing window, a soft neoprene rubber gasket attached to the open bottom perimeter, a vacuum gauge on the inside, and a valve assembly attached to the vacuum hose connection. The box is placed over a seam section, which has been thoroughly saturated with a soapy water solution (1 oz. soap to 1 gallon water). The rubber gasket on the bottom perimeter of the box must fit snugly against the soaped seam section of the liner, to ensure a leak-tight seal. The vacuum pump is energized, and the vacuum box pressure is reduced to approximately 3 to 5 psi gauge. Any pinholes, porosity or non-bonded areas are detected by the appearance of soap bubbles in the vicinity of the defect. Dwell time (i.e., the time the vacuum box is held over each section of extrusion weld) must not be less than ten seconds.
- Air pressure testing is used to test dual-track fusion seams welded using the hot wedge method. Both ends of the channel created between the dual seams are sealed, and the channel is pressurized. The pressure feed device, usually a needle equipped with a pressure gauge, is inserted through the geomembrane into the channel. Air is then pumped into the channel to a

minimum pressure of 30 psi or ½ psi per mil of geomembrane thickness, whichever is greater. The air chamber must sustain the pressure for five minutes without losing more than 4 psi. Following a passing pressure test, the opposite end of the tested seam must be punctured to release the air. The pressure gauge must return to zero; if not, a blockage is most likely present in the seam channel. Locate the blockage and test the seam on both sides of the blockage. The penetration holes must be sealed by extrusion welding after testing.

Prior to and during nondestructive testing, the CQA monitor must perform the following tasks:

- Review project specifications that will be developed in accordance with this LQCP regarding test procedures, and observe that all testing is completed and recorded on the appropriate log in accordance with the project specifications.
- Observe that equipment operators are fully trained and qualified to perform their work.
- Observe that test equipment meets project specifications.
- Observe that the entire length of each seam is tested in accordance with the project specifications.
- Identify the failed areas by marking the area with a waterproof marker compatible with the geomembrane and inform the contractor of any required repairs, then record the repair area on the repair log.
- Observe that all repairs are completed and tested in accordance with the project specifications outlined in this section and Section 3.3.5.
- Record all completed and tested repairs on the repair log and the repair drawing.

Destructive Seam Testing. Destructive seam tests for will be performed at intervals of at least one test per 500 linear feet. Destructive testing will also be performed for individual repairs (or additional seaming for the failed seams) of more than 10 feet of seaming. The CQA monitor must perform additional tests if he suspects a seam does not meet project specification requirements. Reasons for performing additional tests may include, but are not limited to the following:

- Wrinkling in seam area
- Non-uniform weld
- Excess crystallinity
- Suspect seaming equipment or techniques
- Weld contamination

- Insufficient overlap
- Adverse weather conditions
- Possibility of moisture, dust, dirt, debris, and other foreign material in the seam
- Failing tests

There are two types of destructive testing required for the geomembrane installation: peel adhesion (peel) and bonded seam strength (shear) in accordance with ASTM D 6392. The purpose of peel and shear tests is to evaluate seam strength and to evaluate long-term performance. Shear strength measures the continuity of tensile strength through the seam and into the parent material. Peel strength determines weld quality. Test welds must be allowed to cool naturally to ambient temperature prior to testing.

The CQA monitor selects locations where seam samples will be cut for laboratory testing. Select these locations as follows:

- A minimum of one random test within each 500 feet of seam length. This is an average frequency for the entire installation; individual samples may be taken at greater or lesser intervals.
- Sample locations will not be disclosed to the contractor prior to completion of the seam.
- A maximum frequency must be agreed to by the contractor, POR, and the operator at the preconstruction meeting. However, if the number of failed samples exceeds 5 percent of the tested samples, this frequency may be increased at the discretion of the POR. Samples taken as the result of failed tests do not count toward the total number of required tests.

Sampling Procedures. The contractor will remove samples at locations identified by the CQA monitor. The CQA monitor must:

- Observe sample cutting.
- Mark each sample with an identifying number, which contains the seam number and destructive test number.
- Record sample location on the panel layout drawing and destructive seam log.
- Record the sample location, weather conditions, and reason sample was taken (e.g., random sample, visual appearance, result of a previous failure, etc.).

For each destructive test obtain one sample approximately 45 inches long by 12 inches wide, with the weld centered along the length. Cut two 1-inch-wide coupons from each end of the sample. The contractor must test two of these coupons in shear and two in peel (one shear and one peel from each end of sample) using a tensiometer capable of quantitatively measuring the seam strengths. For dual-track wedge welding, both sides of the air channel will be tested in peel. The CQA monitor must observe the tests and record the results on the destructive seam test log. A geomembrane seam sample passes the field testing when the break is Film Tear Bond (FTB) and the seam strength meets the required strength values for peel and shear given previously for trial seams under field seaming and below for third party laboratory testing. As previously discussed, both welds have to pass for dual-track welds. Also, it is recommended that additional samples be obtained as discussed in the following paragraph if there is apparent separation of the weld (i.e., greater than 1/8 inch) during peel testing.

If one or both of the 1-inch specimens fail in either peel or shear, the contractor can, at his discretion: (1) reconstruct the entire seam between passed test locations, or (2) take two additional test samples 10 feet or more in either direction from the point of the failed test and repeat this procedure. For recordkeeping purposes the additional samples will be identified by assigning an identifying letter to the initial destructive test sample number (e.g., DS-6A and B). Only satisfactory tests count toward the required minimum number, and additional tests (i.e., A and B) count as one test, if passing. If the second set of tests pass, the contractor can reconstruct or cap-strip the seam between the two passed test locations. If subsequent tests fail, the sampling and testing procedure is repeated until the length of the poor quality seam is established. Repeated failures indicate that either the welding equipment or equipment operator is not performing properly, and appropriate corrective action must be taken immediately.

If the field test coupons are satisfactory, divide the remaining sample into three parts: one 12-inch by 12-inch section for the contractor, one 12-inch by 16-inch section for the third party laboratory for testing, and one 12-inch by 12-inch section for the operator to archive. The laboratory sample will be shipped to the third party laboratory for overnight delivery and next day testing.

If the laboratory test fails in either peel or shear, the contractor must either reconstruct the entire seam between passing test locations or recover additional samples at least 10 feet on either side of the failed sample for retesting. Sample size and disposition must be as described in the preceding paragraph. This process is repeated until passed tests bracket the failed seam section. All seams must be bounded by locations from which passing laboratory tests have been taken. Laboratory testing governs seam acceptance. In no case can field testing of repaired seams be used for final acceptance.

Third Party Laboratory Testing. Destructive samples must be shipped to the third party laboratory for seam testing. Testing for each sample will include 5 bonded seam shear strength tests and 5 peel adhesion tests (10 for dual-track welds). For dual-track welds each peel test specimen (coupon) will be tested on both sides of the air channel (i.e., the inner and outer welds). At least four of the five specimens tested in peel and shear will meet the minimum strength requirements. The minimum peel strength and the minimum shear strength values must meet the passing criteria listed below. Additionally, 4 out of 5 of the peel test coupons must have no greater than 25 percent seam separation. For dual-track welds if either weld exhibits greater than 25 percent separation or does not meet the required strength, that coupon is considered out of compliance and two out of compliance coupons cause the weld to fail. The third party laboratory must provide test results within 24 hours, in writing or via telephone, to the CQA monitor. Certified test results are to be provided within 5 days. The CQA monitor must immediately notify the POR in the event of a calibration discrepancy or failed test results.

Passing Criteria for Welds. Passing criteria are established by Geosynthetic Institute (GRI) Test Method GM19 for geomembranes (the most recent version of GM19 will be used at the time of each liner construction). A passing extrusion or fusion welded seam will be achieved when the following values are obtained. The following values listed for shear and peel strengths are for 4 out of 5 test specimens (the 5th specimen can be as low as 80 percent of the listed values). Elongation measurements will be omitted for field testing.

• Shear strength (lb/in)	120
• Shear elongation at break (%)	50
• Peel strength (lb/in)	91 (78, Extrusion Weld) & FTB
• Peel separation (%)	25

A passing extrusion or fusion welded seam will be achieved in peel when:

- Yield strength for 4 of 5 specimens (10 tests for dual-track welds) is not less than the above minimum peel strength value and the average of all 5 specimens is not less than the minimum value.
- No greater than 25 percent of the seam width peels (separates) at any point for 4 of 5 specimens (both inner and outer welds for dual-track welds).

A passing extrusion or fusion weld will be achieved in shear when:

- Yield strength for 4 of 5 specimens is not less than the above minimum shear strength value and the average for all 5 specimens is not less than the minimum value.
- Yield strain for 4 out of 5 specimens is at least 25 percent.
- Break strain for 4 out of 5 specimens is at least 50 percent.

3.3.5 Repairs

Any portion of the geomembrane with a detected flaw, or which fails a nondestructive or destructive test, or where destructive tests were cut, or where nondestructive tests left cuts or holes, must be repaired in accordance with the project specifications. The CQA monitor must locate and record all repairs on the repair sheet and panel layout drawing. Repair techniques include the following:

- Patching – used to repair large holes, tears, large panel defects, undispersed raw materials, contamination by foreign matter, and destructive sample locations.
- Extrusion – used to repair small defects in the panels and seams. In general, this procedure will be used for defects less than ⅜-inch in the largest dimension.
- Capping – used to repair failed welds or to cover seams where welds or bonded sections cannot be nondestructively tested.
- Removal – used to replace areas with large defects where the preceding methods are not appropriate. Also used to remove excess material (wrinkles, fishmouths, intersections, etc.) from the installed geomembrane. Areas of removal will be patched or capped.

Repair procedures include the following:

- Abrade geomembrane surfaces to be repaired (extrusion welds only) no more than one hour prior to the repair.
- Clean and dry all surfaces at the time of repair.
- Extend patches or caps at least 6 inches beyond the edge of the defect, and round all corners of material to be patched and the patches to a radius of at least 3 inches. Bevel the top edges of patches prior to extrusion welding.
- Testing of repaired seams consistent with Section 3.3.4 – Construction Testing.

3.3.6 Wrinkles

During placement of cover materials over the geomembrane, temperature changes or creep can cause wrinkles to develop in the geomembrane. Any wrinkles which can fold over must be repaired either by cutting out excess material or, if possible, by allowing the liner to contract by temperature reduction. In no case can material be placed over the geomembrane, which could result in the geomembrane folding. The CQA monitor must monitor geomembrane for wrinkles and notify the contractor if wrinkles are being covered by soil. The CQA monitor is then responsible for documenting corrective action to remove the wrinkles.

3.3.7 Folded Material

All folded geomembrane must be removed. Remnant folds evident after deployment of the roll which are due to the manufacturing process are acceptable.

3.3.8 Geomembrane Anchor Trench

The geomembrane anchor trench will be left open until seaming is completed. Expansion and contraction of the geomembrane will be accounted for in the liner placement into the trench. Prior to backfilling, the depth of penetration of the geomembrane into the anchor trench must be verified by the CQA monitor at a minimum of 100-foot spacing along the anchor trench.

3.3.9 Geomembrane Acceptance

The contractor retains all ownership and responsibility for the geomembrane until acceptance by the operator. In the event the contractor is responsible for placing cover over the geomembrane, the contractor retains all ownership and responsibility for the geomembrane until all required documentation is complete, and the cover material is placed. After panels are placed, seamed, tested successfully, and any repairs are made, the completed installation will be walked by the operator and contractor representatives. Any damage or defect found during this inspection will be repaired properly by the installer. The installation will not be accepted until it meets the requirements of both representatives. In addition, the geomembrane will be accepted by the POR only when the following has been completed:

- The installation is finished.
- All seams have been inspected and verified to be acceptable.
- All required laboratory and field tests have been completed and reviewed.
- All required contractor-supplied documentation has been received and reviewed.
- All as-built record drawings have been completed and verified by the POR. The as-built drawings show the true panel dimensions, the location of all seams, trenches, pipes, appurtenances, and repairs.
- Acceptance of the GLER by TCEQ.

3.3.10 Bridging

Bridging must be removed.

3.4 Geotextiles

Geotextiles will be used to prevent clogging of drainage materials. The main usage of geotextiles will be enveloping drainage stone used for chimney drains in the leachate collection system (LCS). Geotextiles for the LCS will meet the design requirements set forth in Appendix IIIC – Leachate and Contaminated Water Management Plan and in Table 3-3 of this LQCP.

3.4.1 Delivery

During delivery the CQA monitor must observe the following:

- Equipment used to unload the rolls will not damage the geotextile.
- Rolls are wrapped in impermeable and opaque protection covers.
- Care is used when unloading the rolls.
- All documentation required by the project specifications has been received and reviewed for compliance.
- Each roll is marked or tagged with the manufacturer's name, project identification, lot number, roll number, and roll dimensions.
- Materials are stored in a location that will protect the rolls from precipitation, mud, dirt, dust, puncture, cutting, or any other damaging or deleterious conditions.

Any damaged rolls must be rejected and removed from the site or stored at a location separate from accepted rolls, designated by the operator. All rolls which do not have proper manufacturer's documentation must also be stored at a separate location until all documentation has been received and approved.

3.4.2 Testing

The geotextile manufacturer will conduct manufacturer quality control (MQC) testing and certify that the materials delivered to the site comply with project specifications. The material certification will be reviewed by the POR and approved for the project prior to acceptance of any of the material. The MQC testing will include the following tests with at least one test for each 100,000 square feet of geotextile delivered.

- Grab tensile strength/elongation (ASTM D 4632)
- Mass per unit area (ASTM D 5261)
- Thickness (ASTM D 5199)
- Puncture resistance (ASTM D 4833)
- Trapezoidal tear strength (ASTM D 4533)
- Hydraulic tests (ASTM D 4491)
- Apparent opening size (ASTM D 4751)

Where optional procedures are noted in the test method, the specification requirements of this LQCP prevail. The POR will review all test results and report any nonconformance.

3.4.3 Geotextile Installation

Surface Preparation. Prior to geotextile installation, the CQA monitor must observe the following:

- All lines and grades have been verified by the surveyor.
- The supporting surface does not contain stones that could damage the geotextile or the underlying geomembrane.
- There are no excessively soft areas that could result in damage to the geotextile, or other components of the liner system.
- Construction stakes and hubs have been removed.

Geotextile Placement. During geotextile placement, the CQA monitor must:

- Observe the geotextile as it is deployed, and record all defects and disposition of the defects (panel rejected, patch installed, etc.). Repairs are to be made in accordance with the specifications outlined in Section 3.4.4.
- Observe that equipment used does not damage the geotextile by handling, equipment transit, leakage of hydrocarbons, or other means.
- Observe that people working on the geotextile do not smoke, wear shoes that could damage the geotextile, or engage in activities that could damage the geotextile.
- Observe that the geotextile is securely anchored in an anchor trench.
- Observe that the geotextiles are anchored to prevent movement by the wind.
- Observe that the panels are overlapped a minimum of six inches.
- Examine the geotextile after installation to ensure that no potentially harmful foreign objects are present.
- Observe that seams (where required) are continuously sewn or thermal bonded in accordance with the manufacturer's recommendations and the project specifications.

The CQA monitor must inform both the contractor and POR if the above conditions are not met.

3.4.4 Repairs

Repair procedures include:

- Patching – used to repair large holes, tears, large defects, and destructive sample locations.
- Removal – used to replace areas with large defects where the preceding method is not appropriate.

Holes, tears, and defects must be repaired in the following manner. Soil or other material which may have penetrated the defect must be removed completely prior to repair. If located on a slope, the defect must be patched using the same type of geotextile and double-seamed into place. Should any tear, hole, or defect exceed 30 percent of the width of the roll, the roll will be cut off and the defect removed or the roll removed and replaced. If the defect is not located on a slope, the patch must be made using the same type of material seamed into place with a minimum of 24 inches overlap in all directions. Seams will be either thermal bonded or sewn in accordance with the manufacturer's recommendations.

3.5 Drainage Geocomposite – Geonet and Geotextile

Drainage geocomposite will be used for the LCS and temporary dewatering system (see Section 5) if installed. Drainage geocomposite used for the construction will meet the requirements set forth in Appendix IIIE - Geotechnical Report and Appendix IIIC - Leachate and Contaminated Water Management Plan of the Site Development Plan along with this LQCP. Additionally, manufacturer's testing for geotextile and drainage geocomposite for the composite liner are listed in Table 3-3. Drainage geocomposite for the composite liner will also meet the requirements listed in Table 3-3. Specifications for single and double-sided drainage geocomposite for the leachate collection system are provided in Appendix IIIC – Leachate and Contaminated Water Management Plan and Table 3-3 of this LQCP. Table 3-4 lists the required testing and properties for the dewatering system.

3.5.1 Delivery

Upon delivery the CQA monitor must observe the following:

- The drainage geocomposite is wrapped in rolls with protective covering.
- The rolls are not damaged during unloading.
- Protect the drainage geocomposite from mud, soil, dirt, dust, debris, cutting, or impact forces.
- Each roll must be marked or tagged with proper identification.

Any damaged rolls will be rejected and removed from the site or stored at a location, separate from accepted rolls, designated by the operator. All rolls which do not have proper manufacturer's documentation will also be stored at a separate location until all documentation has been received and approved. Geonet and geotextile may be used as an alternative to single-sided geocomposite. The references herein to geocomposite also apply to geonet and geotextile as applicable.

3.5.2 Testing

The drainage geocomposite manufacturer (or supplier) will conduct quality control testing and certify that all materials delivered to the site comply with the specifications listed in Table 3-3 for the composite liner and Table 3-4 for the dewatering system. The minimum testing frequency will be one test sample per 100,000 square feet of geocomposite (or geonet/geotextile). See footnote 2 of Table 3-3 and Table 3-4 for testing frequency for transmissivity. The material certifications will be reviewed by the POR to verify that the geocomposite meets the values given in Table 3-3 and Table 3-4.

Geonet will be tested by the manufacturer for thickness, tensile strength, and carbon black content. Geotextile will be tested for mass per unit area, grab tensile strength, and AOS. The finished geocomposite will be tested for peel adhesion and transmissivity (note that the geocomposite transmissivity tests need to be conducted by a third party laboratory only under the specific conditions listed in Table 3-3 and Table 3-4, footnote 2). The manufacturer's testing for drainage material is also summarized in Table 3-3 and Table 3-4. Additionally, material strength parameters used for geotechnical analysis in Appendix IIIE – Geotechnical Report will be verified prior to construction, and slope stability analysis will be updated as necessary based on site-specific material data.

Where optional procedures are noted in the test method, the specification requirements of this LQCP prevail. The CQA monitor will review all test results and will report any nonconformance to the POR and to the contractor.

3.5.3 Installation

Surface Preparation. Prior to drainage geocomposite installation, the CQA monitor must observe the following:

- All lines and grades have been verified by the surveyor (where required).
- The subgrade has been prepared in accordance with the earthwork specifications outlined in Section 2.
- When placed over a geomembrane, the geomembrane installation, including all required documentation, has been completed.
- The supporting surface does not contain stones that could damage the geocomposite or the geomembrane.

Drainage Geocomposite Placement. During placement, the CQA monitor must:

- Verify that single-sided geocomposite is placed on slopes less than 7H:1V and double-sided geocomposite is placed on slopes greater than or equal to 7H:1V.
- Observe the drainage geocomposite as it is deployed and record defects and disposition of the defects (panel rejected, patch installed, etc.). Repairs are to be made in accordance with the specifications outlined in Section 3.5.4.
- Verify that equipment used does not damage the drainage geocomposite or underlying geomembrane by handling, trafficking, leakage of hydrocarbons, or by other means.
- Verify that people working on the drainage geocomposite do not smoke, wear shoes that could damage the drainage geocomposite, or engage in activities that could damage the drainage geocomposite or underlying geomembrane.
- Verify that the drainage geocomposite is anchored to prevent movement by the wind (the contractor is responsible for any damage resulting to or from windblown drainage geocomposite).
- Verify that the drainage geocomposite remains free of contaminants such as soil, grease, fuel, etc.
- Observe that the drainage geocomposite is laid smooth and free of tension, stress, folds, wrinkles, or creases.

Table 3-3

Required Testing and Properties of Geotextile and Drainage Geocomposite for Composite Liner[1]

Responsible Party	Material	Test	Standard	Required Property[4]
Manufacturer	Geotextile	Unit Weight	ASTM D 5261	6 oz/sy
		Apparent Opening Size	ASTM D 4751	80 sieve
		Grab Strength	ASTM D 4632	157 lbs
		Tear Strength	ASTM D 4533	56 lbs
		Puncture Strength	ASTM D 4833	56 lbs
		Permeability	ASTM D 4491	0.2 cm/s
Manufacturer	HDPE Geonet	Specific Gravity	ASTM D 1505	0.939 g/cm^3
		Thickness	ASTM D 5199	0.25 inch
		Carbon Black	ASTM D 1603	2%
		Tensile Strength	ASTM D 5035	45 lb/in (Peak)
Third Party Laboratory	Drainage Geocomposite	Transmissivity[2]	ASTM D 4716	See Note 2.
		Strength[3]	ASTM D 5321	Protective Cover/Geocomposite: C_a=100 psf, Δ=16° Single-sided Geocomposite/Smooth Geomembrane: C_a=100 psf, Δ=11° Double-sided Geocomposite/Textured Geomembrane: C_a=100 psf, Δ=15° Smooth or Textured Geomembrane/Clay Liner: C_a=100 psf, Δ=16°
Manufacturer		Peel Adhesion	ASTM D 7005	1.0 lb/in

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[1] The minimum testing frequency will be one test sample per 100,000 square feet.

[2] As noted in Appendix IIIC, Appendices IIIC-A and IIIC-B, the transmissivity of the single-sided geocomposite will be measured at a gradient of 0.023 under normal pressures of 1,000, 8,000 and 10,000 (or higher) psf, boundary conditions consisting of soil/geocomposite/geomembrane with minimum seating time of 100 hours and will be run for the first 100,000 square feet of liner construction. For each additional 100,000 square feet of single-sided geocomposite placement area, one additional transmissivity test will be run under the maximum normal stress (i.e., 10,000 psf) or higher with all the same assumptions as the first three tests. The minimum transmissivity will be ***x10^{-4} m^2/s. The transmissivity of the double-sided geocomposite shall be measured at a minimum gradient of 0.33 under a minimum normal pressure of 1,000, 5,000 and 10,000 (or higher) psf, boundary conditions consisting of soil/geocomposite/geomembrane with a minimum seating time of 100 hours. The minimum transmissivity will be 2.64x10^{-4} m^2/s. For each additional 100,000 square feet of double-sided geocomposite placement area, one additional transmissivity test will be run under the maximum normal stress (i.e., 10,000 psf) or higher with all the same assumptions as the first three tests.

[3] The adhesion and interface friction angle of the geocomposite components will be determined to verify they meet the values used in the slope stability analysis of Appendix IIIE-A. Refer to Appendix IIIE -A for detailed strength information and procedures for calculating factors of safety.

[4] Minimum required property values are based on engineering judgment and experience with similar materials. However, each material will be tested prior to construction to verify that it meets the minimum required properties.

Table 3-4
Required Testing and Properties of Drainage Geotextile for Dewatering System[1, 4]

Responsible Party	Material	Test	Standard	Required Property[3]
Manufacturer	Geotextile	Unit Weight	ASTM D 5261	6 oz/sy
		Apparent Opening Size	ASTM D 4751	80 sieve
		Grab Strength	ASTM D 4632	157 lbs
		Tear Strength	ASTM D 4533	56 lbs
		Puncture Strength	ASTM D 4833	56 lbs
		Permeability	ASTM D 4491	0.18 cm/s
Third Party Laboratory		Strength[2]	ASTM D 5321	Clay Liner/Geotextile: C_a = 100 psf, Δ = 16° Geotextile/Subgrade: C_a = 100 psf, Δ = 16°

[1] The minimum testing frequency will be one test sample per 100,000 square feet.

[2] The adhesion and interface friction angle of the underdrain geotextile will be determined to verify they meet the values used in the slope stability analysis of Appendix IIIE-A. Refer to Appendix IIIE-A-4 for detailed strength information and procedures for calculating factors of safety.

[3] Minimum required property values are based on engineering judgment and experience with similar materials. However, each material will be tested prior to construction to verify that it meets the minimum required properties.

[4] The dewatering system design information is provided in Appendix IIID-C.

- Observe that on slopes the drainage geocomposite is secured in the liner anchor trench and then rolled down the slope.
- Observe that adjacent rolls of drainage geocomposite are overlapped a minimum of six inches, tied, and seamed in accordance with the manufacturer's recommendations.
- Observe that tying is with plastic fasteners in accordance with the manufacturer's recommendations. In the absence of other specifications the drainage geocomposite panels will be tied approximately every 5 feet along the roll length (edges) and every 1 foot along the roll width (ends).
- Observe that the geotextile component is overlapped and either heat bonded or sewn together.

3.5.4 Repairs

Repair procedures include:

- Holes or tears in the drainage geocomposite will be repaired by placing a patch extending 2 feet beyond the edges of the hole or tear.
- Secure patch to the originally installed drainage geocomposite by tying every 6 inches.

- Where the hole or tear width across the roll is more than 50 percent of the roll width the damaged area will be cut out across the entire roll and the two portions of the drainage geocomposite will be joined.

3.6 Equipment on Geosynthetic Materials

Construction equipment on the liner system will be minimized to reduce the potential for liner puncture. The CQA monitor will verify that small equipment such as generators are placed on scrap liner material (rub sheets) above geosynthetic materials in the liner system. Aggregate drainage layers and/or protective cover will be placed using low ground pressure equipment. The CQA monitor will verify that the geosynthetics are not displaced while the soil layers are being placed.

Unless otherwise specified by the POR, lift thicknesses of protective cover soil placed over geosynthetics will conform with the following guidelines.

Equipment Ground Pressure (psi)	Minimum Lift Thickness (in)
<5.0	12
5.1 - 8.0	18
8.1 - 16.0	24
>16.0	36

No equipment will be left running and unattended over the lined area.

3.7 Reporting

The POR will submit to the TCEQ a GLER for approval of the flexible membrane liner, leachate collection system and protective cover. Section 6 describes the documentation requirements.

4 QUALITY ASSURANCE FOR PIPING

4.1 Introduction

This section describes CQA procedures for the installation of HDPE pipe for the leachate collection system used for the composite liner.

The goal of the pipe CQA program is to assure that proper construction techniques and procedures are used, and that the project is built in accordance with the project construction drawings and specifications developed for each liner construction. The following specifications apply to the leachate collection system piping:

- Minimum internal diameter = 5.845 inches for leachate collection pipe and nominal diameter of 18 inches for riser pipe
- Standard dimension ratio = 17
- Perforation hole diameter = 0.5 inches (if slotted pipe is used, standard slot width = 0.125 inches)
- Young's modulus for pipe material = 33,000 psi
- For LCS design/requirements regarding chemical resistance, refer to Appendix IIIC.

4.2 Pipe and Fittings

4.2.1 General

Construction must be conducted in accordance with the project construction drawings and specifications for each liner constructed. Piping design and specifications are provided in Appendix IIIC – Leachate and Contaminated Water Management Plan. To monitor compliance, a CQA program will be implemented that includes: (1) a review of the manufacturer's quality control testing, (2) material conformance testing, and (3) construction monitoring. If manufacturer data is not available, third-party conformance testing may be utilized.

4.2.2 Delivery

The CQA monitor will observe:

- That upon delivery, the pipe and pipe fittings are in compliance with the requirements of the project specifications developed for each liner construction.
- That a storage location is selected in which the pipe and pipe fittings are protected from excessive heat, cold, construction traffic, hazardous chemicals, solvents, and theft. If the pipe and pipe fittings are stored at a location where other construction materials are present, the CQA monitor will assure that stacking or insertion of the other construction materials onto or into the pipe and pipe fitting is prohibited. The CQA monitor will periodically examine the storage area to observe that the pipe fittings are undamaged and are protected.
- That upon transporting pipe and fittings from the storage location to the construction site, the contractor will use pliable straps, slings, or ropes to lift the pipe. Steel cables or chains will not be allowed to transport or lift the pipe or fittings.
- That the contractor will provide that a pipe greater than 20 feet in length will be lifted with at least two support points. The contractor will not drop, impact, or bump into the pipe, particularly at the pipe ends. Pipe and fitting ends must be cleaned of all dirt, debris, oil, or any other contaminant which may prohibit making a sound joint.

The CQA monitor will document all activities associated with the handling and storage of the pipe and fittings is in accordance with the project specifications as outlined in this LQCP.

4.2.3 Conformance Testing

Prior to the installation of pipe, the pipe manufacturer will provide to the POR a quality control certificate for each lot or batch of pipe provided. The quality control certificate will be signed by a responsible party employed by the pipe manufacturer, such as the quality control manager. The quality control certificate and documentation will include:

- A description of the pipe delivered to the project, including but not limited to the strength classification, diameter, dimension ratio, perforations, and production lot.
- Properties sheet including, at a minimum, all specified properties, measured using test methods indicated in the project specifications that will be developed for each liner construction, or equivalent.

- A certification that property values given in the properties sheet are minimum values and are guaranteed by the pipe manufacturer.
- A list of quantities and descriptions of materials other than the base resin which comprise the pipe. The sampling procedure and results of testing for actual samples manufactured in the same lot as the pipe delivered to the project.

The POR will confirm that:

- The property values certified by the pipe manufacturer meet the project specifications as outlined in this LQCP for each liner construction.
- The measurements of properties by the pipe manufacturer are properly documented and that the test methods used are acceptable.
- Verification that the quality control certificates have been provided at the specified frequency for all lots or batches of pipe, and that each certificate identifies the pipe lot/batch related to it.

4.2.4 Pipe and Fitting Installation

Surface Preparation. Prior to pipe installation, the CQA monitor must observe the following:

- All lines and grades have been verified by the contractor and project surveyor.
- The pipe trenches are swept clean of any deleterious material which may damage the pipe or geomembrane or may clog the pipe.
- Pipe perforations for leachate collection system are drilled in the pipe outside of the drainage trench where the pipe is to be laid. The drill cuttings must be completely removed from the pipe prior to being placed in the pipe trench.
- Pipe perforations are the correct size and spacing according to the project specifications that will be developed for each liner construction. Perforations can be factory-installed slots or predrilled holes, or field-drilled holes.

Pipe and Fitting Placement. During pipe and fitting installation, the CQA monitor will:

- Observe all pipe, pipe fittings, and joints as the pipe is being laid. The CQA monitor will observe that pipes and fittings are not broken, cracked, or otherwise damaged or unsatisfactory. Prior to fusing (if fusion welding is utilized), the pipe installer will provide for a fusion surface area which is clean and free of moisture, dust, dirt, debris of any kind, and foreign material.

- If fusion welding is utilized, verify welder credentials and that the procedure is consistent with the pipe manufacturer's recommendations.
- Observe that the pipe and fittings are being constructed in accordance with specifications that will be developed in accordance with this LQCP for each liner construction and accepted practices.
- Observe that the people and equipment utilized to install the pipe and fittings do not damage the pipe or fittings, or any other component of the liner system.

5 LINERS CONSTRUCTED BELOW THE HIGHEST GROUNDWATER LEVEL

5.1 Introduction

Liners constructed below the groundwater table could potentially experience uplift due to hydrostatic pressure acting on the geomembrane liner. This section of the LQCP describes procedures for short and long-term protection of the liner system due to hydrostatic uplift pressure that may result from liner construction below the highest measured groundwater table.

Excavations at the site remaining for this permit amendment will be limited to Cells 7 and 8. Excavations for Cells 7 and 8 will generally be founded in upper or lower clay stratum. When founded in the lower clay stratum, excavations will penetrate the upper sand stratum, with the upper sand stratum exposed in the excavation sideslopes. In addition, the excavation grades for Cells 7 and 8 are below the highest measured groundwater surface shown on Figure IIID-A-1 in Appendix IIID-A. Therefore, a dewatering system will be installed in these cells to control the potential short-term hydrostatic uplift pressures.

Long-term liner stability will be provided in the form of ballast that will be created by the weight of protective cover, solid waste, and final cover as applicable. Ballast calculations are included in Appendix IIID-B – Example Ballast Thickness Calculations. Ballast is provided for the entire undeveloped area (Cells 7 and 8). The highest measured groundwater surface is included in Appendix IIID-A and used in the ballast calculations.

5.2 Highest Measured Groundwater Levels

Based on the current hydrogeologic investigations and previous investigations, the site geology affecting landfill development consists of three zones: Upper Clay Stratum, Upper Sand Stratum, and Lower Sand Stratum zones. Water levels from the current monitoring wells and piezometers screened in the Upper Sand Stratum indicate that the groundwater potentiometric surface is higher than the excavation floor. As discussed in Section 5.3, a temporary dewatering system is designed for the undeveloped portion of the landfill (Cells 7 and 8) to control hydrostatic pressure that may act on the liner system. Long-term stability of the liner system will be ensured by the use of ballast to counteract potential uplift pressures created by hydrostatic forces acting on the bottom of the liner system.

A highest groundwater potentiometric surface map is included in Appendix IIID-A. Detailed groundwater information is presented in Appendix IIIG – Geology Report. Drawing IIID-A-1 shows the current highest measured groundwater potentiometric surface at the site.

As each new cell is designed, the highest measured groundwater levels will be adjusted accordingly for possible higher well level data, and the highest measured groundwater potentiometric contours for the individual cell will be used for design of ballast. Any temporary hydrostatic relief system design different than the one presented in Appendix IIID-C will be submitted to the TCEQ for approval as a modification to the LQCP.

5.3 Temporary Dewatering System for Cells 7 and 8

A dewatering system will be used for the undeveloped portion of the landfill (Cells 7 and 8) where the groundwater is expected to be above top of liner grades. As shown in Appendix IIID-C (Sheet IIID-C-3), the dewatering system will be installed that includes a geocomposite on the sideslopes and gravel backfilled trenches around the perimeter of the cells and along the centerline of the cells. Refer to Section 5.31 for more information.

The dewatering system is designed to lower the groundwater surface to below the sideslope excavation grades. Groundwater collected by the dewatering system will drain to a sump where water will be removed by a submersible pump and pumped to the perimeter stormwater management system, consistent with the TPDES permit for this site, collected for irrigation, or used for dust control. If groundwater contamination is detected at any of the wells upgradient to the dewatering system during routine monitoring events, the groundwater removed by the dewatering system will be tested for the detected constituents to ensure that the surface water discharge standard for that particular chemical constituent is met prior to discharge. The pumps will be activated upon installation of the dewatering system and will remain operational until the BER is approved. The pumps will be operated automatically by pressure transducers. Control levels for the automatic pump will be set to maintain sump liquid levels below the top of the dewatering sump.

The dewatering system will remain operational until enough ballast is placed in the form of protective cover, solid waste, and/or final cover over the impacted area. Once sufficient ballast is in place, and with the approval of TCEQ, the dewatering system will be decommissioned. A different hydrostatic pressure relief system may be used at the site if it is designed using the same methodology as the design included in Appendix IIID-C (e.g., relieve potential hydrostatic uplift pressure that may develop on the geomembrane liner) and approved by TCEQ through a permit modification. If during future cell design the conditions are such that a different system (e.g. vertical dewatering wells or a combination of options) is considered more efficient, the system design will be submitted to TCEQ for approval as a permit modification to the LQCP.

During construction of Sector 7, the POR will verify that the Lower Clay Stratum will be stable and will not experience uplift due to the watering bearing zone below the Lower Clay Stratum. The POR will monitor the groundwater level in WCP-5 on a weekly basis starting at a minimum 2 months prior to construction until after liner construction is completed. A demonstration showing that the Lower Clay Stratum did not experience uplift will be included in the GLER for Sector 7. Survey information reported by the POR demonstrating that uplift did not occur in the Lower Clay Stratum will be included in the GLER.

5.3.1 Dewatering System Design

The proposed dewatering system is comprised of a double-sided drainage geocomposite installed on the sideslopes of the excavation beginning at 1 foot above the highest measured groundwater elevations. The geocomposite discharges water into gravel backfilled trenches located at the toe of the sideslope. Groundwater contacting the geocomposite drains into the toe trench, then flows by gravity to the underdrain sump installed immediately below the Cell 7 and 8 leachate sump. A 3-inch diameter SDR 17 HDPE perforated pipe provides a conduit for drainage from the sideslope to the sump. A submersible pump is placed into the groundwater sump, and groundwater is pumped from the sump into the perimeter stormwater channel or ditch, collected for irrigation, or used for dust control. The location of the dewatering trench is shown on Sheet IIID-C-3 (refer to Appendix IIID-C for additional information).

5.4 Control of Seepage During Construction

Seepage of free water from the exposed soils is not expected during liner construction due to the temporary dewatering system that will be in place before liner construction is initiated. During construction, the subgrade must be maintained in a firm and unyielding condition to provide a satisfactory foundation for construction of the soil liner. If unexpected seepage is encountered, the wet soils will be over-excavated and replaced with compacted general fill (as described in Section 2.3.3) to seal off the seepage. Soft areas will be undercut to firm material and refilled with suitable compacted general fill. The fill will be free from organics, foreign objects, and other deleterious matter. The fill will also be compacted sufficiently to provide a firm base for soil liner placement. The temporary dewatering system has been designed to both lower the groundwater potentiometric surface to below the excavation grades and to prevent potential uplift of the liner system

5.5 Dewatering System Design

5.5.1 Documentation

The dewatering system installation will be incorporated into the SLER for each cell. in accordance with Section 6. The installed dewatering system will be operated until a Ballast Evaluation Report (BER) prepared in accordance with Section 6.3 is approved by the TCEQ.

5.6 Liner System Ballast

Ballasting is required to protect the liner system from hydrostatic uplift pressures in areas of the landfill that excavations will extend below the highest measured groundwater table. The protective cover soil above the liner system and the waste placed above the liner system will provide the necessary ballast (weight) for protection of the liner system from hydrostatic uplift.

The factor of safety against hydrostatic uplift must be calculated for those portions of the liner where the liner is below the highest measured groundwater table. The calculated factor of safety against uplift at the geomembrane liner (using the weight of the protective cover and waste only) must be 1.5 or greater. The thickness of protective cover and waste required to ballast against hydrostatic uplift must be calculated and submitted with the GLER. Procedures for calculating the anticipated hydrostatic uplift forces, factor of safety against uplift, and required thicknesses of ballast are included in Appendix IIID-B. Additionally, example ballast calculations are included in Appendix IIID-B. The most recent highest measured groundwater table data as defined in Section 5.2 will be used for ballast calculation. The ballast demonstration included in Appendix IIID-B must be updated each time a dewatering system is installed to account for possible higher hydrostatic head measurements.

5.6.1 Waste-As-Ballast Placement Record

When waste is used for ballast, landfill personnel working under the supervision of the site manager will be on site full-time during the placement of the first 5 feet of waste over the liner system. The site manager will verify and document on a daily basis that this lower 5 feet of waste does not contain large bulky items, which cannot be compacted to the required density. The site operator will also document on a daily basis that the waste used for ballast has been properly compacted with compaction equipment which weighs in excess of 40,000 pounds.

The site manager will complete and sign a waste-as-ballast placement record that will be attached to the BER (see Section 6 for BER required documentation), and maintained with the site operating record. The form to be used by the site manager is included in Appendix IIID-D. One form will be required for each area (or combination of areas) described by each approved GLERs.

5.7 Liner Performance Verification

For a liner requiring ballasting, the POR will verify that the ballast meets the established criteria and uplift of the liner system did not occur during construction or waste filling. The verification will include (but not limited to) site inspections, estimates of waste compaction and unit weight, and ballast thickness measurements will be documented in the BER, which will be submitted to the TCEQ for approval (see Section 6). In the event that uplift is detected, the POR will develop a corrective action plan to evaluate and potentially remediate the uplift. The POR and operator will immediately contact the TCEQ and implement initial procedures as soon as the uplift is detected.

5.7.1 Observations for Indications of Seepage

The POR will observe the liner subgrade for the presence of seepage during construction. To aid in the documentation that short-term uplift has not occurred during ballast placement, the POR will provide a summary of where seepage, if any, was observed, the methods and procedures used to control the seepage, and observations that seepage has been controlled.

5.7.2 Surveying During Construction

To document that short-term uplift has not occurred during construction of the liner, the POR will verify that the geomembrane liner elevations are consistent with the top of soil liner elevations surveyed immediately following soil liner installation. The POR will also verify that the protective cover elevations have not increased from those submitted with the SLER. The protective cover elevations will be surveyed once in the period between the SLER approval and initiation of waste placement to document no short-term uplift has occurred. Survey measurements to check against uplift will be taken at the same points established over the top of the protective cover layer following completion of protective cover installation.

5.8 Documentation

Documentation for issues related to construction below the high water table will be included in the SLER and BER. These documents are discussed in detail in Section 6.

6 DOCUMENTATION

The CQA plan depends on thorough monitoring and documentation of all construction activities. Therefore, the POR and CQA monitor will document that all quality assurance requirements have been addressed and satisfied. Documentation will consist of daily recordkeeping, testing and installation reports, nonconformance reports (if necessary), progress reports, photographic records, and design and specification revisions. The appropriate documentation will be included in the SLER, GLER, and BER (if required). Standard report forms will be provided by the POR prior to construction.

6.1 Preparation of SLER and GLER

The POR will submit to the TCEQ a SLER for review and acceptance of each soil liner portion of the composite liner. After construction of the geosynthetics portion of the liner, the POR will submit a GLER to the TCEQ for review and acceptance. All of these reports will be approved by TCEQ prior to placement of solid waste over the specified constructed area.

Testing, evaluation, and submission of the SLERs and GLERs for the composite liner system will be in accordance with this LQCP. The construction methods and test procedures documented in the SLERs and GLERs will be consistent with this LQCP, the TCEQ MSWR, and project construction drawings and specifications.

At a minimum, the SLER and GLER will contain:

- A summary of all construction activities.
- A summary of all laboratory and field test results.
- Sampling and testing location drawings.
- A description of significant construction problems and the resolution of these problems.
- As-built record drawings signed and sealed by a licensed Texas land surveyor.
- A statement of compliance with the permit LQCP and construction plans.
- The reports will be signed and stamped by a professional engineer(s) licensed to practice in the state of Texas.

The as-built record drawings will accurately identify the constructed location of all work items, including the piping and anchor trenches. The POR will review and verify that as-built drawings are correct. As-built drawings will be included in the SLER and GLER as appropriate.

6.2 Reporting Requirements

The SLER and GLER will be signed and sealed by the POR and signed by the operator and submitted in triplicate (including all attachments) to the MSW Permits Section of the Waste Permits Division of the TCEQ for review and acceptance. If the executive director provides no response, either written or verbal, within 14 days of receipt, the owner or operator may continue facility construction or operation. Any notice of deficiency received from the TCEQ will be promptly addressed and incorporated into the SLER/GLER report. No solid waste will be placed over the constructed liner areas until the final acceptance is obtained from the TCEQ. Additionally, upon approval of this application if a new liner area is developed, prior to accepting any solid waste to the newly developed liner area, a pre-opening inspection will be requested. The TCEQ staff will conduct a pre-opening inspection within 14 days of the request. If the TCEQ does not provide a written or verbal response 14 days after conducting the pre-opening inspection, the newly developed liner area will be considered acceptable for solid waste placement, given that the SLER and GLER for the area are also submitted to the TCEQ in accordance with this section.

If a layer of waste is not placed over the top of the protective cover in the dewatering system installation area within 6 months, then the POR will visually observe that the liner is not damaged (e.g., excessive erosion) due to prolonged exposure of the surface of the protective cover. Repairs will be done promptly and the POR will report findings and measures taken to repair damage in a letter report to the executive director for review and acceptance.

6.3 Ballast Evaluation Report

A BER will be completed and filed with the TCEQ in duplicate when it is determined that ballasting and dewatering is no longer necessary. The report will provide documentation that enough ballast has been placed in a lined area to offset the potential hydrostatic uplift forces which may exist below the liner system. The report will include (1) verification that the liner did not undergo uplift during construction, using the method identified in the liner quality control plan; (2) certification that ballast met the criteria established in this liner quality control plan; and (3) signature and seal of an independent licensed professional engineer performing the evaluation and signature of the facility operator or his authorized representative.

At a minimum, the information listed below will be included as applicable with the BER:

- Description of the dewatering system installed.
- The top of protective cover elevations immediately after construction compared to the elevations obtained between SLER approval and waste placement, to document the liner did not undergo uplift prior to placement of waste (whether waste ballast is required or not).
- If waste is used for ballast, verification from the site manager that the weight of the compaction equipment being used to compact the waste ballast is no less than 40,000 pounds, and that this compaction equipment was utilized during the entire period of placing waste ballast.
- If waste is used for ballast, documentation of the observations that the initial 5 feet of waste used for ballast on the liner system is free of brush and large bulky items, which may not be compacted to the required density.
- A waste-as-ballast placement record (Appendix IIID-D) completed and signed by the site manager.
- Survey of the top of waste to document that the required waste ballast thickness has been placed.
- Water level measurements taken in the site monitor well/piezometer system adjacent to the liner construction area to verify that the groundwater level has not exceeded the design high water level. A discussion addressing the areas where the bottom of the soil liner extends below the highest measured potentiometric surface, and an updated highest measured potentiometric surface map will be included.
- Final ballast thickness calculation using procedures included in Appendix IIID-B and the as-built minimum densities and thicknesses for each component as well as updated groundwater levels.

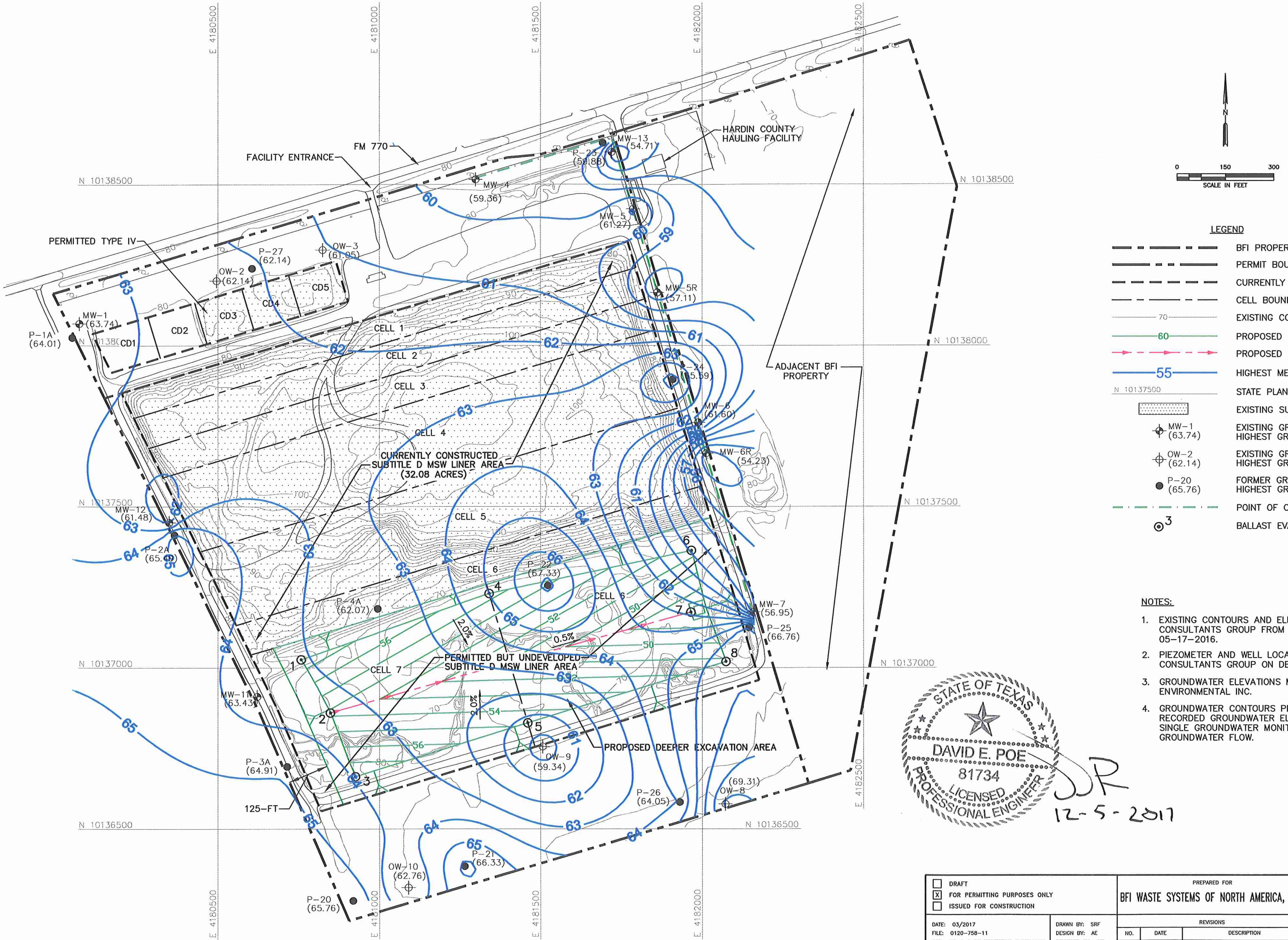
If the executive director provides no response within 14 days of the date of receipt, the owner or operator may discontinue dewatering or ballasting operations.

APPENDIX IIID-A

HIGHEST MEASURED GROUNDWATER INFORMATION

STATE OF TEXAS
DAVID E. POE
81734
LICENSED
PROFESSIONAL ENGINEER

12-5-2017



LEGEND

— - — - —	BFI PROPERTY BOUNDARY
— - - —	PERMIT BOUNDARY
— — — —	CURRENTLY PERMITTED LIMITS OF WASTE
— - — -	CELL BOUNDARY
—70—	EXISTING CONTOUR
—60—	PROPOSED EXCAVATION CONTOUR
→ - → - →	PROPOSED LEACHATE COLLECTION LINE
—55—	HIGHEST MEASURED GROUNDWATER CONTOUR
N 10137500	STATE PLANE COORDINATE GRID
▨	EXISTING SUBTITLE-D LINER AREA
⌖ MW-1 (63.74)	EXISTING GROUNDWATER MONITOR WELL WITH MEASURED HIGHEST GROUNDWATER ELEVATION POSTED IN FT-MSL
⌖ OW-2 (62.14)	EXISTING GROUNDWATER OBSERVATION WELL WITH MEASURED HIGHEST GROUNDWATER ELEVATION POSTED IN FT-MSL
● P-20 (65.76)	FORMER GROUNDWATER PIEZOMETER WITH MEASURED HIGHEST GROUNDWATER ELEVATION POSTED IN FT-MSL
— · — · —	POINT OF COMPLIANCE
⊙3	BALLAST EVALUATION POINT

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS DEVELOPED BY WEAVER CONSULTANTS GROUP FROM AERIAL PHOTOGRAPHY FLOWN 05-17-2016.
2. PIEZOMETER AND WELL LOCATION COORDINATES SURVEYED BY WEAVER CONSULTANTS GROUP ON DECEMBER 28, 2016.
3. GROUNDWATER ELEVATIONS MEASURED BY SWL AND HYDREX ENVIRONMENTAL INC.
4. GROUNDWATER CONTOURS PRODUCED USING EACH WELL'S HIGHEST RECORDED GROUNDWATER ELEVATION AND DO NOT REPRESENT A SINGLE GROUNDWATER MONITORING EVENT OR ACTUAL GROUNDWATER FLOW.

DAVID E. POE
81734

12-5-2017

☐ DRAFT
☒ FOR PERMITTING PURPOSES ONLY
☐ ISSUED FOR CONSTRUCTION

DATE: 03/2017
FILE: 0120-758-11
CAD: IIID-A-1_GW MONITORING SYSTEM.DWG

DRAWN BY: SRF
DESIGN BY: AE
REVIEWED BY: NT

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

PREPARED FOR
BFI WASTE SYSTEMS OF NORTH AMERICA, LLC

REVISIONS

NO.	DATE	DESCRIPTION
1	08/2017	FIRST NOD RESPONSE
2	11/2017	OWNERSHIP CHANGE

MAJOR PERMIT AMENDMENT HIGHEST MEASURED GROUNDWATER CONTOUR MAP

HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS

WWW.WCGRP.COM

FIGURE IIID-A-1

0:\0120\758\2214B EXPANSION\IIID\IIID-A-1 HIGHEST MEASURED GW MAP.dwg, 11/15/2017 8:13:46 AM, rsellers, 1:2

APPENDIX IIID-B

EXAMPLE BALLAST THICKNESS CALCULATIONS

Includes pages IIID-B-1 through IIID-B-9

STATE OF TEXAS
DAVID E. POE
81734
LICENSED
PROFESSIONAL ENGINEER

12-5-2017

BALLAST THICKNESS CALCULATION

The ballast requirements evaluated in this appendix are based on the current highest measured groundwater surface shown on Drawing IIID-A-1. As discussed in Section 5.6, the ballast calculations are performed assuming that the maximum hydrostatic uplift pressure acts at the geomembrane liner. The evaluation points on the sideslopes were calculated in the vertical and the normal directions.

The actual thickness of ballast required must be calculated and submitted with the Soil Liner Evaluation Report (SLER). A summary of the procedure, which will be used to calculate ballast thickness, is discussed below. Example calculations are also presented on pages IIID-B-4 through IIID-B-9. The lined area may be divided into smaller subareas to determine the ballast requirements. The thickness of ballast required will be calculated using the following methodology:

A. The highest groundwater potentiometric surface elevations will be determined from the updated water level data as discussed in Section 5.2 and illustrated in Appendix IIID-A.

At each evaluation point, determine the maximum hydrostatic uplift pressures acting at the geomembrane liner. Evaluation points are shown on Figure IIID-B-5.

At each evaluation point, determine the uplift pressure acting on the geomembrane liner using the unit weight of water times the vertical distance from the geomembrane liner to the highest measured water table.

$$P_{H2O}=\gamma_{H2O}*H$$

where:
γ_{H2O} = unit weight of water (pcf)
H = vertical distance from the bottom of the liner (ft)
P_{H2O} = uplift pressure on the base of the liner (psf)

B. At each evaluation point, determine the resisting pressure for both vertical and normal directions on sideslopes against uplift.

Determine the vertical and normal resisting pressure at the evaluation points using the unit weight of the protective cover layer times the thickness of the protective cover layer.

$$\sum R_{i,v} = \sum(\gamma_i{*}T_{i,})$$

where: $T_{i,v}$ = thickness of ballast component (e.g., protective cover) in vertical direction

γ_i = unit weight (pcf) of ballast component

$R_{i,v}$ = resisting pressure (psf) provided by each ballast component in vertical direction

$$\sum R_{i,n} = \Sigma(\gamma_i{*}T_{i,n})$$

where: $T_{i,n}$ = thickness of ballast component (protective cover) in normal direction

γ_i = unit weight (pcf) of ballast component (protective cover)

$R_{i,n}$ = resisting pressure (psf) provided by each ballast component (protective cover) in normal direction

C. Evaluate the factor of safety in the vertical and normal direction at each evaluation point as a ratio of the total resisting pressure to uplift pressure.

The factor of safety (FS) against uplift due to the hydrostatic pressure acting at the geomembrane liner in the vertical and normal direction is calculated as the resisting pressure determined in B divided by the uplift pressure determined in A.

$$FS_v = \sum R_{i,v} / P_{H2O}$$

$$FS_n = \sum R_{i,n} / P_{H2O}$$

If the factor of safety is less than 1.2, additional ballast will be necessary to offset the hydrostatic forces. See Section D for determining the thickness of additional ballast if necessary.

D. Determine the additional ballast necessary to offset hydrostatic pressures acting at the bottom of the liner in the vertical and normal direction.

If the factor of safety calculated in Section C is less than 1.2, determine the thickness of additional ballast in the form of waste (T_{waste}) in the vertical and normal direction to offset the hydrostatic uplift pressure at the evaluation point.

Use a factor of safety of 1.5 against uplift pressure when utilizing solid waste and protective cover.

Use a unit weight of 1200 lb/cy for in-place solid waste per Title 30 TAC §330.337(h)(2).

Calculate the minimum required waste column thickness that provides additional ballast to offset the hydrostatic uplift pressure with a factor of safety of 1.5 in the vertical direction.

$$R_{waste,v} = \gamma_{waste} * T_{waste,v}$$

where: $T_{waste,v}$ = waste thickness (ft) in vertical direction

γ_{waste} = unit weight of waste (pcf)

$R_{waste,v}$ = resisting pressure of waste (psf) in vertical direction

$$P_{H2O} = \frac{\sum R_{i,v}}{1.5} + \frac{R_{waste,v}}{1.5}$$

Substituting appropriate values and solving for height of waste in the vertical direction:

$$T_{waste,v} = \frac{1.5}{\gamma_{waste}} * \left(P_{H2O} - \frac{\sum R_{i,v}}{1.5} \right)$$

Calculate the minimum required waste column thickness that provides additional ballast to offset the hydrostatic uplift pressure with a factor of safety of 1.5 in the normal direction.

$$R_{waste,n} = \gamma_{waste} * T_{waste,n}$$

where: $T_{waste,n}$ = waste thickness (ft) in normal direction

γ_{waste} = unit weight of waste (pcf)

$R_{waste,n}$ = resisting pressure of waste (psf) in normal direction

$$P_{H2O} = \frac{\sum R_{i,n}}{1.5} + \frac{R_{waste,n}}{1.5}$$

Substituting the appropriate values and solving for height of waste in the normal direction:

$$T_{waste,n} = \frac{1.5}{\gamma_{waste}} * \left(P_{H2O} - \frac{\sum R_{i,n}}{1.5} \right)$$

If waste and protective cover do not provide enough ballast against uplift, final cover will be used for ballast with a factor of safety of 1.5.

HARDIN COUNTY LANDFILL
0120-758-11-02
EXAMPLE BALLAST THICKNESS CALCULATIONS
EVALUATION OF SIDEWALL OF LINER

Required: Provide example calculations to be used to estimate the amount of ballast required for the sidewall of the liner prior to decommissioning the dewatering system.

Solution: Estimate the amount of ballast needed for the sidewall of the liner.

An example calculation using Evaluation Point No. 2 is shown below. A summary of the calculation results for each evaluation point located on the liner side slopes is shown on Sheet IIID-B-8. Sheet IIID-B-9 shows the location of the evaluation points and the top of waste elevation required for ballast at each evaluation point.

Definition of terms/variables:

H = Maximum groundwater head above top of clay liner, ft
P_{H20} = Maximum uplift pressure created by groundwater head, psf
$R_{pc,\,v}$ = Counteracting ballast pressure from protective cover - vertical, psf
$R_{pc,\,n}$ = Counteracting ballast pressure from protective cover - normal, psf
E_{H20} = Highest potentiometric surface elevation, ft-msl
E_{liner} = Elevation of top of clay liner (geomembrane), ft-msl
$E_{waste,\,v}$ = Required top of waste elevation needed for ballast - vertical, ft-msl
$E_{waste,\,n}$ = Required top of waste elevation needed for ballast - normal, ft-msl
γ_{H20} = Unit weight of water, pcf
γ_{pc} = Unit weight of protective cover, pcf
γ_{waste} = Unit weight of waste, lb/cy (Assumed to be 1200 lb/cy per 30 TAC Section 330.337(h)(2).)
$T_{pc,\,v}$ = Thickness of protective cover as ballast - vertical, ft
$T_{pc,\,n}$ = Thickness of protective cover as ballast - normal, ft
$T_{waste,\,v}$ = Required waste thickness needed for ballast - vertical, ft
$T_{waste,\,n}$ = Required waste thickness needed for ballast - normal, ft
$E_{pc,v}$ = Elevation of top of protective cover - vertical, ft-msl
$E_{pc,n}$ = Elevation of top of protective cover - normal, ft-msl
$FS_{pc,\,v}$ = Calculated factor of safety with protective cover installed - vertical
$FS_{pc,\,n}$ = Calculated factor of safety with protective cover installed - normal
$E_{fc,\,v}$ = Design top of final cover elevation - vertical, ft-msl
$E_{fc,\,n}$ = Design top of final cover elevation - normal, ft-msl

(Refer to Appendix IIIA for Landfill Completion Plan)

$E_{top\ waste,\,v}$ = Design top of waste elevation - vertical, ft-msl
$E_{top\ waste,\,n}$ = Design top of waste elevation - normal, ft-msl
T_{fc} = Approximate thickness of final cover, ft (note this thickness is assumed the same for the vertical and normal directions)

Prep By: DEP
Date: 12/5/2017

Chkd By: NT
Date: 12/5/2017

HARDIN COUNTY LANDFILL
0120-758-11-02
EXAMPLE BALLAST THICKNESS CALCULATIONS
EVALUATION OF SIDEWALL OF LINER

Example calculation using Evaluation Point No. 2

Parameters:

$E_{H20} =$ 63.4 ft-msl
$E_{liner} =$ 59.0 ft-msl
$\gamma_{H20} =$ 62.4 pcf
β = side slope angle = 18.43
$\cos \beta =$ 0.9487
$T_{pc,\,v} =$ 2.1 ft ($T_{pc,v}/\cos \beta$)
$T_{pc,\,n} =$ 2.0 ft

$\gamma_{pc} =$ 116 pcf
$\gamma_{waste} =$ 1200 lb/cy
$E_{fc,\,v} =$ 110.0 ft-msl
$E_{fc,\,n} =$ 110.0 ft-msl
$T_{fc} =$ 3.5 ft

Calculate the maximum groundwater head above the top of clay liner.

$H = E_{H20} - E_{liner}$
$H =$ 4.4 ft

Calculate the maximum hydrostatic uplift pressure created by the groundwater head.

$P_{H2O} = (\gamma_{H2O} \times H)$
$P_{H2O} =$ 275 psf

Calculate the counteracting ballast pressure from the protective cover in the vertical and normal directions.

$R_{pc,\,v} = (\gamma_{pc} \times T_{pc,\,v})$
$R_{pc,\,v} =$ 245 psf

$R_{pc,\,n} = (\gamma_{pc} \times T_{pc,\,n})$
$R_{pc,\,n} =$ 232 psf

Compare the uplift pressure to the ballast pressure by calculating the factors of safety in the vertical and normal direction with protective cover as ballast at the evaluation point.

$FS_{pc,\,v} = R_{pc,\,v}/P_{H2O} =$ 0.9

$FS_{pc,\,n} = R_{pc,\,n}/P_{H2O} =$ 0.8

The minimum required factor of safety for protective cover as ballast is 1.2. Since the factor of safety against uplift is less than 1.2 additional ballast (in the form of waste) will be necessary to counteract the hydrostatic uplift pressure acting at the top of clay liner (geomembrane). If the factor of safety against uplift was 1.2 or greater, then no additional ballast would be necessary indicating that the protective cover provides enough ballast to counteract the hydrostatic uplift pressure acting at the top of clay liner. When solid waste is necessary as ballast, a factor of safety of 1.5 is used for protective cover and solid waste.

EXAMPLE BALLAST THICKNESS CALCULATIONS
EVALUATION OF SIDEWALL OF LINER

Determine amount of additional ballast in the form of waste necessary to offset the hydrostatic pressure acting at the top of clay liner in the vertical and normal direction. Use a factor of safety of 1.5 for protective cover and solid waste.

$T_{waste,\,v} = [(1.5 \text{ x } P_{H2O})\text{-}R_{pc,\,v}]/\gamma_{waste}$
$T_{waste,\,v} =$ 3.8 ft

$E_{waste,\,v} = E_{liner} + T_{pc,\,v} + T_{waste,\,v}$
$E_{waste,\,v} =$ 64.9 ft-msl

$T_{waste,\,n} = [(1.5 \text{ x } P_{H2O})\text{-}R_{pc,\,n}]/\gamma_{waste}$
$T_{waste,\,n} =$ 4.0 ft

$E_{waste,\,n} = E_{liner} + T_{pc,\,n} + T_{waste,\,n}$
$E_{waste,\,n} =$ 65.0 ft-msl

Check to verify that the required top of waste elevation is less than the design top of waste elevation in the vertical and normal direction.

$E_{top\ waste,\,v} = E_{fc,\,v} - T_{fc}$
$E_{top\ waste,\,v} =$ 106.5 ft-msl

$E_{top\ waste,\,v} > E_{waste,\,v}$
$106.5 > 64.9$

$E_{top\ waste,\,n} = E_{fc,\,n} - T_{fc}$
$E_{top\ waste,\,n} =$ 106.6 ft-msl

$E_{top\ waste,\,n} > E_{waste,\,n}$
$106.6 > 65.0$

The required top of waste elevation needed as ballast is less than the design top of waste elevation in the vertical and normal directions. Therefore, the design top of waste elevation allows for the required top of waste elevation needed for ballast in the vertical and normal directions. If the top of waste elevation did not provide enough ballast, then the final cover is used to provide additional ballast against uplift using a factor of safety of 1.5.

HARDIN COUNTY LANDFILL
0120-758-11-02
EXAMPLE BALLAST THICKNESS CALCULATIONS
EVALUATION OF SIDEWALL OF LINER

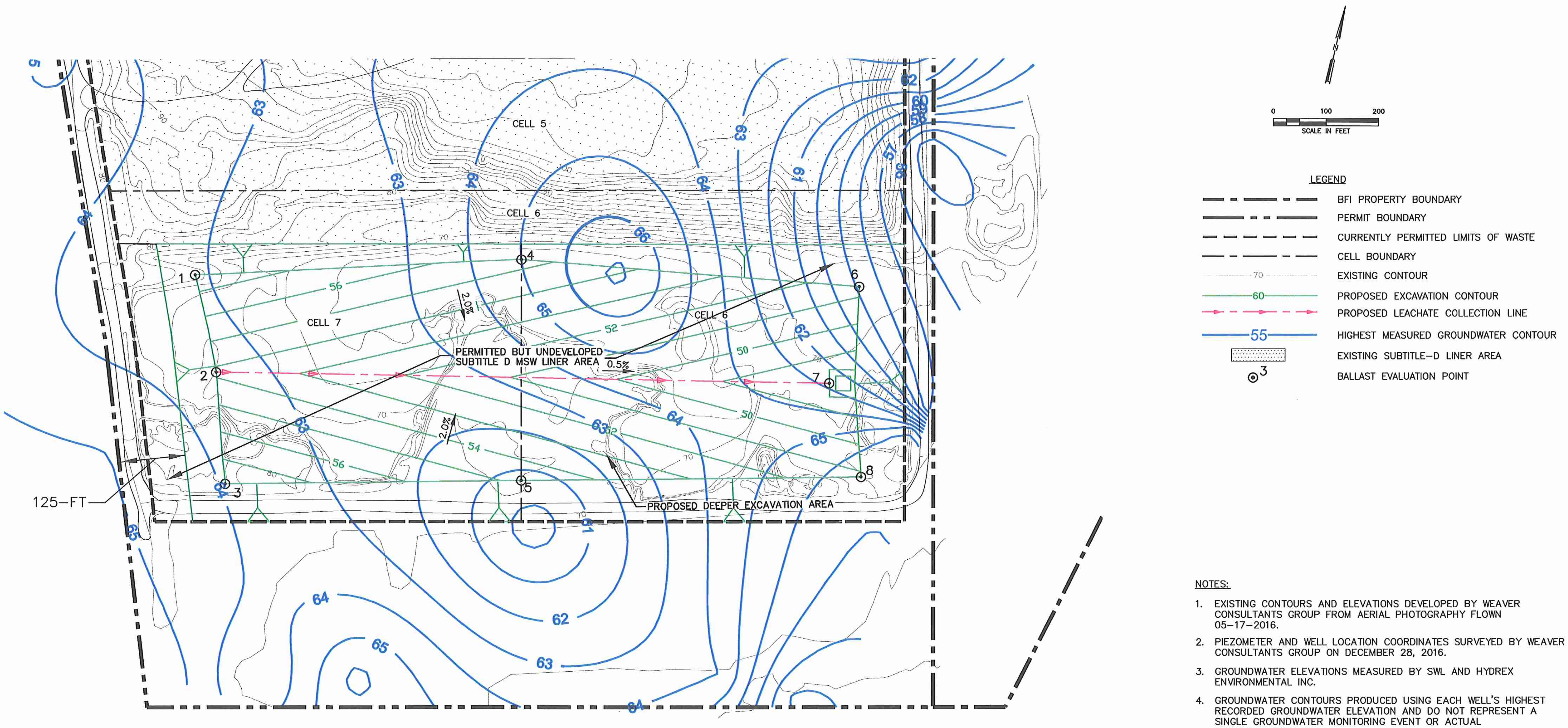
Unit Weight of Water = 62.4 pcf
Unit Weight of Protective Cover = 116 pcf
Unit Weight of Waste = 1200 pcy
Unit Weight of Final Cover = 116 pcf

Thickness of Protective Cover - Vertical = 2.1 ft
Thickness of Protective Cover - Normal = 2.0 ft
Thickness of Final Cover - Vertical = 3.6 ft
Thickness of Final Cover - Normal = 3.5 ft

Evaluation Point	Highest Potentiometric Surface Elevation^2 E_{H2O} (ft-msl)	Elevation of Top of Clay Liner E_{liner} (ft-msl)	Maximum Groundwater Head Above Top of Clay Liner H (ft)	Maximum Uplift Pressure Created by Groundwater Head P_{H2O} (psf)	Elevation of Top of Protective Cover - Vertical $E_{pc,v}$ (ft-msl)	Elevation of Top of Protective Cover - Normal $E_{pc,n}$ (ft-msl)	Counteracting Ballast Pressure from Protective Cover - Vertical $R_{pc,v}$ (psf)	Counteracting Ballast Pressure from Protective Cover - Normal $R_{pc,n}$ (psf)	Calculated Factor of Safety with Protective Cover Installed - Vertical	Calculated Factor of Safety with Protective Cover Installed - Normal	Factor of Safety - Vertical > 1.2?	Factor of Safety - Normal > 1.2?	Required Waste Thickness Needed for Ballast - Vertical $T_{wb,v}$ (ft)^1	Required Waste Thickness Needed for Ballast - Normal $T_{wb,n}$ (ft)^1	Required Top of Waste Elevation Needed for Ballast - Vertical $E_{wb,v}$ (ft-msl)	Required Top of Waste Elevation Needed for Ballast - Normal $E_{wb,n}$ (ft-msl)	Design Top of Waste Elevation - Vertical $E_{top\ waste,v}$ (ft-msl)	Design Top of Waste Elevation - Normal $E_{top\ waste,n}$ (ft-msl)	Required Waste Needed for Ballast Elevation < Design Top of Waste Elevation - Vertical?	Required Waste Needed for Ballast Elevation < Design Top of Waste Elevation - Normal?	Counteracting Ballast Pressure from Protective Cover, Waste, and Final Cover - Vertical $Rf_{c,v}$ (psf)	Counteracting Ballast Pressure from Protective Cover, Waste, and Final Cover - Normal $Rf_{c,n}$ (psf)	Calculated Factor of Safety with Final Cover Installed - Vertical	Calculated Factor of Safety with Final Cover Installed - Normal	Factor of Safety - Vertical > 1.5?	Factor of Safety - Normal > 1.5?
1	63.1	62.5	0.6	37	64.6	64.5	245	232	6.5	6.2	YES	YES	N/A	N/A	N/A	N/A	106.4	106.5	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
2	63.5	59.0	4.5	281	61.1	61.0	245	232	0.9	0.8	NO	NO	4.0	4.3	65.1	65.3	106.4	106.5	YES	YES	N/A	N/A	N/A	N/A	N/A	N/A
3	63.9	62.5	1.4	87	64.6	64.5	245	232	2.8	2.7	YES	YES	N/A	N/A	N/A	N/A	96.4	96.5	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
4	65.0	60.3	4.7	293	62.4	62.3	245	232	0.8	0.8	NO	NO	4.4	4.7	66.8	67.0	204.4	204.5	YES	YES	N/A	N/A	N/A	N/A	N/A	N/A
5	60.7	59.5	1.2	75	61.6	61.5	245	232	3.3	3.1	YES	YES	N/A	N/A	N/A	N/A	93.9	94.0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
6	59.0	56.0	3.0	187	58.1	58.0	245	232	1.3	1.2	YES	NO	N/A	1.1	N/A	59.1	93.9	94.0	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
7	62.8	53.0	9.8	612	55.1	55.0	245	232	0.4	0.4	NO	NO	15.1	15.4	70.2	70.4	93.9	94.0	YES	YES	N/A	N/A	N/A	N/A	N/A	N/A
8	66.4	56.0	10.4	649	58.1	58.0	245	232	0.4	0.4	NO	NO	16.4	16.7	74.5	74.7	93.9	94.0	YES	YES	N/A	N/A	N/A	N/A	N/A	N/A

potentiometric surface. The highest measured potentiometric surface can only be adjusted upward.

[2] Refer to Section 5.2 for discussion on highest measured groundwater. Also see Drawing IIID-A-1 in Appendix IIID-A.



NOTES:

1. EXISTING CONTOURS AND ELEVATIONS DEVELOPED BY WEAVER CONSULTANTS GROUP FROM AERIAL PHOTOGRAPHY FLOWN 05–17–2016.
2. PIEZOMETER AND WELL LOCATION COORDINATES SURVEYED BY WEAVER CONSULTANTS GROUP ON DECEMBER 28, 2016.
3. GROUNDWATER ELEVATIONS MEASURED BY SWL AND HYDREX ENVIRONMENTAL INC.
4. GROUNDWATER CONTOURS PRODUCED USING EACH WELL'S HIGHEST RECORDED GROUNDWATER ELEVATION AND DO NOT REPRESENT A SINGLE GROUNDWATER MONITORING EVENT OR ACTUAL GROUNDWATER FLOW.

DAVID E. POE
81734

12-5-2017

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DATE: 03/2017
FILE: 0120–758–11
CAD: IIID–B–9_BALLAST EVAL POINTS.DWG

DRAWN BY: SRF
DESIGN BY: AE
REVIEWED BY: NT

Weaver Consultants Group
TBPE REGISTRATION NO. F–3727

PREPARED FOR
BFI WASTE SYSTEMS OF NORTH AMERICA, LLC

REVISIONS

NO.	DATE	DESCRIPTION
1	08/2017	FIRST NOD RESPONSE
2	11/2017	OWNERSHIP CHANGE

MAJOR PERMIT AMENDMENT BALLAST EVALUATION POINTS

HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS

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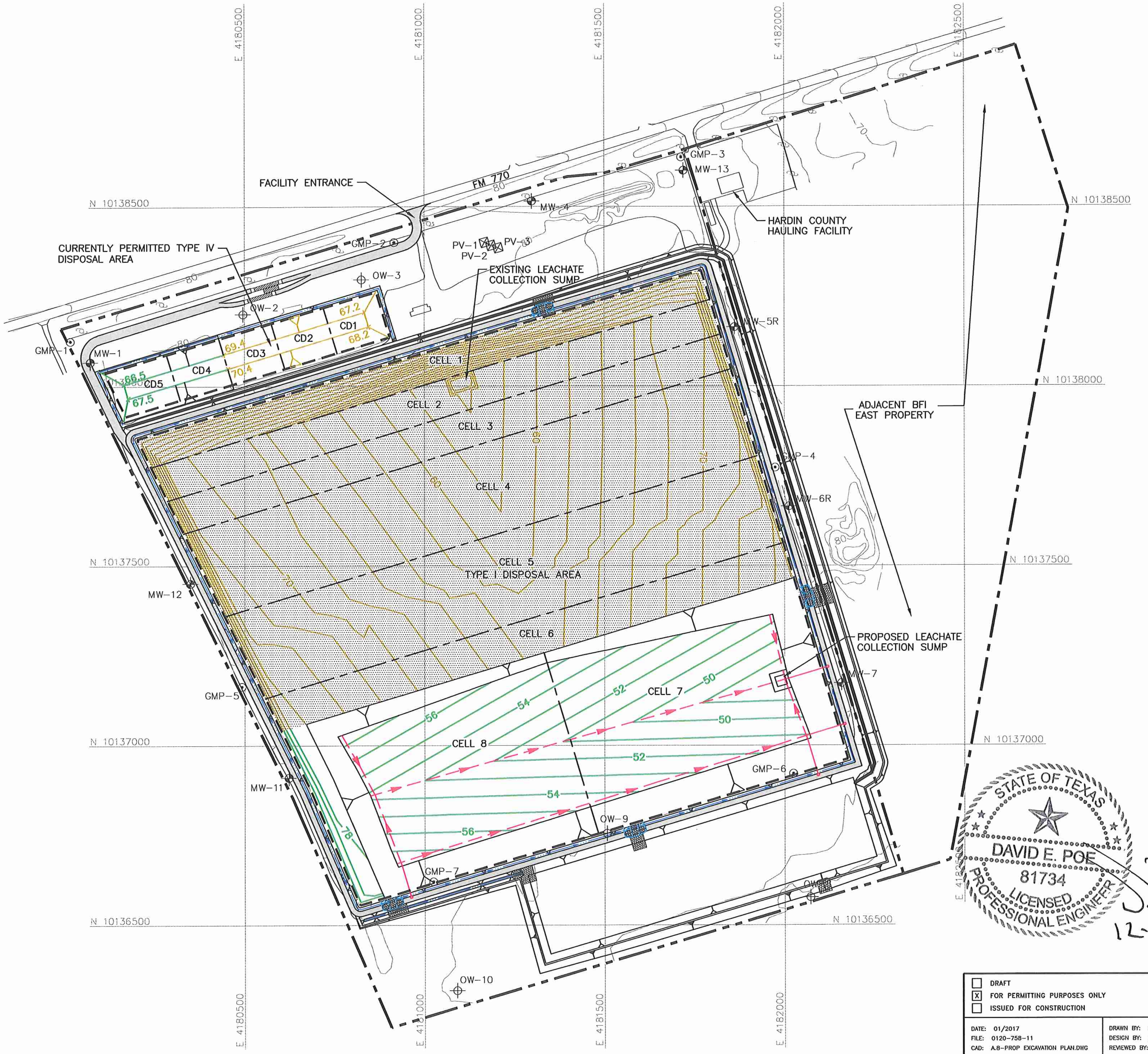
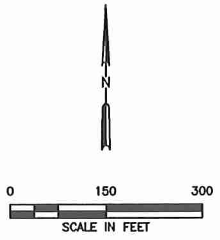
FIGURE IIID–B–9

O:\0120\758\2214B EXPANSION\IIID\IIID–B–9 BALLAST CALC POINTS.dwg, 11/15/2017 8:14:41 AM, rsellers, 1:2





LEGEND

- BFI EAST PROPERTY BOUNDARY
- PERMIT BOUNDARY
- CURRENTLY PERMITTED LIMITS OF WASTE
- 70 EXISTING CONTOUR (SEE NOTE 1)
- N 10137000 STATE PLANE COORDINATE SYSTEM (SEE NOTE 1)
- CELL BOUNDARY
- 50 PROPOSED EXCAVATION CONTOUR
- 70 CONSTRUCTED TOP OF PROTECTIVE COVER CONTOUR
- PROPOSED LEACHATE LINE
- PROPOSED LEACHATE RISER
- ▨ EXISTING SUBTITLE D COMPOSITE LINER AREA
- MW-1 EXISTING GROUNDWATER MONITOR WELL
- OW-2 EXISTING GROUNDWATER OBSERVATION WELL
- GMP-1 EXISTING GAS MONITORING PROBE
- $^{3}_{765.0}$ BALLAST EVALUATION POINT (SEE NOTE 5)
- 700 GROUNDWATER ELEVATION CONTOUR IN FT-MSL (SEE NOTE 4)

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS DEVELOPED BY WEAVER CONSULTANTS GROUP FROM AERIAL PHOTOGRAPHY FLOWN 05-17-2016. GRID SYSTEM IS TIED TO THE TEXAS STATE PLANE COORDINATE SYSTEM NAD 83, CENTRAL ZONE.
2. CURRENTLY PERMITTED TYPE IV CELLS INCLUDES CD1 THROUGH CD5. CD1, CD2, AND CD3 ARE CURRENTLY DEVELOPED AND RECEIVED TYPE IV WASTE.
3. THE PROPOSED EXCAVATION PLAN CONTOURS WERE DEVELOPED BY WEAVER CONSULTANTS GROUP AS PART OF THE DESIGN BASIS MEMORANDUM FOR THE PROPOSED MAJOR AMENDMENT APPLICATION.
4. GROUNDWATER CONTOURS WERE PRODUCED USING EACH WELL'S HIGHEST RECORDED GROUNDWATER ELEVATION AND DO NOT REPRESENT A SINGLE GROUNDWATER MONITORING EVENT OR ACTUAL GROUNDWATER FLOW.
5. BALLAST EVALUATION POINT IS SHOWN WITH CALCULATED TOP OF WASTE ELEVATION REQUIRED FOR BALLAST IN FT-MSL. N/A INDICATES THAT PROTECTIVE COVER IS SUFFICIENT BALLAST.

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12-5-2017

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DATE: 01/2017
FILE: 0120-758-11
CAD: A.8-PROP EXCAVATION PLAN.DWG

DRAWN BY: SRF
DESIGN BY: AE
REVIEWED BY: NT

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

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MAJOR PERMIT AMENDMENT BALLAST EVALUATION SITE PLAN

HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS

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DRAWING IIID-B-II

APPENDIX IIID-C

TEMPORARY DEWATERING SYSTEM DESIGN
CELLS 7 AND 8

Includes pages IIID-C-1 through IIID-C-30

STATE OF TEXAS
DAVID E. POE
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LICENSED PROFESSIONAL ENGINEER

12-5-2011

TEMPORARY DEWATERING SYSTEM DESIGN FOR CELLS 7 AND 8

This appendix includes the design of the temporary dewatering system that is designed to collect groundwater for the portions of the site that are excavated below the highest measured groundwater levels. This design is applicable to the undeveloped Cells 7 and 8 only. Groundwater collected by the dewatering system will drain to sump located as shown on Sheet IIID-C-3. Any groundwater collected in the sump will be pumped to the perimeter stormwater system and discharged from the site consistent with TPDES Stormwater Permit, used for irrigation, or used for dust control. The pumps will be activated upon installation of the dewatering system and will remain operational until the BER is approved. The pumps will be operated automatically by pressure transducers. Control levels for the automatic pump will be set to maintain sump liquid levels below the top of the sump. Calculations and design information for the dewatering system design are included in the following pages.

As shown on Sheet IIID-C-3, the groundwatering system design is comprised of a double-sided drainage geocomposite installed on the sideslopes of the excavation beginning 1 foot above the highest measured groundwater elevation. The drainage geocomposite collects groundwater and directs it to gravel backfilled trenches as shown on Sheet IIID-C-3. The trenches each contain a pipe which gravity drains the collected groundwater towards a sump located directly below the LCS sump.

The dewatering system has been designed to effectively reduce the groundwater levels below liner grades. A different hydrostatic pressure relief system may be used at the site if it is designed using the same methodology as the design included in this SLQCP (e.g., relieve potential hydrostatic uplift pressure that may develop on the geomembrane liner). If during future cell designs the conditions are such that a different system is considered more efficient, the system design will be submitted to TCEQ for approval as a permit modification.

This appendix includes design calculations for the temporary hydrostatic pressure relief system. As each cell is designed, water level data will be updated to ensure that the highest measured groundwater levels are considered for the design. Any changes in the system will be submitted to TCEQ for approval. Considering these design calculations, the following material performance specifications will be included in the project specifications.

Geocomposite Drainage Layer

The dewatering system geocomposite will consist of an HDPE geonet (200-mil thick) with a non-woven geotextile (minimum 6 oz/sy weight) heat bonded to both sides of the geonet.

Collection Pipe

The underdrain collection pipe will consist of 3-inch-diameter, SDR 17 HDPE perforated pipe.

Riser Pipe

Groundwater collected in the underdrain trench will be removed by a submersible pump placed down an 18-inch diameter, SDR-17 HDPE pipe. The pipe will be perforated in the sump and solid for the length extending up the sidewall above the sump.

Geotextile Around Drainage Stone

The geotextile around the drainage stone will have a weight of at least 8 oz/sy and meet the minimum requirements given in the following design.

HARDIN COUNTY LANDFILL
0120-758-11-02
APPENDIX IIID-C
TEMPORARY DEWATERING SYSTEM

Required:

The purpose of this appendix is to provide a temporary dewatering system below the composite liner system for the undeveloped portion of Cells 7 and 8. Design drawings and details depicting the layout of the dewatering system are provided on Sheets IIID-C-3 and IIID-C-4. Note that the system demonstration below conservatively assumes that the 4-foot thick sand layer in the cell sidewalls that is being controlled has constant recharge.

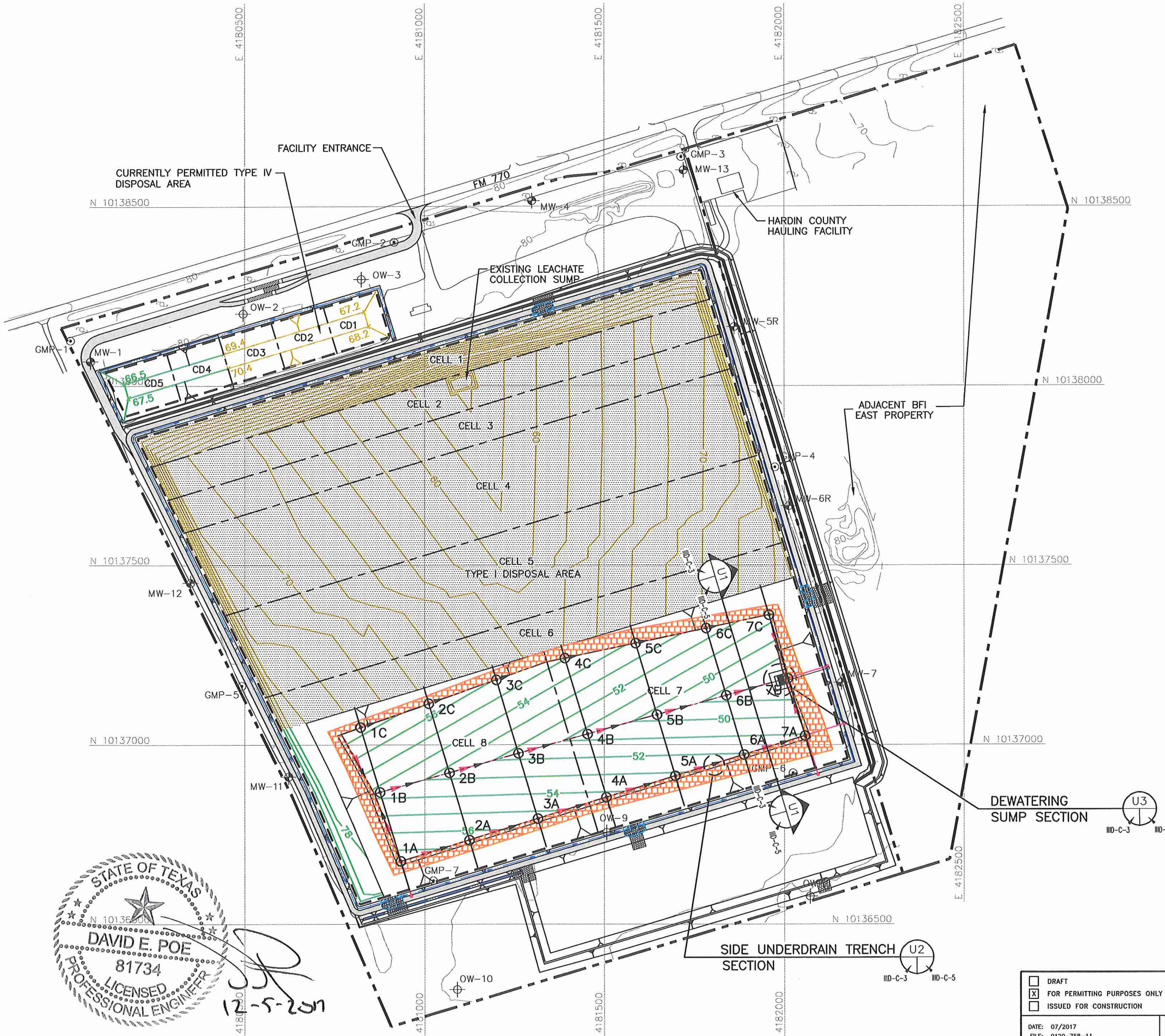
Method:

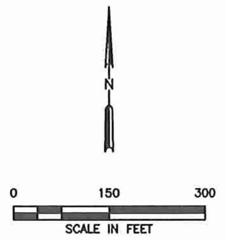
1. Estimate the seepage into the dewatering geocomposite.
2. Determine the required transmissivity of the dewatering geocomposite.
3. Determine the required ultimate transmissivity of the dewatering geocomposite.
4. Determine the required permeability of the dewatering geocomposite geotextile.
5. Design the geotextile over the drainage backfill.
6. Estimate the flow into the dewatering pipe.
7. Determine the flow capacity of the dewatering pipe.
8. Determine required pipe perforation based on characteristics of the surrounding drainage media.
9. Determine the sump size and pump capacity.

References:

1. Bass, J., *Avoiding Failure of Leachate Collection and Cap Drainage Systems*, Pollution Technology Review No. 138, Noyles Data Corporation, 1986
2. Texas Natural Resource Conservation Commission, *Leachate Collection System Handbook*, 30 TAC 330.201, 1993.
3. Koerner, R.M., *Designing with Geosynthetics*, Third Edition, Prentice Hall, Inc., 1994.
4. *Dewatering and Groundwater Control,* TM5-818-5, November 1983
5. Phillips 66 Driscopipe, System Design, 1991.
6. Heisler, Sanford I., P.E., Wiley Engineer's Desk Reference, John Wiley & Sons, Inc., New York, 1998.
7. GSE Drainage Design Manual, May 2004.
8. Acar, Yalcin B.& Daniel, David E., *Geoenvironment 2000 Characterization, Containment, Remediation, and Performance in Environmental Geotechnics,* Volume 2, American Society of Civil Engineers, 1995.
9. Gray, Donald H., Koerner, Robert M., Qian, Xuede, Geotechnical Aspects of Landfill Design and Construction, 2002.
10. Geosynthetic Institute, GRI Standard GC-8, 2001.
11. GRI White Paper #4, *Reduction Factors (RFs) Used in Geosynthetic Design,* Feb. 3, 2005, revised Mar. 1, 2007.

O:\0120\758\2214B EXPANSION\IIID\IIID-C-3_DEWATERING SYSTEM LAYOUT.dwg, 11/15/2017 8:17:03 AM, rsellers, 1:2





LEGEND

— — - — — -	BFI EAST PROPERTY BOUNDARY
——— - - ———	PERMIT BOUNDARY
— — — — —	CURRENTLY PERMITTED LIMITS OF WASTE
——70——	EXISTING CONTOUR (SEE NOTE 1)
N 10137000	STATE PLANE COORDINATE SYSTEM (SEE NOTE 1)
— — — —	CELL BOUNDARY
——50——	PROPOSED EXCAVATION CONTOUR
——70——	CONSTRUCTED TOP OF PROTECTIVE COVER CONTOUR
— — ◄ — —	PROPOSED LEACHATE LINE
——○	PROPOSED LEACHATE RISER
— - — ◄ — - —	PROPOSED DEWATERING LINE
——○	PROPOSED DEWATERING RISER
▨	EXISTING SUBTITLE D COMPOSITE LINER AREA
▨	DEWATERING DRAINAGE GEOCOMPOSITE AREA (SEE NOTE 3)
MW-1	EXISTING GROUNDWATER MONITOR WELL
OW-2	EXISTING GROUNDWATER OBSERVATION WELL
⊙ GMP-1	EXISTING GAS MONITORING PROBE
⊙ 1A	EVALUATION POINT

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS DEVELOPED BY WEAVER CONSULTANTS GROUP FROM AERIAL PHOTOGRAPHY FLOWN 05-17-2016. GRID SYSTEM IS TIED TO THE TEXAS STATE PLANE COORDINATE SYSTEM NAD 83, CENTRAL ZONE.
2. CURRENTLY PERMITTED TYPE IV CELLS INCLUDES CD1 THROUGH CD5. CD1, CD2, AND CD3 ARE CURRENTLY DEVELOPED AND RECEIVED TYPE IV WASTE.
3. DEWATERING DRAINAGE GEOCOMPOSITE AREA EXTENDS 1 FOOT ABOVE THE HIGHEST MEASURED GROUNDWATER SURFACE AS SHOWN ON DRAWING IIID-A-1.

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81734

12-5-2017

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DATE: 07/2017
FILE: 0120-758-11
CAD: IIID-C-3-DEWATERING LAYOUT.DWG

DRAWN BY: SRF
DESIGN BY: AE
REVIEWED BY: NT

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MAJOR PERMIT AMENDMENT DEWATERING SYSTEM LAYOUT

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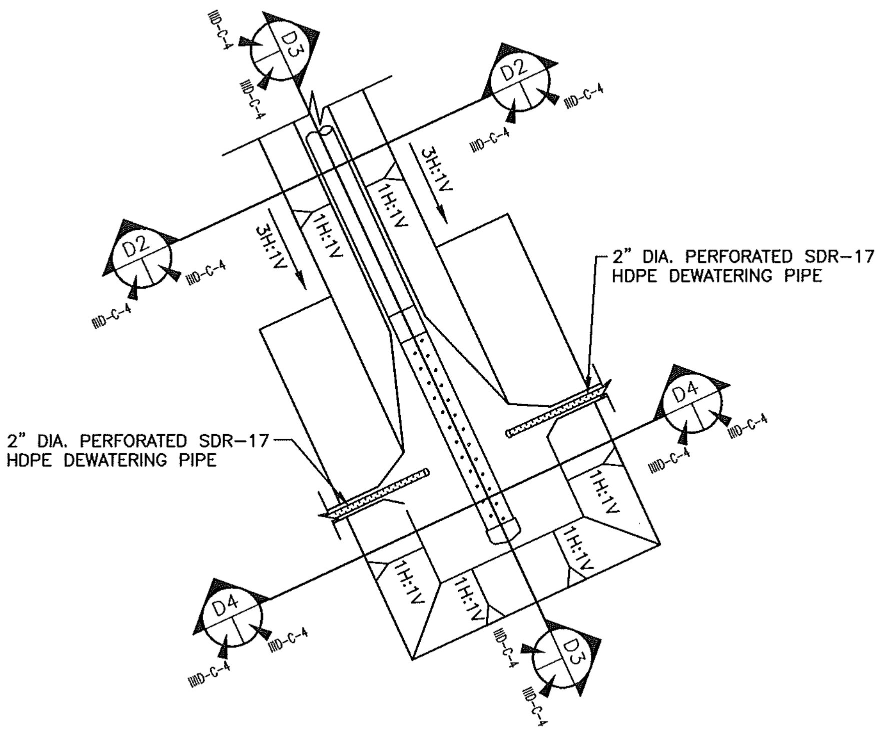
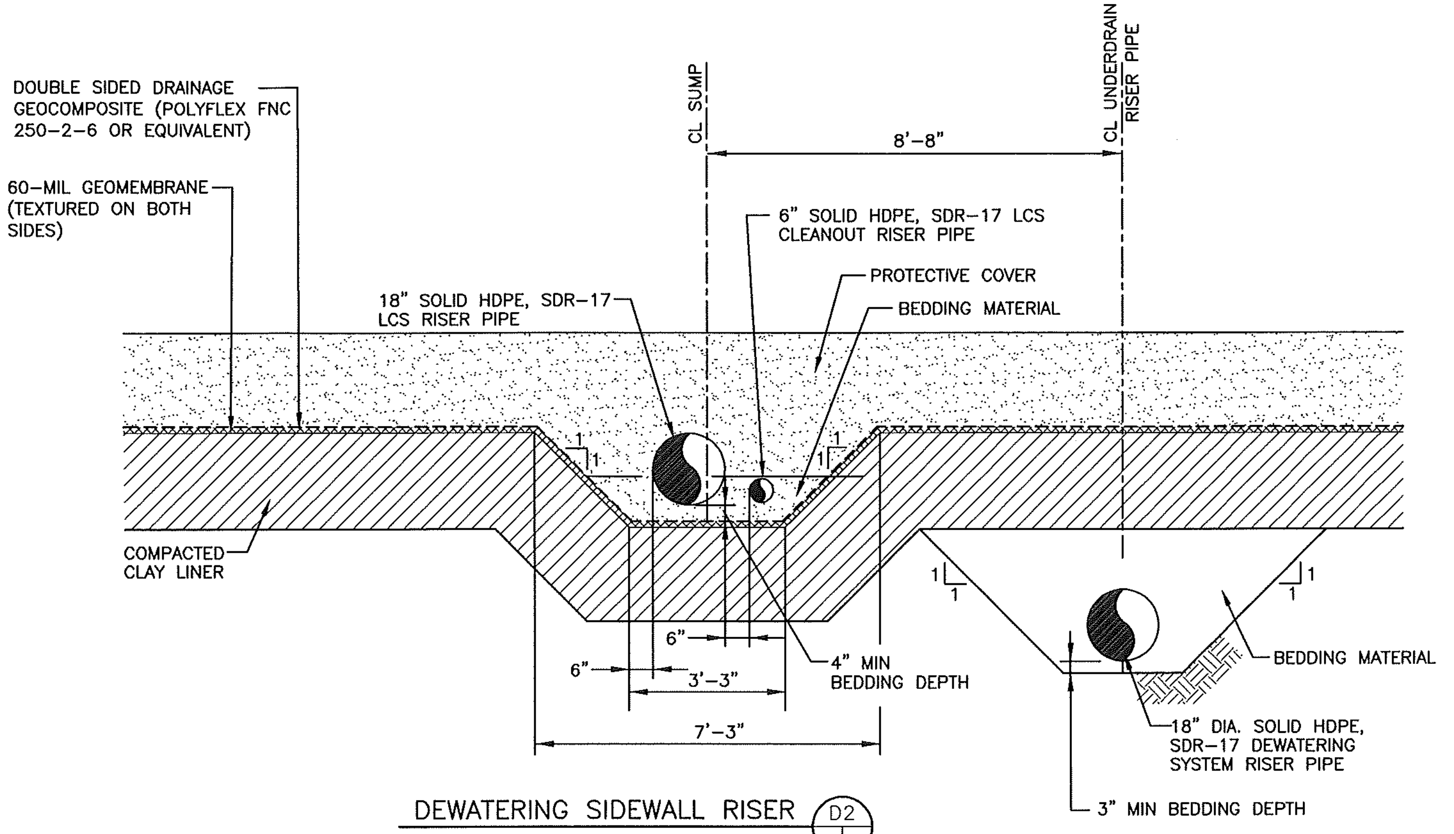
SHEET IIID-C-3



DEWATERING COLLECTION SUMP



DEWATERING SIDEWALL RISER

STATE OF TEXAS
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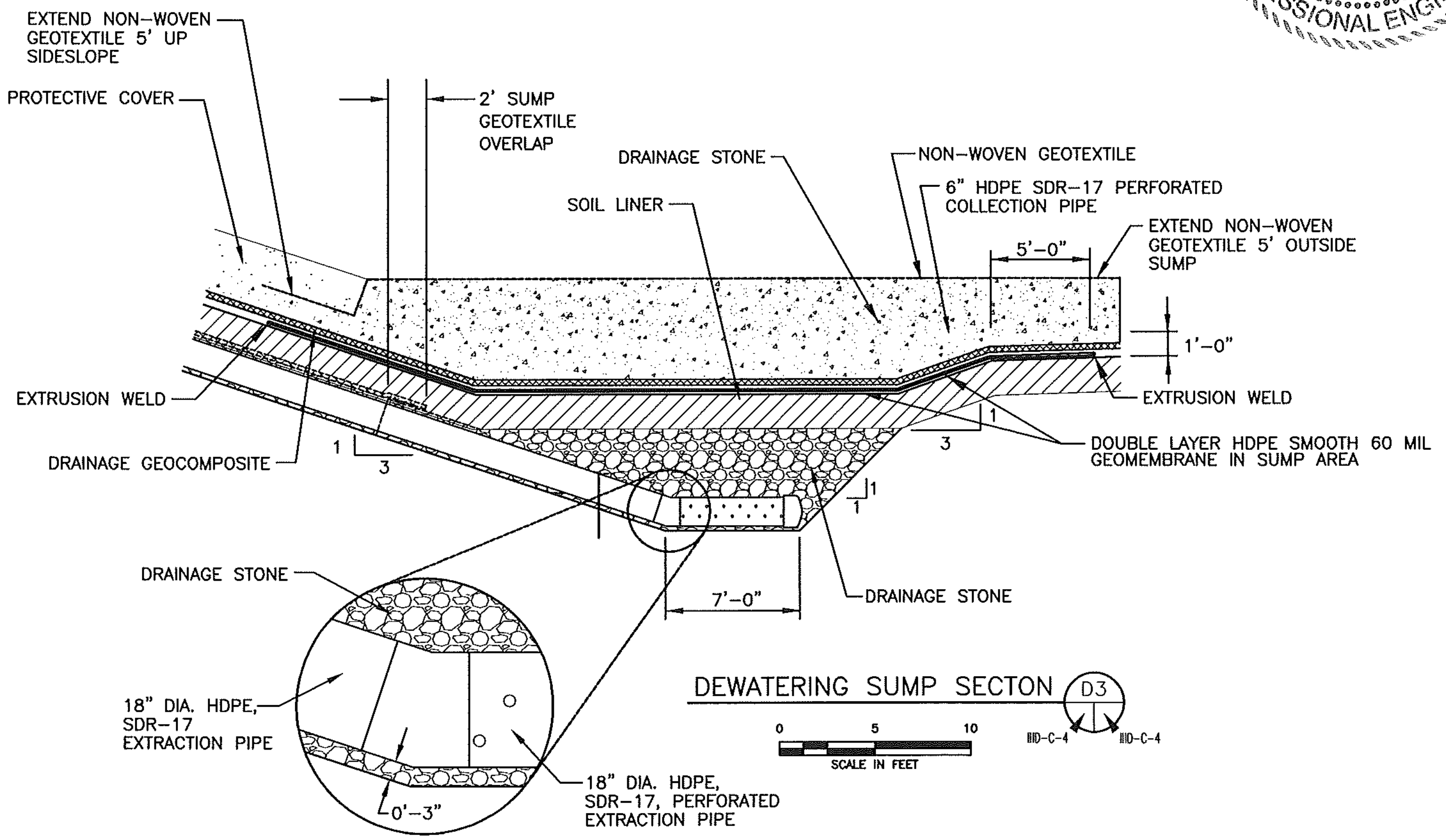
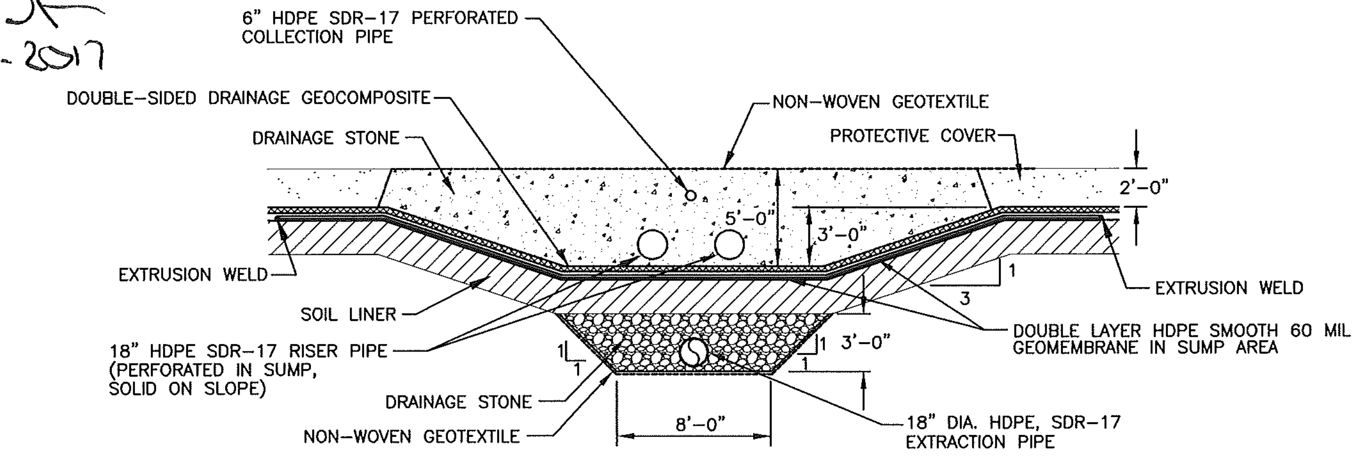
LICENSED
PROFESSIONAL ENGINEER

DEP
12-5-2017



DEWATERING SUMP SECTON



DEWATERING SUMP SECTION

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DATE: 08/2017
FILE: 0120-758-11
CAD: IIID-C-4 DEWATERING DETAILS.DWG

DRAWN BY: RDM
DESIGN BY: AE
REVIEWED BY: NT

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1	08/2017	FIRST NOD RESPONSE
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MAJOR PERMIT AMENDMENT
DEWATERING DETAILS

HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS

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SHEET IIID-C-4

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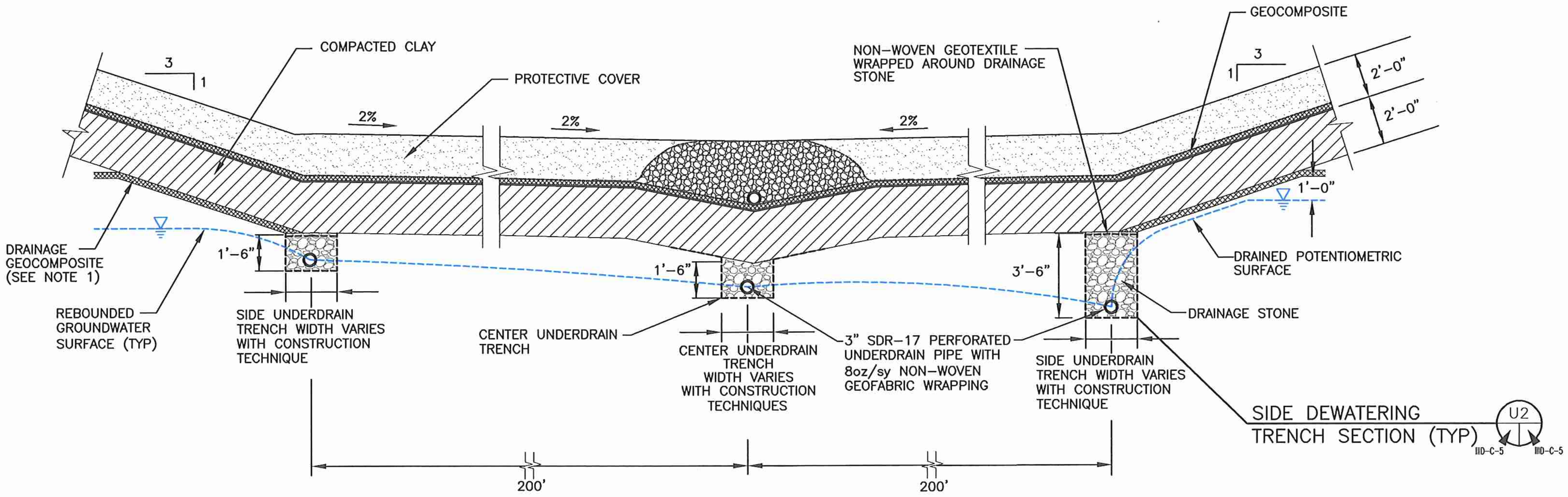
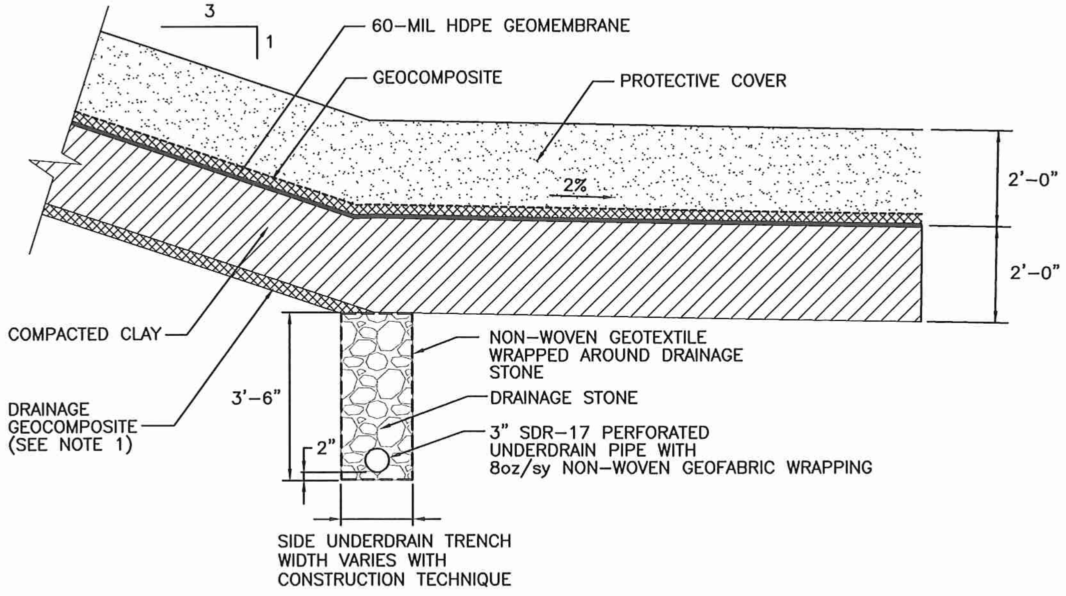


TYPICAL DEWATERING SECTION U1 IID–C–3 IID–C–5

0 3 6
SCALE IN FEET



SIDE DEWATERING TRENCH SECTION (TYP) U2 IID–C–3,5 IID–C–5

0 2 4
SCALE IN FEET

NOTE:

1. DEWATERING GEOCOMPOSITE WILL BE A MINIMUM 200–MIL THICK GEONET WITH 6oz/sy (MIN) GEOTEXTILES HEATBONDED TO BOTH SIDES.

STATE OF TEXAS
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LICENSED PROFESSIONAL ENGINEER
12-5-2017

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DATE: 03/2017
FILE: 0120–758–11
CAD: IID–C–5 DEWATERING DETAILS.DWG

DRAWN BY: RDM
DESIGN BY: AE
REVIEWED BY: NT

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MAJOR PERMIT AMENDMENT
DEWATERING SYSTEM DETAILS

HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS

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SHEET IIID–C–5

O:\0120\758\2214B EXPANSION\IIID\IIID–C–5 DEWATERING–DTLS.dwg, 11/15/2017 8:17:52 AM, rsellers, 1:2

1. Estimate the seepage into the dewatering geocomposite.

$q = kiA$

where:

q = inflow rate (cfs/ft)

k = hydraulic conductivity (cm/s) [Available slug tests of piezometers installed into the upper sand stratum indicate hydraulic conductivity values ranging from 7.52×10^{-6} (1991 SWL tests) to 3.03×10^{-4} cm/sec (2017 WCG test). A conservative value of 1×10^{-4} cm/sec was selected for the dewatering system design. The dewatering system design also conservatively assumed constant recharge of the entire upper sand stratum. Additionally, as shown below, conservative reduction factors were used in the analysis. The dewatering design was analyzed assuming groundwater entering geocomposite from 4-foot thick upper sand zone only.]

i = gradient (ft/ft)

A = inflow area (sf)

k =	1.0E-04	cm/s =	3.3E-06	ft/s
i =	0.33	ft/ft	(for 3H:1V slope)	
L=	12	ft (assumed contact length between geocomposite and upper sand)		
A =	12	sf (for unit width)		

q = 1.30E-05 cfs for unit width of geocomposite

2. Determine the required transmissivity of the dewatering geocomposite.

T_{req} = q calculated in Step 1 (Ref. 3)

T_{req} = 1.30E-05 cfs/ft

3. Determine the required ultimate transmissivity of the dewatering geocomposite.

The following reduction factors will be applied to determine the ultimate transmissivity.

Table 1 - Reduction Factors[1]

RF_{SCB} = Reduction factor for soil clogging and blinding	2.0
RF_{CR} = Reduction factor for creep reduction of void space	2.0
RF_{IN} = Reduction factor for adjacent materials intruding into void spaces	1.2
RF_{CC} = Reduction factor for chemical clogging	1.2
RF_{BC} = Reduction factor for biological clogging	1.1
Overall Reduction Factor (ORF) =	6.3

[1] Reduction factors obtained from Ref. 11.

$T_{ult} = T_{req} * ORF$

where:

T_{ult} = ultimate transmissivity (cfs/ft)
T_{req} = required geocomposite transmissivity calculated in Step 2 (cfs/ft)
ORF = overall reduction factor from Table 1

$T_{ult} = 1.03x10^{-5}$ cfs/ft * 6.3

$\boxed{T_{ult} = 8.23E-05 \text{ cfs/ft}}$

4. Determine the required permeability of the dewatering geocomposite.

$k_{lab} = T_{ult} / t$

where:

k_{lab} = required laboratory permeability (cm/s)
T_{ult} = ultimate transmissivity from Step 3 (cfs/ft)
t = thickness of geocomposite (mil)

T_{ult} = 8.23E-05 cfs/ft
t = 200 mil

k_{lab} = 4.94E-04 cfs/ft^2
k_{lab} = 0.02 cm/s

The minimum required permeability will be 0.02 cm/s per ASTM D 4491.

5. Design the dewatering geotextile over the toe drain.

The design calculations assume that the granular drainage material will consist of ASTM No. 467 aggregate with a hydraulic gradient greater than 1.0 cm/s and more than 90 percent gravel.

Retention:

Based on Chart 1 - "Soil Retention Criteria," given on page IIID-C-12, the apparent opening size (O_{95}) is determined as follows.

$d'_0 =$ 0.1 in
$d'_{50} =$ 0.5 in
$d'_{100} =$ 1.5 in

$C'_u =$ $(d'_{100}/d'_0)^{1/2} =$ 3.87 > 3

C'_u is greater than 3; therefore the granular drainage material is widely graded and it is also assumed that the relative density of the soil is less than 35%.

$O_{95} <$ $(9/C'_u)*d'_{50}$ = 1.16 mm

Permeability:

The required permeability is listed in Step 4.

Survivability:

Based on Table 2, "Survivability Strength Requirements," provided on page IIID-C-13, geotextile properties should be selected considering high contact stresses (i.e., heavy confining stresses).

Summary of required properties for geotextile around the dewatering trench pipe:

Apparent opening size	<	1.16	mm
Grab tensile strength	>	157	lbs
Elongation	>=	50	%
Puncture strength	>	56	lbs
Trapezoid tear	>	56	lbs
Permeability	>=	0.02	cm/s

6. Estimate the flow into the dewatering pipe.

The following calculation uses the largest geocomposite area upgradient of a dewatering pipe.

The area (A) was calculated by estimating the area of 1/2 the sideslope length of Cells 7 and 8 combined (3,250 feet combined), multipled by 12 feet of exposed upper sand layer. Flow into the drainage geocomposite at the geocomposite/clay soil interface was disregarded for this calculation.

$Q = kiA$

where:

Q = inflow rate (cfs)
k = hydraulic conductivity (cm/s)
i = gradient (ft/ft)
A = inflow area (sf)

k =	1.0E-04	cm/s
	3.3E-06	ft/s
i =	0.33	ft/ft
A =	19,500	sf

$\boxed{Q_{max} = 0.021 \text{ cfs}}$

7. Determine the flow capacity of the dewatering pipe.

$$Q_{full} = \frac{1.486}{n} AR^{2/3} S^{1/2}$$

Where: A = Cross-sectional area of pipe, with d representing the inside diameter in feet
R = Hydraulic radius of pipe in feet under full flow conditions

~~Using a 2-inch SDR 17 pipe:~~ Using a 3-inch SDR 17 pipe:

ID = 3.020 in
= 0.252 ft

$A = \frac{(\pi \times d^2)}{4}$ A = 0.050 sq ft

R = d / 4 R = 0.063 ft

The 3-inch diameter dewatering pipe will be placed at the locations shown on Sheet IIID-C-3. The slope is calculated as follows.

Slope Calculation:

Excavation elevation at downgradient end of dewatering pipe = 48.5 ft-msl
Contour at the upgradient end of dewatering pipe = 57.6 ft-msl
Total length of dewatering pipe (between sump and upgradient of pipe on one side) = 1625 ft

$$\text{Slope (S)} = \frac{57.6 - 48.5}{1625} = 0.56\%$$

S = Design slope of pipe S = 0.0056 ft / ft
n = Manning's number n = 0.009 from Ref. 6

Q_{full} = 0.097 cfs > Q_{max} = 0.021 cfs

The capacity of the 3-inch-diameter pipe is larger than the maximum calculated flow into the dewatering pipe. Therefore, the design is acceptable.

8. Determine required pipe perforation based on characteristics of the surrounding drainage media.

Pipe perforations must allow free passage of groundwater and also prevent migration of drainage media into dewatering pipes. Therefore, size of perforations depends on media particle size.

For dewatering pipes with circular holes:

$$\frac{D_{85}\text{ of Filter}}{\text{Hole Diameter}} > 1.7$$

Where: D_{85} = Particle size for which 85% of all particles are smaller than

Assume: Drainage media is a minimum ASTM D number 57 aggregate
(Size 467 can also be used)

D_{85} = 19 mm
= 0.748 in

Hole diameter, d = 0.438 in

Check values to find that:

$$\frac{D_{85}\text{ of Filter}}{\text{Hole Diameter}} = 1.71 \quad > 1.7 \quad \text{(acceptable)}$$

9. Determine the sump size and pump capacity.

The pump size is determined by converting the flow into the dewatering sump into gallons per minute. Conservatively multiply the Qmax calculated above by 2 to represent drainage from both sides of the combined cells 7 and 8.

Q_{max}= 0.042 cfs = 19 gpm

Therefore, a 20 gpm pump will be sufficient for groundwater removal at the site. A different pump size may be used if justified by the POR, and will be included in the SLER. Note that the pump calculations are conservative as they do not account for drawdown and depletion of groundwater in the 4-foot-thick upper sand layer. Actual pump volumes may be significantly less than calculated.

Sump Sizing:

Assume: Pump cycle time 10 minutes or greater.
Sump will be constructed of horizontal 18-inch diameter HDPE pipe with 12-inch deep pump sump.

Calculation:

Pumping rate =	20	gpm
Recharge rate =	19	gpm
Assumed period between pump starts =	190	minutes
GW volume to sump between starts =	3600.6	gallons
Time to pump sump down =	180.0	minutes
Time between pump on cycles =	10.0	minutes

Assumed period between pump start in above adjusted until desired time between pump cycles obtained.

GW volume between pump stop and start	189.0	gallons
Pipe sump diameter =	18	inches
Pipe sump length =	4.6	feet

Use 7-foot length of 18-inch diameter HDPE pipe for the horizontal groundwater dewatering sump.

APPENDIX IIID-D

WASTE-AS-BALLAST PLACEMENT RECORD

Includes pages IIID-D-1 through IIID-D-2

STATE OF TEXAS
DAVID E. POE
81734
LICENSED
PROFESSIONAL ENGINEER

12-5-2017

WASTE-AS-BALLAST PLACEMENT RECORD

This form is to be completed by the site manager or designated representative for all landfill areas utilizing waste as ballast. One form will be developed for each area (or combination of areas) described by approved liner evaluation reports. This form is to be submitted with the Ballast Evaluation Report (BER) for the evaluated area and may be referenced by the Professional of Record (POR) in order to verify that the placement of ballast is in compliance with the Liner Quality Control Plan (LQCP). The site operator must prepare and sign supporting documentation on a daily basis verifying the area of waste placement, the waste material in the first 5 feet of waste was free of large bulky items, daily operation of the pressure relief/dewatering system, and a wheeled trash compactor having a minimum weight of 40,000 pounds was used.

A. GENERAL INFORMATION

Area documented by this record (provide site grid coordinates of each corner) ____________

____

Soils and Liner Evaluation Report document date(s) and approval date(s) for this area ______

Date of initial waste placement ____________

Date of completion of first 5 feet of waste in place over entire area ________________

Total required waste-as-ballast thickness for this area (Note: Calculations for determining the required thickness of waste as ballast are included with the LQCP/BER for this area.) ________________

Date when minimum required thickness of waste was achieved ________________

B. WASTE EQUIPMENT USED

What type of compaction equipment was used? ____________________________

Did the compactor have a minimum gross weight of 40,000 pounds? ________________

Was this compactor used throughout the entire period covered by this record? ___________

If a minimum 40,000-pound wheeled trash compactor was not used throughout the period covered by this record, attach documentation of initial and final survey data (if not previously provided as part of the BER) of the ballasted area and measurements of truck weights at the scalehouse for the time period covered by the BER for use in determining in-place waste density. Is this documentation complete and accurate? ____________________

C. FIRST WASTE LIFT CONSIDERATIONS

Describe type(s) of waste placed in first 5 feet of waste over the top of the liner protective cover ______________________________

Does the first 5 feet of waste contain any large bulky waste items which would damage the underlying liner system or which cannot be compacted to the required density? ______________

D. WASTE COMPACTION METHODS

Approximate loose waste layer thickness prior to compaction ______________

Minimum number of compactor passes for each waste layer ______________

Maximum slope of compacted waste layers ______________

E. PRESSURE RELIEF/DEWATERING SYSTEM

Was the pressure relief/dewatering system (if required) operated continuously during the period covered by this record? __________ Is the pressure relief/dewatering system presently in operation? ______________________________

SIGNATURE OF PERMITTEE OR OPERATOR

The waste overlying the area described in this record has been placed and compacted as described in this record and in accordance with the Liner quality control plan and Site Operating Plan.

______________________________	Itasca Landfill
(Signature)	(Business Name or Facility)
______________________________	______________________________
(Typed or Printed Name)	
______________________________	______________________________
(Title)	(Address, City, Zip Code)
______________________________	______________________________
(Date Signed)	(Phone No.)

Note: This completed form must be submitted with the BER and placed in the Operating Record and available for review.

HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS
TCEQ PERMIT NO. MSW 2214B

PART III – SITE DEVELOPMENT PLAN
APPENDIX IIIE
GEOTECHNICAL REPORT

Prepared for

BFI Waste Systems of North America, LLC

March 2017
Revised August 2017

Revised December 2017

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12-5-2017

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12-5-17

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WCG Project No. 0120-758-11-02

This document is intended for permitting purposes only.

CONTENTS

STATE OF TEXAS
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12-5-17

CONTENTS (Continued)

STATE OF TEXAS
DAVID E. POE
81734
LICENSED
PROFESSIONAL ENGINEER

12-5-17

TABLES

DAVID E. POE
81734

12-5-17

1 INTRODUCTION

This appendix addresses §330.63(e)(5)(A) and (B).

The purpose of this report is to present the geotechnical analysis and design for the proposed major permit amendment for the vertical expansion of the Hardin County Landfill. This report is based on the geotechnical testing information that has been compiled from the subsurface investigations at the site.

The Hardin County Landfill is an existing 79-acre Municipal Solid Waste (MSW) Disposal Facility (TCEQ Permit No. MSW-2214B) owned and operated by BFI Waste Systems of North America, LLC, a subsidiary of Republic Services, Inc. (RSI). The facility includes a 49.6-acre Type I disposal area and a 2.4-acre Type IV disposal area. The expansion will consist of reconfiguring the approximately 16.6 acres of unconstructed Type I MSW disposal area, and increasing the peak elevation of the entire Type I MSW landfill from the currently permitted 115 feet msl to 234 feet msl. No changes to 2.4-acre Type IV disposal area are proposed for this amendment.

The following geotechnical report provides the analysis for the expansion of the Type I MSW disposal unit.

This geotechnical report also provides geotechnical recommendations for construction of the remaining unconstructed landfill components, including bottom liner and final cover systems. The construction quality control as well as material and construction specifications for the groundwater protection components of the landfill are provided in Appendix IIID – Liner Quality Control Plan.

2 SUBSURFACE CHARACTERIZATION AND GEOTECHNICAL TESTING

2.1 Subsurface Characterization

Subsurface characterization of the existing 79-acre permit boundary area is supported by 45 soil borings. These include 17 borings by Southwestern Laboratories, Inc. (SWL) (1990 and 1991), 21 borings by Hydrex Environmental Inc. (Hydrex) (1998, 2005, and 2010), and 6 borings completed by WCG (January 2017). The site-specific lithology has previously been characterized by SWL in 1994 (Permit No. MSW 2214) and by Hydrex in 2001 (Permit No. MSW 2214A). The locations of the 45 soil borings are shown in Appendix IIIG, Figure IIIG-B.1. Site-specific geologic cross sections have been constructed and are included in Appendix IIIG – Geology Report, Appendix IIIG-C. The borehole descriptions are summarized as follows:

- A 1990 subsurface characterization by SWL included at least nine geotechnical borings that were advanced to evaluate subsurface conditions for the proposed landfill facility. Attachment 4 of the facility's permitted Site Development Plan (SDP) states that 9 borings within the 79-acre permit boundary (B-1, 2, 3, 4, 5, 6, 8, 9, and 12) and an additional unspecified number of off-site borings were drilled in May 1990. With the exception of boring B-9, no lithologic information is available for these initial investigatory boreholes and they have not been included in the summation of existing facility exploration borings in the original permit application. Per the currently approved Attachment 4, the Texas Department of Health (TDH) required SWL to re-drill the 9 borings referenced above to greater depths and advance additional borings to characterize the site. In accordance with this TDH request, SWL redrilled several borings (within 5 feet of the former borehole locations) and drilled additional borings from December 1990 to January 1991.
- A 1990-1991 subsurface characterization continuation by SWL included an additional 16 geotechnical borings (P-1A, P-2A, P-3A, P-4A, B-5A, B-6A, B-8A, B-12A, and P-20 through P-27) that were drilled to further evaluate subsurface conditions for the proposed landfill facility. Twelve of these borings were completed as piezometers. All of the boreholes fully penetrated the Upper Sand Stratum uppermost aquifer, most of the borings encountered the underlying low permeability Lower Clay Stratum, and five of the borings penetrated a portion of the Lower Sand Stratum.

- A 1998 subsurface characterization by Hydrex Environmental included 20 geotechnical borings for the installation of 6 groundwater monitor wells (MW-1 through MW-6) and 7 LFG monitoring probes (GMP-1 through GMP-7). Most of these boreholes penetrated the Upper Sand Stratum uppermost aquifer and in most cases encountered the top of the underlying low permeability Lower Clay Stratum.
- A 2005 subsurface characterization by Hydrex Environmental included six geotechnical borings for the installation of 6 groundwater monitor wells (MW-7 through MW-12). These boreholes penetrated the Upper Sand Stratum uppermost aquifer.
- A 2010 subsurface characterization by Hydrex Environmental included three geotechnical borings for the installation of 3 groundwater monitor wells (MW-5R, MW-6R, and MW-13). These boreholes penetrated the Upper Sand Stratum uppermost aquifer and encountered the top of the underlying low permeability Lower Clay Stratum.
- A January 2017 subsurface characterization by Weaver Consultants Group, LLC included site exploration by advancing six additional boreholes within the proposed 16.6-acre southern vertical expansion area at the locations shown in Appendix IIIG – Geology Report, Figure IIIG-B.1. Geotechnical borings WC-1 through WC-5 were continuously sampled using hollow stem auger, Shelby tubes, or split spoon techniques. All recovered subsurface material samples were retained for laboratory testing.

Geological/lithological logs were prepared for each of the borings described above, and are included in Appendix IIIG – Geology Report, Appendix IIIG-B.

2.2 Geotechnical Testing

As required by Title 30 Texas Administrative Code (TAC) §330.63(e)(5), geotechnical laboratory testing was performed of soil samples obtained from the site and tested in accordance with industry practice and recognized procedures as discussed below. The objective of the testing was to demonstrate the suitability of the soils and strata for the uses they are intended for in this application.

The geotechnical testing performed on soil samples obtained during the 1990-1991 piezometer drilling and the 2017 geotechnical investigations, as described in this section, are shown in tables included in Appendix IIIE-C of this report.

Table 2-1
Geotechnical Test Methods Performed

Test	Test Method
Sieve Analysis (Passing No. 200)	ASTM D1140
Atterberg Limits (Liquid & Plastic Limit)	ASTM D4318
Moisture Content	ASTM D2216
Hydraulic Conductivity	EM 1110-2-1906
Consolidated-Undrained Shear Strength	ASTM D6528
One-dimensional Consolidation	ASTM D4186

A total of 79 moisture content analyses, 73 Atterberg limits, and 48 sieve analyses (passing the #200 sieve) were performed on samples obtained from the 1990-1991 investigations. An additional 10 Atterberg limits, 11 sieve analyses (passing the #200 sieve), 8 dry density tests, 5 hydraulic conductivity tests, 2 consolidation tests, and one consolidated-undrained shear strength test were performed on samples obtained during the 2017 investigations. The results of the geotechnical testing are summarized in tables presented in Appendix IIIE-C of this report.

In addition to the above testing, hydraulic conductivity tests were performed on remolded clay samples obtained from the site for the 1995 permit application. The hydraulic conductivity testing was performed by Southwest Laboratories, and were performed to demonstrate the suitability of on-site excavations for use in liner construction. The results of the testing demonstrate that the on-site clay soils are suitable for use in bottom liner and final cover construction with a laboratory permeability results of less than 1x10^{-7} cm/s. In addition to the laboratory hydraulic conductivity tests, in-situ hydraulic conductivity (slug) testing was performed on the upper sand to determine the sand layer's hydraulic conductivity for developing the dewatering system design which is preened in Appendix IIID – Liner Quality Control Plan.

Additionally, 13 cells have been constructed at the landfill since 1995. The permit set criteria as shown in Table 2-3 for low permeability soils used for liner construction. Review of the landfill SLERs demonstrates that the on-site soils are suitable for liner construction, with laboratory permeability results less than 1x10^{-7} cm/s.

The shear strength properties analyzed during the 2017 geotechnical testing were used in Section IIIE-A of this report to select typical short and long-term strength parameters for the Upper Clay Stratum as required for stability and settlement analyses.

3 TEST CORRELATIONS AND SUMMARIES

3.1 General

This section of the report addresses the generalized stratigraphy for the site, potential uses of materials that may be excavated during construction, and typical properties of those materials. Results from laboratory testing, field testing, and observations were used for the material correlations. The results of the testing performed on site soils are presented on piezometer logs and boring logs provided in Appendix IIIG – Geology Report, Appendix IIIG-B, and as summarized in tables included in Appendix IIIE-C of this report.

3.2 Site-Specific Stratigraphy

Based on the boring logs that have been obtained from the site and the subsurface characterization provided in Appendix IIIG – Geology Report, the stratigraphic units are generalized into four stratum within the depths investigated at the site. The following terminology is consistent with terminology used in the Appendix IIIG – Geology Report.

- Upper Clay Stratum
- Upper Sand Stratum (Uppermost Aquifer)
- Lower Clay Stratum (Aquiclude/tard)
- Lower Sand Stratum
- Basal Clay Stratum

Each of the stratum is briefly described below. Additional discussion is provided in Appendix IIIG – Geology Report.

3.2.1 Upper Clay Stratum

The uppermost site-specific stratigraphic unit is the Upper Clay Stratum. This unit is continuous beneath the landfill and characterized as predominantly dry to moist, low permeability clay and silty clay with minor amounts of sand. This uppermost unit also includes a minor subunit of discontinuous surficial sand and silt. The Upper Clay Stratum ranges in thickness from 8 to 39 feet site-wide. Discontinuous thin silt and sand filled partings and seams are common in this strata. Slickensides are noted within the Upper Clay Stratum at depths ranging from 12 to 20 feet.

3.2.2 Upper Sand Stratum

Beneath the Upper Clay Stratum lies the Upper Sand Stratum. The Upper Sand Stratum constitutes the landfill's permitted groundwater monitoring zone (uppermost aquifer). This stratum is continuous beneath the landfill. The Upper Sand Stratum is characterized as a saturated sandy silt, silty sand, clayey sand, or sandy clay with a range in thickness from 1 foot to 30 feet site-wide. The Upper Sand sediments are generally much finer in the western borings often occurring as a sandy clay or as zones of thinly interbedded seams of wet sands and silts bound by moist silty clay. The Upper Sand Stratum sediments become coarser toward the east with more abrupt and well defined transitions between the Upper Sand and bounding Upper Clay and Lower Clay strata.

3.2.3 Lower Clay Stratum

Beneath the Upper Sand Stratum lies the Lower Clay Stratum. The Lower Clay Stratum is the permitted lower confining unit (aquiclude) to the Upper Sand Stratum (uppermost aquifer). This subsurface characterization verifies that this stratum is the lower confining unit for the Upper Sand. The Lower Clay Stratum is characterized as predominantly dry to moist silty clay with minor amounts of sand present in matrix and interbedded horizontally in thin partings and laminations. The Lower Clay Stratum is continuous beneath the site.

3.2.4 Lower Sand Stratum

The Lower Sand Stratum lies beneath the Lower Clay Stratum aquiclude. Three of the SWL borings (P-23, P-24, and P-25) encountered the top of the Lower Sand Stratum. All three of these borings are located along the eastern landfill permit boundary. All six of the deep borings completed by WCG in 2017 (WC-1, WC-2, WC-3, WC-4, WC-5, and WCP-5) penetrated the Lower Sand Stratum. Lithologic data from these borings indicate an abrupt increase in Lower Sand Stratum thickness and sediment coarseness within the eastern quarter of the southern undeveloped waste footprint in the cross section figures included in Appendix IIIG – Geology Report, Appendix IIIG-C. The Lower Sand Stratum is characterized as a thick sequence of loose to unconsolidated saturated silty sand with a thickness of 44 to 46 feet on the east end of the undeveloped waste footprint.

3.2.5 Basal Clay Stratum

Beneath the Lower Sand Stratum lies the Basal Clay Stratum. The Basal Clay Stratum is characterized as a moist silty clay of low permeability. Five of the deep geotechnical borings completed by WCG in 2017 (WC-1, WC-2, WC-4, WC-5, and

WCP-5) encountered the Basal Clay Stratum. The Basal Clay Stratum is the lower confining unit the saturated Lower Sand Stratum.

3.3 Material Requirements

Construction of the landfill will require clay or clayey soils which can be compacted to have an in-place hydraulic conductivity of 1×10^{-7} cm/s or less for the soil liner portion of the composite liner and Type IV C&D liner, and an in-place hydraulic conductivity of 1×10^{-5} cm/s for the soil infiltration layer of the final cover system for both the Type I Subtitle D area and Type IV C&D area.

Soil will also be required for protective cover on the liner, operational cover (daily and intermediate), the erosion layer component of the composite final cover, berm construction, and other miscellaneous general fill. Granular material (i.e., gravel) will be used for the leachate collection sumps, leachate collection chimneys and may be used for groundwater dewatering collection trenches. Typical material requirements for various soil structures are summarized in Table 3-2.

Testing requirements and construction quality control and quality assurance for liner soils are detailed in Appendix IIID – Liner Quality Control Plan (LQCP). Testing requirements and construction quality control and quality assurance for final cover soils are detailed in Appendix IIIJ-A – Standard Subtitle D Final Cover System Quality Control Plan (FCSQCP). Liner and final cover details are presented in Appendix IIIA-A – Liner and Final Cover System Details.

3.4 In-Situ Materials at Liner Subgrade

Prior to the installation of liner components, exposed in-situ materials will not include loose or soft soils. The presence of cracks, fissures, and fractures will not be allowed in the exposed surface of the base grade. In-situ base grade soils will be firm and will not exhibit significant rutting from the construction traffic.

3.5 Properties of Soil Liner

Material used to construct the soil liner is required to have a minimum liquid limit of 30 and plasticity index of 15. The hydraulic conductivity for the soil liner is required to be 1×10^{-7} cm/s or less (refer to Table 3-2). The soil liner material and construction requirements are discussed in detail in Appendix IIID – LQCP.

3.6 Properties of Liner Protective Cover Material

A two-foot thick protective soil cover will be placed over the leachate collection layer. The protective cover is required to protect the liner and leachate collection system (LCS) from erosion and construction activity and will consist of on-site soils. The LCS drainage media will be extended through the protective cover along the entire length of the leachate collection trenches and over the sumps. This extension of the drainage media through the protective cover ("chimney drain") will allow transmission of leachate to the LCS. Details of the chimney drain are provided on Drawing IIIA-A.5 in Appendix IIIA-A – Landfill Unit Design Information.

3.7 Properties of Composite Final Cover Soils

3.7.1 Properties of Infiltration Layer Soil

Type I MSW Final Cover

Material used to construct the composite final cover soil infiltration layer component of the Type I MSW final cover is required to have a minimum liquid limit of 30 and a plasticity index of 15. The purpose of this layer is to reduce infiltration of surface water into the fill. As defined in Appendix IIIJ – Closure Plan, the infiltration layer for Type I MSW area will consist of 18 inches of earthen material with a coefficient of permeability no greater than $1x10^{-5}$ cm/s overlain by a synthetic membrane. Alternatively, a geosynthetic clay liner (GCL) may be substituted for the 18-inch-thick earthen material layer described above.

Type IV Final Cover

The infiltration layer for Type IV C&D area will consist of 18 inches of earthen material with a coefficient of permeability no greater than $1x10^{-5}$ cm/s overlain by 12 inches topsoil. Alternatively, a GCL may be substituted for the 18-inch-thick earthen material layer described above, overlain by 12 inches of topsoil/protective cover.

3.7.2 Properties of Erosion Layer

As shown in Appendix IIIA – Landfill Unit Design Information (Appendix IIIA-A), the Type I final cover system will include a 24-inch-thick erosion layer, and the Type IV final cover system will include a 12-inch-thick erosion layer. This layer may be spread and placed as two lifts (for the 24-inch-thick layer) or a single lift (for the 12-inch-thick layer) over the entire final cover area. After spreading, the layer will be rolled lightly to reduce future erosion but not to the extent that compaction would inhibit plant growth. The top 6 inches of the erosion layer will be capable of sustaining vegetative growth. The completed erosion layer will be seeded with local and/or introduced grasses and maintained to establish vegetation.

3.8 Properties of Operational Cover Material

Operational cover includes daily cover and intermediate cover. The materials excavated from the site may be used for operational cover. As listed in Table 3-2, operational cover soil is not restricted by physical property. However, the intermediate cover layer must be capable of sustaining vegetation growth. Any of the soil materials encountered in the excavation may be used for operational cover provided that they meet the maximum particle size criterion and were not previously mixed with waste materials.

3.9 Properties of Earth Fill Material

Earth fill material may be required for subgrade preparation, embankments, haul roads, and other miscellaneous fill. Material availability, compatibility, and long-term maintenance requirements should be considered when evaluating the excavated soils for use as earth fill. General fill material placed below the composite liner (e.g., over-excavated areas within the liner construction area) will be placed in uniform lifts which do not exceed 8 inches in loose thickness, and general fill material for structural fill (e.g., perimeter berm construction and liner anchor trench backfill) will be placed in uniform lifts which do not exceed 12 inches in loose thickness. The fill will be compacted to at least 95 percent of standard Proctor maximum dry density (ASTM D 698) at a moisture content at or above the optimum moisture content when it is used for backfill below the soil liner.

Table 3-1
Typical Properties of On-Site Materials

Laboratory Test[1]		Typical Values[2,3]			Number of Tests		
		Upper Clay Stratum	Upper Sand Stratum	Lower Clay Stratum	Upper Clay Stratum	Upper Sand Stratum	Lower Clay Stratum
Classification	**Test Method**						
Liquid Limit (LL)	ASTM D 4318	46.4	22.6	50.2	65	7	9
Plastic Limit (PL)	ASTM D 4318	16.9	17.7	21.2	65	7	9
Plasticity Index (PI)	ASTM D 4318	29.5	4.9	29	65	7	9
% Passing 200 Sieve	ASTM D 1140	80.3	53.2	88.3	41	9	6
Natural Moisture Content (%)	ASTM D 2216	20.7	21.8	26.7	69	9	9
Dry Unit Weight (pcf)		93.5	--	95.2	3	0	4
Shear Strength	**Test Method**						
Total Triaxial Shear Strength (c, psf/phi, degrees)	ASTM D 6528	718/11.4	NA[5]	NA[5]	1	0	0
Effective Triaxial Shear Strength (c, psf/phi, degrees)		644/12.6	NA[5]	NA[5]	1	0	6
Hydraulic Conductivity (cm/s) (Vertical)[4]	EM-1110-2-1906 Appendix VII/ASTM D 5084	1.3x10^{-7}	1.1x10^{-8}	2.8x10^{-8}	2	2	1

[1] Laboratory test results were obtained from site subsurface investigations. Laboratory test data from site subsurface investigations are presented in Appendix IIIE-C.

[2] Values listed are averages of the data for each soil layer or material, unless otherwise noted.

[3] Refer to Appendix IIIG – Geology Report for information regarding geologic units that exist at the site.

[4] Only results for 2017 testing are shown. Values from 1995 permit application or SLERS is not provided.

[5] Strength testing was not performed for these strata. The geotechnical analyses assumes failure planes and settlement confined to Upper Clay Stratum.

Table 3-2
Typical Soil Requirements for Landfill Construction

Landfill Component	Soil Description	Classification	Test Parameters				Material Source[3]
			LL	PI	%-200	Hydraulic Conductivity (cm/s)	
Soil Liner	clay, clayey sand, or sandy clay	CL, CH, SC	30 min	15 min	30 min	1x10^{-7} max	On site
Final Cover Infiltration Layer	clay, clayey sand, or sandy clay	CL, CH, SC	30 min	15 min	30 min	1x10^{-5} max[1]	On site
Protective Cover	soil free of deleterious material	Not applicable					On site[2]
Erosion Layer	clay, silty clay, sandy clay, or clayey sand	CL, CH, ML, SC	Suitable to support plant growth				On site
Operational cover (daily cover, intermediate cover)	soil not previously mixed with solid waste	Not applicable	--	--	--	--	On site
Earth Fill: Perimeter Berm & Subgrade Preparation	clay, clayey sand, or sandy clay	CL, CH, SC	--	--	--	--	On site

[1] Hydraulic conductivity of the composite final cover infiltration layer will be 1x10^{-5} cm/s or less. GCL may be substituted for the final cover infiltration layer.

[2] Leachate collection chimney drains will be extended through the protective cover at select locations and will be exposed adequately for transmission of leachate to the collection system.

[3] If on site materials meeting the required properties do not exist, off site material source can be used.

4 CONSTRUCTION CONSIDERATIONS

4.1 General

This section contains recommendations for excavation of the landfill, control of seepage during excavation, soil liner construction, leachate collection layer materials, operational cover soils, final cover construction, and perimeter embankment construction.

As mentioned in Section 1, the landfill consists of two separate units: a 49.6-acre Type I MSW disposal area, and a 2.4-acre Type IV disposal area. The footprint of both disposal units will remain unchanged for this expansion. The Type I MSW disposal area will expand by lowering the bottom liner system to an elevation above the previously established Elevation of Deepest Excavation (EDE) and vertically expanding the entire landfill. No changes to the Type IV C&D area are proposed for this expansion application.

Excavations for the remaining 16.6 acres of Type I disposal area (Cells 7 and 8) are founded in either the previously described Upper Clay Stratum, underlying Upper Sand Stratum or Lower Clay Stratum. The currently developed Subtitle D liners of the Type I area constructed after permit issuance in 1995 (Cells 1 through 6) include groundwater dewatering systems for temporary groundwater hydrostatic uplift pressure relief. The liner design for the undeveloped portion of the Type I MSW disposal area includes a temporary groundwater sideslope dewatering system.

4.2 Landfill Excavation

The excavation for the liner construction will be performed in a manner that will achieve reasonable segregation of liner quality material (Upper Clay Stratum) from the topsoil or underlying Upper Sand Stratum. Soil materials to be used for liner construction will be stockpiled separately, according to construction material properties outlined in Section 3.3 and visual observation during excavation.

Excavation of the soils encountered will be achieved with an excavator, scraper, or similar equipment. Local areas of the Upper Sand Stratum which underlies the Upper Clay Stratum likely will be encountered and penetrated intermittently within the excavation and/or as the depth of excavation increases. Blasting of hard rock will not be required and will not be used at this site.

Excavation side slopes will be graded no steeper than 3 horizontal to 1 vertical. Excavation cut slopes around the perimeter may require erosion protection if an extended period of time occurs between excavation and liner construction. Interim erosion protection can be accomplished by diverting runoff away from the slopes. "Track walking" with a bulldozer up and down the slopes will create the effect of "mini-dikes" with the bulldozer tracks, which will reduce erosion.

Prior to beginning construction of the liner components, the subgrade area will be stripped to a depth sufficient to remove all loose surface soils or soft zones within the exposed excavation. The liner base grades will be proof-rolled with heavy, rubber-tired construction equipment or equivalent to detect soft areas. Soft areas will be undercut to firm material and backfilled with suitable compacted earth fill, as discussed in Section 3.9. Preparation of the liner base grades will result in a surface that is stable and that does not exhibit significant rutting from the construction traffic. The prepared liner base grades will be approved by a Professional of Record, tested to verify that it meets material requirements outlined in Section 3.4, and surveyed to verify grades.

4.3 Control of Water Seepage During Excavation Applicability

The excavation within the undeveloped area of the landfill will penetrate the Upper Sand Stratum, a groundwater bearing formation located at the site. As discussed in Appendix IIIG – Geology Report, the groundwater of the uppermost aquifer flows from east to west or northwest beneath the site. Section 5 of this report includes the design for construction below the groundwater table.

4.4 Soil Liner Construction

Both landfill floor grades and excavation side slopes will be lined with a 2-foot-thick compacted clay liner for the Type I MSW disposal area and a 3-foot-thick compacted clay liner for the Type IV C&D disposal area.

The compacted clay liners will have a maximum hydraulic conductivity of $1x10^{-7}$ cm/s. Details for the liner system are provided in Appendix IIIA (Appendix IIIA-A). Adequate soil liner material will be available from proposed landfill excavations, onsite, or offsite borrow sources to provide material for the liner construction. Laboratory tests performed during liner construction will verify that this material is adequate to meet the compacted clay liner requirements listed in 30 TAC §330.339(c)(5).

The soils used for liner construction will have the minimum soil properties listed in Table 4-1, which will be verified by preconstruction testing in a soils laboratory. The following soil liner properties are included in Appendix IIID – LQCP.

Table 4-1
Soil Liner Properties

Test	Specifications
Hydraulic Conductivity of Remolded Soils[1]	1.0x10^{-7} cm/s or less
Plasticity Index, percent	15 minimum
Liquid Limit, percent	30 minimum
Passing No. 200 Sieve, percent	30 minimum
Passing 1-inch Sieve, percent	100

[1] A hydraulic conductivity test will be performed on soil samples remolded per ASTM D 698 in accordance with Appendix IIID – LQCP.

Representative preliminary sampling will be performed on the materials that will be used for soil liner construction. Prior to construction of each new lined cell, conformance tests that include liquid limit, plastic limit, percent passing the No. 200 sieve, standard Proctor (ASTM D 698) and remolded hydraulic conductivity tests will be performed for the liner soils. Additional conformance tests will be conducted if there are visual changes in the borrow material or the liquid limit or plasticity index vary by more than 10 points. The soil liner construction and testing procedures are outlined in Appendix IIID – LQCP.

4.5 Drainage Materials

The LCS drainage material will consist of a drainage geocomposite over the entire membrane liner bottom and sideslopes. Each cell remaining to be constructed (Cells 7 and 8) will have a bottom slope toward an LCS trench (i.e., pipe enveloped in gravel and geotextile) that will collect leachate from the bottom and sideslopes. The leachate collection system details are illustrated in Appendix IIIA (Appendix IIIA-A). The material specifications and construction procedures for the LCS components are presented in Appendix IIID – LQCP. The LCS design and demonstrations are provided in Appendix IIIC – Leachate and Contaminated Water Management Plan.

4.6 Liner Protective Cover

The Type I MSW liner protective cover is required to be a minimum of 24 inches thick. The purpose of the protective cover is to protect the geosynthetics (i.e., geomembrane and drainage geocomposite) from solid waste placed over the liner system. To ensure passage of leachate into the leachate collection system, drainage passages (chimney drains) will be constructed through the protective cover. The chimney drains will be installed over the LCS collection pipes as shown in Appendix IIIA (Appendix IIIA-A). The protective cover will be placed with construction equipment in one lift such that it covers the leachate collection layer completely. The protective cover material will be free of solid waste and will not require compaction under the density-controlled construction procedures.

4.7 Operational Cover Soils

Operational cover soils include protective cover placed over the geocomposite drainage layer, daily cover (placed over the waste each day), and intermediate cover (placed over waste in areas that will not receive additional fill for at least 6 months). All soils excavated at the site may be used for operational cover.

4.8 Composite Final Cover Construction

4.8.1 Final Cover Infiltration Layer Construction

The infiltration layer of the final cover system will be constructed with clay and will be a minimum of 18 inches thick. The purpose of this layer is to reduce infiltration of surface water into the fill. As defined in Appendix IIIJ – Closure Plan, areas of the Type I MSW disposal area with a synthetic bottom liner will receive an infiltration layer consisting of 18 inches of earthen material with a coefficient of permeability no greater than $1x10^{-5}$ cm/s overlain by a synthetic membrane. A GCL may be substituted for the soil infiltration layer. The Type IV disposal area will receive an infiltration layer consisting of 18 inches of earthen material with a coefficient of permeability no greater than $1x10^{-5}$ cm/s.

4.8.1 Final Cover Erosion Layer Construction

As shown in Appendix IIIA-A, the composite final cover system for the Type I MSW disposal area will include a 24-inch-thick erosion layer. The final cover for the Type IV disposal area will include a 12-inch-thick erosion layer. The erosion layer will protect the infiltration layer and will support vegetative growth. The Type I area erosion layer may be spread and placed in two lifts (18 inches and 6 inches) over the entire cap area as the final cover is constructed. The Type IV area erosion layer will be placed in 2 lifts (6 inches soil and 6 inches topsoil) or a single lift (12 inches topsoil).

After spreading, each lift will be rolled lightly to reduce future erosion but not to the extent that compaction would inhibit plant growth. The top 6 inches of the erosion layer will consist of (1) topsoil stockpiled during the excavation process, (2) other on-site excavated soils amended as necessary to be capable of sustaining vegetation, and/or (3) imported soil materials. Whether placed in a single lift or two lifts, the erosion layer (top of final cover) will sustain vegetative growth.

4.9 Perimeter Embankment Construction

Perimeter embankments (berms) may be constructed to prevent surface water flow from entering the landfill excavation or to accommodate the liner and cover grades shown in Appendix IIIA-A. The embankment will have side slopes no steeper than 3 horizontal to 1 vertical (3H:1V). A sufficient amount of soil is available from the landfill excavations to construct the perimeter embankment and other features that require earth fill material.

Prior to beginning embankment fill, the subgrade area will be stripped to a depth sufficient to remove all topsoil and vegetation. Topsoil will be stockpiled for later use. The subgrade area will be proof-rolled with heavy, rubber-tired construction equipment to detect soft areas. Soft areas will be undercut to firm material and backfilled with suitable compacted earth fill. The subgrade preparation will result in a subgrade surface that is stable and does not exhibit significant rutting from construction equipment traffic.

The embankments will be constructed of onsite soils free of organic or other objectionable materials. The general fill placed below the composite liner (e.g., over excavated areas within the liner construction area) will be spread in maximum 8-inch-thick horizontal lifts and compacted to a minimum of 95 percent of standard Proctor density at a moisture content with the range of 95 percent compaction. A minimum of one standard Proctor test (ASTM D 698) will be performed on each representative soil used as earth fill material. Each lift will receive a minimum of four passes with a heavy tamping roller unless adequate compaction can be demonstrated with fewer passes. Moisture-density field testing and full-time monitoring during construction will be performed in accordance with Appendix IIID – LQCP. As necessary, the outside slope of all constructed embankments will be vegetated to minimize erosion and desiccation.

5 DESIGN FOR CONSTRUCTION BELOW THE GROUNDWATER TABLE

5.1 General

Portions of the excavation within the undeveloped area of the landfill (Cells 7 and 8) will penetrate the Upper Sand Stratum, which has the ability to impose uplift pressure from groundwater on the bottom liner system. Discussion of groundwater is provided in Section 3 – Groundwater Investigation Report of the Appendix IIIG – Geology Report. The groundwater in the Upper Sand Stratum is expected to exert uplift pressure on the liner system over portions of the sideslope grades that penetrate the Upper Sand Stratum. Therefore, the future cells are provided with a temporary hydrostatic uplift pressure relief (or groundwater dewatering) system that is required to be operated until enough ballast in the form of soil (e.g., liner protective cover and/or final cover) and solid waste are deposited over these areas. Appendix IIID – LQCP includes the design for the dewatering system (Appendix IIID-C) and ballast demonstrations (Appendix IIID-B).

The dewatering system design for Cells 7 and 8 includes a trench installed at the toe of slope excavation with geocomposite installed over the Upper Sand Stratum, allowing groundwater to drain into the toe trench. The gravel back-filled trench will be graded to drain to either an open excavation area or a sump.

The groundwater dewatering system design included in Appendix IIID-C has been developed based on the following:

- Hydraulic conductivity values of the Upper Sand Stratum identified in Section 3 – Groundwater Investigation Report of Appendix IIIG – Geology Report.
- Excavation plan included in Appendix IIIA, Figure IIIA-A.1, of this permit application.

The groundwater system design is based on the assumption that the Upper Sand Stratum is the primary water-bearing formation that can affect landfill construction, and that Upper Sand Stratum exposed in the excavation sideslopes requires dewatering.

5.2 Dewatering System Design

The evaluation of potential water inflow into the dewatering system and the design of the dewatering system for the undeveloped portions the Type I MSW area is provided in Appendix IIID – LQCP.

5.3 Design of Ballast for Liner Protection Against Potential Hydrostatic Uplift Pressure

The evaluation of ballast used to protect the liner against hydrostatic pressure for the currently installed and future dewatering system areas is provided in Appendix IIID – LQCP.

6 SLOPE STABILITY ANALYSIS

6.1 General

This slope stability analysis has been developed to analyze excavation slopes, interim slopes, and landfill completion slopes using critical sections for each condition. XSTABL 5.2, a computer program developed to model general slope stability by the Simplified Bishop and Simplified Janbu methods, was used to analyze the stability of excavation slopes, constructed liner slopes, constructed protective cover slopes, interim fill slopes, and the final cover configuration of the site. Circular failure surfaces using the Simplified Bishop method were used for the stability analyses of all of the slopes for the configurations listed above. The Simplified Janbu method using Rankine Blocks was used to analyze the transitional stability at the interfaces of underlying materials and liner materials. Additionally, infinite slope stability analysis has been performed to evaluate interface stability for the liner and the final cover system components. The slope stability analyses are provided in Appendix IIIE-A.

6.2 Sections Selected for Analysis

Slope stability analyses were performed on critical sections of the Type I MSW area to evaluate the stability of the excavation, end of liner construction, interim fill, and final cover configuration slopes. The geometries of the slopes analyzed were determined by reviewing the proposed excavation plan and final contour plan. The evaluation locations were selected to analyze critical slopes consisting of profiles that include the landfill configuration as well as natural materials at the toe and below the landfill excavation. The interim fill slope was analyzed using an assumed profile as discussed in Section 6.3. Figures showing the location of the cross sections are included in Appendix IIIE-A (refer to Appendix IIIE-A-1 for the stability analysis location plans and cross-section figures, and IIIE-A-2 for the stability analysis output files.)

6.3 Configurations Analyzed

The excavation, constructed liner, interim, and final landfill configurations were modeled to represent critical slope conditions, and the analysis was performed using circular and transitional failure surfaces. The maximum final fill slopes will be 4 horizontal to 1 vertical (4H:1V), while interim fill slopes, constructed liner slopes, and excavation slopes could be as steep as 3H:1V. Therefore, the excavation, constructed liner, and interim fill slopes were analyzed with an angle of 3H:1V, while a maximum of 4H:1V final cover slope was used for slope stability analysis. The excavation plan and the proposed final contour plan showing the location of the cross sections selected for analysis are included in Appendix IIIE-A. The interim condition was analyzed considering a 3H:1V slope with a horizontal length of approximately 415 feet. The actual analysis of the interim condition is conservative. If actual interim slopes longer than 415 feet are developed during site operations, an additional analysis will be completed at that time and maintained in the Site Operating Record.

6.4 Input Parameters

The cross sections for slope stability analyses were developed from the proposed excavation and final cover plans. The soil parameters were selected based on a review of the boring logs and laboratory test results from the subsurface investigation studies at the site and upon engineering judgment and experience with similar materials. Table 6-1 summarizes the material properties and strength parameters used for the stability analyses.

6.5 Results of Stability Analysis

The results of the stability analyses indicate that the proposed excavation, constructed liner, interim waste fill, and final configuration slopes are stable under the conditions analyzed. Tables 6-2 and 6-3 summarize the results of the stability analyses and compare the calculated factor of safety to the recommended minimum factor of safety. The recommended minimum factors of safety for the conditions analyzed were determined using recommendations from the USACE "Design and Construction of Levees" manual (EM 1110-2-1913) and the EPA's "Technical Guidance Manual for Design of Solid Waste Disposal Facilities." Computer-generated slope stability analysis output is included in Appendix IIIE-A-2.

Additionally, an infinite slope stability analysis has been performed to evaluate interface stability for the liner and the final cover system components. Since the liner system will be stable with the protective cover on the slope during construction, there should not be any tensile stress buildup at the anchor trench (the infinite slope stability analysis and the anchor trench design are provided in Appendix IIIE-A-4). The actual material interface strength parameters will be verified prior to construction. The analysis was developed using peak strength values and a factor of safety of 1.5 (long-term condition) and 1.3 (short-term condition). If large displacement (i.e., residual strength) values are tested for, then a factor of safety of 1.0 will be required.

Table 6-1
Summary of Material Weight and Strength Parameters Used in the Slope Stability Analysis

Strength Parameters | **Comments**

Final Cover System

Material Strength Parameters			Interface Strength Parameters	
Cohesion (lb/ft^2)	Friction Angle (degrees)	Unit Weight (lb/ft^3)	Adhesion (lb/ft^2)	Friction Angle (degrees)
100	16	116	Topslope 100	11
			4H:1V Sdeslope 100	15

The final cover system includes the erosion layer, drainage geocomposite (single-sided on topslopes and double-sided on 4H:1V sideslopes), geomembrane liner (smooth on topslopes and textured on 4H:1V sideslopes), and compacted clay infiltration layer. This system is modeled as a single layer for the global stability analysis. In addition, an infinite stability analysis was performed to establish the minimum interface strength requirements for each layer of the final cover system. The minimum interface strength requirements are specified in Appendix IIIJ-A.

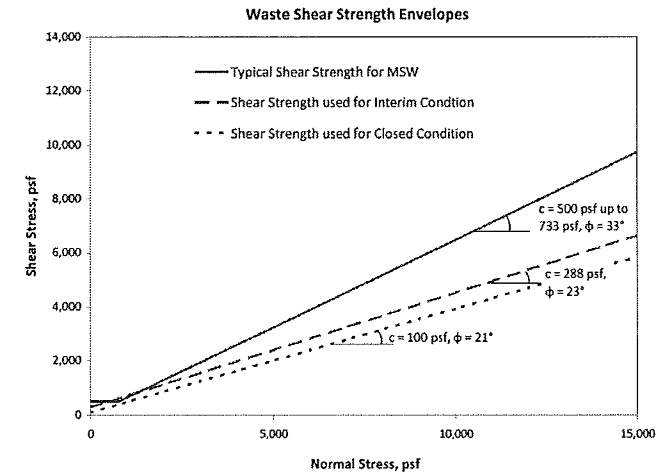
For the rotational global stability analysis, the final cover system is modeled as a single layer and the strength parameters represent the compacted clay infiltration layer and the erosion layer. The two geosynthetic layers are not included in the global analysis because they provide a negligible contribution to the forces that are resisting movement. The strength values selected for the final cover system represent strength values typically used in the industry and these same strength values have been used in various permit applications approved by TCEQ. The global stability analysis uses the material strength parameters (i.e., cohesion of 100 lb/ft^2 and a friction angle of 16 degrees). The unit weight of the final cover system is consistent with the protective cover of the liner system, and is based on experience with final cover construction. The global stability analyses are included in Appendix IIIE-A-2.

The interface slope stability analysis, which is performed using an infinite slope stability analysis procedure for the final cover system, was developed to show that certain landfill components that are placed on top of each other, such as a geomembrane and compacted clay layer (or geomembrane and geocomposite), will not experience sliding failure due to the lack of strength between these components. The strength parameters were developed from Geosynthetic Research Institute (GRI) publications (e.g., "Direct Shear Database of Geosynthetic-to-Geosynthetic and Geosynthetic-to-Soil Interfaces" by George R. Koerner, GRI, Folsom, PA, June 14, 2005). Although the strength parameters (i.e., adhesion and interface friction) used for the application were selected based on published data, it should be noted that these strength parameters will also be tested and verified at the time of each final cover construction event to ensure that the as-built strength parameters meet or exceed the strength parameters used for the design (refer to Appendix IIIJ-A, Section 3.4.3 for the specific design specifications). As noted in Appendix IIIJ-A, Table 3-3, the strength parameters listed are for the weakest interface to provide for a conservative design.

Solid Waste

Material Strength Parameters			Interface Strength Parameters	
Cohesion (lb/ft^2)	Friction Angle (degrees)	Unit Weight (lb/ft^3)	Adhesion (lb/ft^2)	Friction Angle (degrees)
Interim Condition 288	23	66	Interface strength parameters are not applicable to the solid waste layer because the interface between the waste/final cover system and the waste/liner system is not a critical interface.	
Closed Condition 100	21	66		

The strength parameters for solid waste were based on information contained in the following references: Pagotto and Rimoldi (1987), Landva and Clark (1990), and Richardson and Reynolds (1991). These sources list cohesion and friction angle values that range from 210 lb/ft^2 to 605 lb/ft^2 and 18° (these are residual strength or large displacement values which require a factor of safety of 1) to 43°, respectively. Refer to Appendix IIIE-A-3 (page IIIE-A-3-9) for more information. Based on this information, the waste strength values are conservatively selected to be a cohesion of 288 psf and a friction angle of 23°. These waste strength parameters have been used extensively in the past for MSW landfills in Texas. These strength parameters (c=288psf and ϕ=23°) are also consistent with the stability analysis included in Appendix IIIA-C (page IIIA-C-4-3). As shown in the figure to the right, the shear strength envelope used in this application is below the typical MSW strength envelope published in "Soil Strength and Slope Stability" (Duncan and Wright, 2005).



Liner System

Material Strength Parameters			Interface Strength Parameters	
Cohesion (lb/ft^2)	Friction Angle (degrees)	Unit Weight (lb/ft^3)	Adhesion (lb/ft^2)	Friction Angle (degrees)
Protective Cover	100 16	116	Floor Grades 100	11
Liner System	100 16	120	3H:1V Sideslope 100	15

The liner system includes a 2-foot-thick (MSW) compacted clay layer, 60-mil geomembrane (smooth geomembrane on the floor of the landfill and textured on the 3H:1V sideslopes), drainage geocomposite (single-sided on floor grades and double-sided on 3H:1V sideslopes), and a 2-foot-thick protective cover soil layer. This system is modeled as two layers for the global stability analysis: the compacted clay liner and the soil protective cover. In addition, both a transitional and an infinite stability analysis were performed to establish the minimum interface strength requirements for each layer of the liner system. The minimum interface strength requirements are specified in Appendix IIID.

For the rotational global stability analysis, the liner system is modeled as two layers: the compacted clay liner and the soil protective cover layer. The two geosynthetic layers are not included in the global analysis because they provide a negligible contribution to the forces that are resisting movement. The strength values selected for the liner system represent strength values typically used in the industry. Duncan and Wright (2005) provides a comprehensive discussion regarding strength parameters for a liner system. In Chapter 5 – Shear Strengths of Soil and Municipal Solid Waste, a significant amount of data are presented and evaluated for compacted clay liners. The results indicate that the lowest cohesion value for compacted cohesive soils is 9 kPa (187 lb/ft^2) and the lowest reported friction angle value is 19 degrees. Therefore, selected values of 100 lb/ft^2 for cohesion and 16 degrees of friction angle conservatively represent the liner system. Soil properties used in the slope stability analysis are subject to verification at the time of each liner construction. Section 2.4.3 in Appendix IIID – LQCP includes the material strength tests required for soil used for liner construction. Protective cover and compacted clay liner soil unit weight values are based on experience with liner system construction. Compacted clay is assigned a slightly higher moist unit weight compared with the protective cover because of the compaction effort that is applied to the clay liner. The global stability analysis is included in Appendices IIIE-A-1 and IIIE-A-3.

The interface slope stability analysis, which is performed using an infinite slope stability analysis procedure for the liner system, was developed to show that certain landfill components that are placed on top of each other, such as a geomembrane and compacted clay layer (or geomembrane and geocomposite), will not experience sliding failure due to the lack of strength between these components. The strength parameters were developed using materials from Geosynthetic Research Institute (GRI) publications (e.g., "Direct Shear Database of Geosynthetic-to-Geosynthetic and Geosynthetic-to-Soil Interfaces" by George R. Koerner, GRI, Folsom, PA, June 14, 2005). Although the strength parameters (i.e., adhesion and interface friction) used for the application were selected based on published data, it should be noted that these strength parameters will also be tested and verified at the time of each liner construction event to ensure that the as-built strength parameters meet or exceed the strength parameters used for the design (refer to Appendix IIID Section 3.5.2 for the specific design specifications). As noted in Appendix IIID, Table 3-5, the strength parameters listed are for the weakest interface to provide for a conservative design.

The transitional slope stability analysis was performed using Simplified Janbu Method using Rankine Blocks. This analysis is similar to the interface slope stability analysis discussed above. The purpose of this analysis is to test the critical interfaces under a variety of loading conditions (refer to Appendices IIIE-A-1 and IIIE-A-2) for more information – i.e., the loading conditions reflect different landfill configurations). Like the global slope stability analysis, XSTABL is also used for this analysis. However, for the transitional analysis the liner system strength parameters are modified to reflect the interface strength parameters. As noted above, these strength parameters will also be tested and verified at the time of each liner construction event to ensure that the as-built strength parameters meet or exceed the strength parameters used for the design.

Table 6-1
Summary of Material Weight and Strength Parameters Used in the Slope Stability Analysis
(Continued)

Strength Parameters					Comments
Liner System					The liner system includes a 2-foot-thick (MSW) compacted clay layer, 60-mil geomembrane (smooth geomembrane on the floor of the landfill and textured on the 3H:1V sideslopes), drainage geocomposite (single-sided on floor grades and double-sided on 3H:1V sideslopes), and a 2-foot-thick protective cover soil layer. This system is modeled as two layers for the global stability analysis: the compacted clay liner and the soil protective cover. In addition, both a transitional and an infinite stability analysis were performed to establish the minimum interface strength requirements for each layer of the liner system. The minimum interface strength requirements are specified in Appendix IIID. For the rotational global stability analysis, the liner system is modeled as two layers: the compacted clay liner and the soil protective cover layer. The two geosynthetic layers are not included in the global analysis because they provide a negligible contribution to the forces that are resisting movement. The strength values selected for the liner system represent strength values typically used in the industry. Duncan and Wright (2005) provides a comprehensive discussion regarding strength parameters for a liner system. In Chapter 5 – Shear Strengths of Soil and Municipal Solid Waste, a significant amount of data are presented and evaluated for compacted clay liners. The results indicate that the lowest cohesion value for compacted cohesive soils is 9 kPa (187 lb/ft^2) and the lowest reported friction angle value is 19 degrees. Therefore, selected values of 100 lb/ft^2 for cohesion and 16 degrees of friction angle conservatively represent the liner system. Soil properties used in the slope stability analysis are subject to verification at the time of each liner construction. Section 2.4.3 in Appendix IIID – LQCP includes the material strength tests required for soil used for liner construction. Protective cover and compacted clay liner soil unit weight values are based on experience with liner system construction. Compacted clay is assigned a slightly higher moist unit weight compared with the protective cover because of the compaction effort that is applied to the clay liner. The global stability analysis is included in Appendices IIIE-A-1 and IIIE-A-2. The interface slope stability analysis, which is performed using an infinite slope stability analysis procedure for the liner system, was developed to show that certain landfill components that are placed on top of each other, such as a geomembrane and compacted clay layer (or geomembrane and geocomposite), will not experience sliding failure due to the lack of strength between these components. The strength parameters were developed using materials from Geosynthetic Research Institute (GRI) publications (e.g., "Direct Shear Database of Geosynthetic-to-Geosynthetic and Geosynthetic-to-Soil Interfaces" by George R. Koerner, GRI, Folsom, PA, June 14, 2005). Although the strength parameters (i.e., adhesion and interface friction) used for the application were selected based on published data, it should be noted that these strength parameters will also be tested and verified at the time of each liner construction event to ensure that the as-built strength parameters meet or exceed the strength parameters used for the design (refer to Appendix IIID, Section 3.5.2 for the specific design specifications). As noted in Appendix IIID, Table 3-5, the strength parameters listed are for the weakest interface to provide for a conservative design. The transitional slope stability analysis was performed using Simplified Janbu Method using Rankine Blocks. This analysis is similar to the interface slope stability analysis discussed above. The purpose of this analysis is to test the critical interfaces under a variety of loading conditions (refer to Appendices IIIE-A-1 and IIIE-A-2) for more information – i.e., the loading conditions reflect different landfill configurations). Like the global slope stability analysis, XSTABL is also used for this analysis. However, for the transitional analysis the liner system strength parameters are modified to reflect the interface strength parameters. As noted above, these strength parameters will also be tested and verified at the time of each liner construction event to ensure that the as-built strength parameters meet or exceed the strength parameters used for the design.
Material Strength Parameters			Interface Strength Parameters		
Cohesion (lb/ft^2)	Friction Angle (degrees)	Unit Weight (lb/ft^3)	Adhesion (lb/ft^2)	Friction Angle (degrees)	
Protective Cover 100	16	116	Floor Grades 100	11	
Liner System 100	16	120	3H:1V Sideslope 100	15	
Embankment Soil					The embankment soil will be constructed similar to the compacted clay liner; therefore, it will be assigned the same material strength values as the liner system.
Material Strength Parameters			Interface Strength Parameters		
Cohesion (lb/ft^2)	Friction Angle (degrees)	Unit Weight (lb/ft^3)	Adhesion (lb/ft^2)	Friction Angle (degrees)	
100	16	125	Interface strength parameters are not applicable to the embankment soil, because the interface between the embankment soil and the liner system is addressed in the interface testing of the liner system.		
Upper Clay Stratum					A triaxial shear test was performed on the Upper Clay Stratum which resulted in cohesion values of 718 psf (total stress) and 644 psf (effective stress). Therefore, cohesion values of 718 psf and 644 psf (total and effective stress conditions, respectively) were selected. The internal friction angles were taken from the triaxial shear test (11.4 degrees for total stress and 12.6 degrees for effective stress). Moist unit weight values are calculated from the moisture content and dry unit weight data obtained from laboratory testing performed on the material. As noted in this table, both effective stress and total stress strength parameters are assigned to this layer. A total stress analysis is applicable to the surface clay soils because there is a potential that some of the clay soils may be saturated or close to a saturated condition. If this is the case, excessive pore pressure could develop within this zone, so a total stress analysis was completed.
Material Strength Parameters			Interface Strength Parameters		
Cohesion (lb/ft^2)	Friction Angle (degrees)	Unit Weight (lb/ft^3)	Adhesion (lb/ft^2)	Friction Angle (degrees)	
Effective 644 Total 718	Effective 12.6 Total 11.4	130.1	Interface strength parameters are not applicable to surface clay because the interface between the bottom of the compacted clay liner and surface clay is not a critical surface.		

Table 6-2
Factor of Safety Summary for Short-Term Slope Stability Analyses

Description		Minimum Factor of Safety Generated		Recommended Minimum Factor of Safety	Acceptable Factor of Safety
Slope Designation	Method of Analysis	Total Stress	Effective Stress		
Excavated Slope E1-1	Bishop-Circular	2.00	2.09	1.3	YES
Excavated Slope E1-2	Rankine-Block	2.95	2.84	1.3	YES
Excavated Slope E1-3[1]	Rankine-Block	6.09	5.94	1.3	YES
Constructed Liner Slope E1-4	Bishop-Circular	1.98	1.98	1.3	YES
Constructed Liner Slope E1-5	Rankine-Block	2.60	2.50	1.3	YES
Interim Waste Slope E2-6	Bishop-Circular	1.61	1.63	1.5	YES
Interim Waste Slope E2-7	Rankine-Block	1.61	2.21	1.5	YES

[1] Analysis incorporates a clay slickensided seam.

Table 6-3
Factor of Safety Summary for Long-Term Slope Stability Analyses

Description		Minimum Factor of Safety Generated		Recommended Minimum Factor of Safety		Acceptable Factor of Safety
Slope Designation	Method of Analysis	Total Stress	Effective Stress	Total Stress	Effective Stress	
Final Cover Slope E3-8	Bishop-Circular	1.98	2.02	1.3	1.5	YES
Final Cover Slope E3-9	Rankine-Block	2.21	1.61	1.3	1.5	YES

7 SETTLEMENT AND HEAVE ANALYSIS

7.1 General

The purpose of the settlement analysis is to demonstrate that the liner system will not be adversely impacted by overlying landfill components (e.g., waste, daily cover, etc.). The settlement analysis also addresses the settlement of the final cover system to demonstrate that the proposed final cover is designed to withstand the potential strain induced by waste settlement.

Settlement of the liner system will occur due to consolidation of the foundation materials from the weight of the landfill components (i.e., protective cover, solid waste and daily cover, and final cover systems). Settlement of the final cover system will occur due to consolidation of foundation soils and consolidation within the solid waste. Total consolidation of final cover consists of primary and secondary consolidation of the waste column.

7.2 Foundation Heave

Potential heave (rebound) due to excavation of overburden soil above the excavation base was estimated using the standard consolidation theory for soils and the swell index obtained from the rebound portion of the consolidation tests. In order to estimate potential for heave, the load is decreased, instead of increasing the load on the soils, to correspond with the projected weight of excavated soil. Using a maximum excavation depth of approximately 32 feet (existing ground elevation minus bottom of excavation at a given location), a heave of approximately 1.16 feet was calculated. The depth of excavation for each individual cell (lined area draining to an LCS sump) is generally uniform (i.e., depth of soil to be removed from the floor grades does not change drastically within a given cell). Therefore, the change in the excavation slopes after heave, which is expected to occur within a short period of time after excavation, will not be significant. The heave analysis for the excavation areas is included in Appendix IIIE-B.

7.3 Foundation Settlement

In general, landfill foundation settlement occurs as foundation materials consolidate due to the weight of the landfill. Foundation settlement was predicted using standard consolidation theory for soils. Consolidation data are provided in

Appendix IIIE-C. The excavation grades are founded primarily within the Upper Clay Stratum. The settlement calculations were based on a maximum applied load of waste overlying the settlement evaluation points shown in Appendix IIIE-C for solid waste with unit weight that varies based on height of fill, and cover with an average unit weight of 116 pcf.

Based on the result of the settlement analysis, the subgrade consolidation will not exceed 3.03 feet. The settlement of the liner system will be generally uniform and will not adversely affect the performance of the liner or leachate collection system. Strain for the liner system is calculated by using the calculated settlement. The maximum strain calculated is 0.0077 percent. This is below the strain values that the liner system components (e.g., geomembrane, geocomposite, compacted clay) can withstand. These calculations are included in Appendix IIIE-B. The final deflected shape of the liner will generally consist of gradual transitions with the differential settlement occurring over several hundred feet or more (horizontal projection). Based on the foregoing discussion, it is concluded that settlement will not adversely affect the liner system or flow in the leachate collection system.

7.4 Final Cover Settlement

Landfill final cover settlement occurs due to settlement of foundation materials and the settlement of waste materials. In general, foundation settlement is insignificant in comparison to the settlement of deposited waste. Waste settlement consists of primary and secondary settlement.

Settlement of solid waste generally begins rapidly as the waste load is placed and continues to occur for long periods of time after the initial placement. Initially, municipal solid waste will undergo primary settlement due to its own weight, final cover, equipment, etc. Primary settlement occurs quickly, generally within the first month after loading. Therefore, the weight of the final cover system is the only remaining factor that contributes to primary consolidation. By the time the construction of the final cover is complete, settlement of the waste due to the weight of the final cover will be complete. Secondary settlement continues at substantial rates for periods of time well beyond primary settlement. It is a combination of mechanical secondary compression, physico-chemical reaction, and biochemical decay. Settlement analysis for the final cover system is presented in Appendix IIIE-B.

A strain analysis has been incorporated into the final cover settlement analysis presented in Appendix IIIE-B. The purpose of the settlement and strain analysis is to demonstrate that the final cover system will be stable as designed and maintain positive drainage. If it is considered that the waste settlement is uniform, then the side slopes are expected to maintain positive drainage. A strain demonstration in Appendix IIIE-B shows that the top deck and side slope areas of the final cover will be stable and maintain positive drainage after settlement.

8 CONCLUSIONS AND RECOMMENDATIONS

- Some of the material excavated at the site can be used for the soil liner, and final cover infiltration layer. Excavated soils that meet the requirements of the final cover infiltration layer may be separately stockpiled and processed as discussed in Appendix IIID, Section 2.
- Stability of the landfill excavation slopes, constructed liner slopes, interim fill slopes, and the final cover slopes is acceptable as designed.
- Stability of the liner and final cover system components is acceptable as designed.
- Foundation heave during excavation is expected to be within an acceptable range.
- Foundation settlement after filling is expected to be within the strain limits of the liner system.
- Settlement of the final cover system will not adversely affect the final cover system components, and the final cover system will function as designed.

APPENDIX IIIE-A

SLOPE STABILITY ANALYSIS

Includes page IIIE-A-1

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12-5-17

CONTENTS

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INTRODUCTION

This appendix includes the slope stability analysis for the landfill slopes during various phases of the site development and the final landfill configuration. General slope stability for the excavation, constructed liner system, and interim and closed conditions was evaluated by using the XSTABL 5.2 computer program. The Simplified Bishop method was used for circular failure surfaces, and the Simplified Janbu method using Rankine Block was used for the transitional slope stability analysis. Infinite slope stability has also been analyzed for the liner and final cover system. The stability analysis for the site is provided in the following five appendices. A generalized soil profile for the slope stability analysis is provided on Sheet IIIE-A-2.

- Appendix IIIE-A-1 includes the slope stability section location plans and sections analyzed.
- Appendix IIIE-A-2 includes the slope stability analysis for the excavated and constructed liner conditions, interim waste fill conditions, and the closed landfill conditions. Results are presented for analysis suing both total and effective stress.
- Appendix IIIE-A-3 includes the infinite slope stability evaluation for the liner and final cover systems.

APPENDIX IIIE-A-1

SLOPE STABILITY ANALYSIS
SECTION PLANS AND CROSS-SECTIONS

Includes pages IIIE-A-1-1 through IIIE-A-1-4

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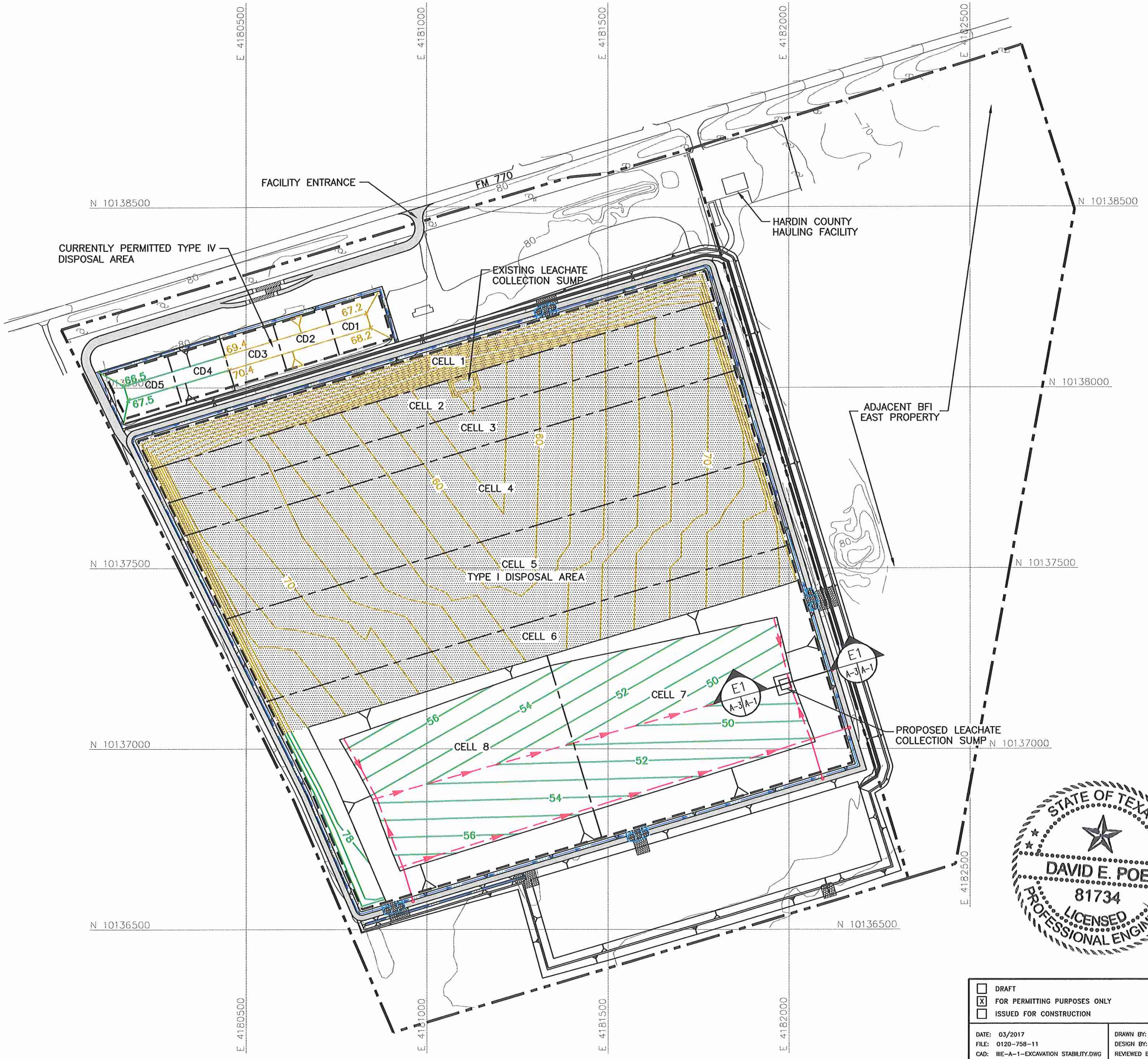

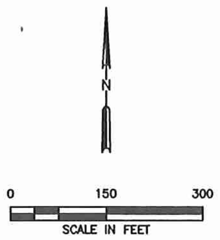

LEGEND

— · · —	BFI EAST PROPERTY BOUNDARY
— · —	PERMIT BOUNDARY
— — —	CURRENTLY PERMITTED LIMITS OF WASTE
—70—	EXISTING CONTOUR (SEE NOTE 1)
N 10137000	STATE PLANE COORDINATE SYSTEM (SEE NOTE 1)
— —	CELL BOUNDARY
—50—	PROPOSED EXCAVATION CONTOUR
—70—	CONSTRUCTED TOP OF PROTECTIVE COVER CONTOUR
— ← —	PROPOSED LEACHATE LINE
—●	PROPOSED LEACHATE RISER
▨	EXISTING SUBTITLE D COMPOSITE LINER AREA

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS DEVELOPED BY WEAVER CONSULTANTS GROUP FROM AERIAL PHOTOGRAPHY FLOWN 05-17-2016. GRID SYSTEM IS TIED TO THE TEXAS STATE PLANE COORDINATE SYSTEM NAD 83, CENTRAL ZONE.
2. CURRENTLY PERMITTED TYPE IV CELLS INCLUDES CD1 THROUGH CD5. CD1, CD2, AND CD3 ARE CURRENTLY DEVELOPED AND RECEIVED TYPE IV WASTE.

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DATE: 03/2017
FILE: 0120-758-11
CAD: IIIE-A-1-EXCAVATION STABILITY.DWG

DRAWN BY: SRF
DESIGN BY: AE
REVIEWED BY: NT

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

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NO.	DATE	DESCRIPTION
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MAJOR PERMIT AMENDMENT EXCAVATION STABILITY SECTION LOCATION

HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS

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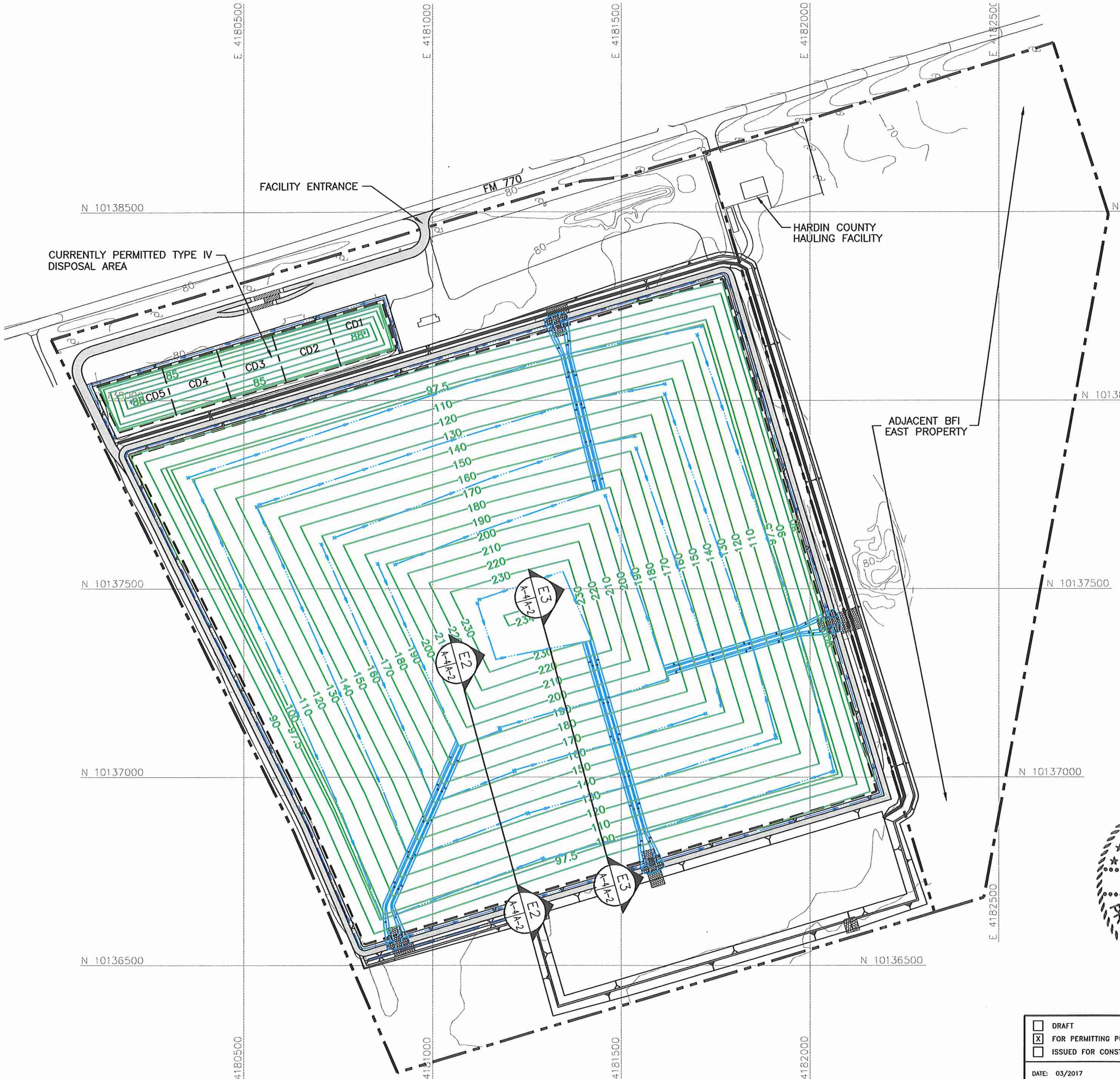
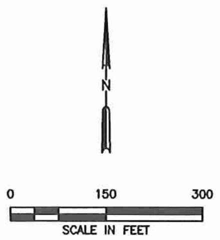
FIGURE IIIE-A-1-1

O:\0120\758\2214B EXPANSION\IIIE\IIIE-A-1 EXCAVATION STABILITY SECTION LOCATION.dwg, 11/15/2017 8:27:00 AM, rsellers, 1:2





LEGEND

— — · — · —	BFI EAST PROPERTY BOUNDARY
—— · · ——	PERMIT BOUNDARY
— — — —	CURRENTLY PERMITTED LIMITS OF WASTE
—— — ——	CELL BOUNDARY
N 10137500	STATE PLANE COORDINATE GRID
——70——	EXISTING CONTOUR
——234——	PROPOSED FINAL CONTOUR (SEE NOTE 3)
—→—→—	PROPOSED DRAINAGE SWALE
▭	PROPOSED DRAINAGE CHUTE

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS DEVELOPED BY WEAVER CONSULTANTS GROUP FROM AERIAL PHOTOGRAPHY FLOWN 05-17-2016. GRID SYSTEM IS TIED TO THE TEXAS STATE PLANE COORDINATE SYSTEM NAD 83, CENTRAL ZONE.
2. MAXIMUM FINAL COVER ELEVATION IS 234 FT-MSL. MAXIMUM TOP OF WASTE ELEVATION IS 230.5 FT-MSL.

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MAJOR PERMIT AMENDMENT
FINAL AND INTERIM STABILITY SECTION LOCATIONS

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FIGURE IIIE-A-1-2

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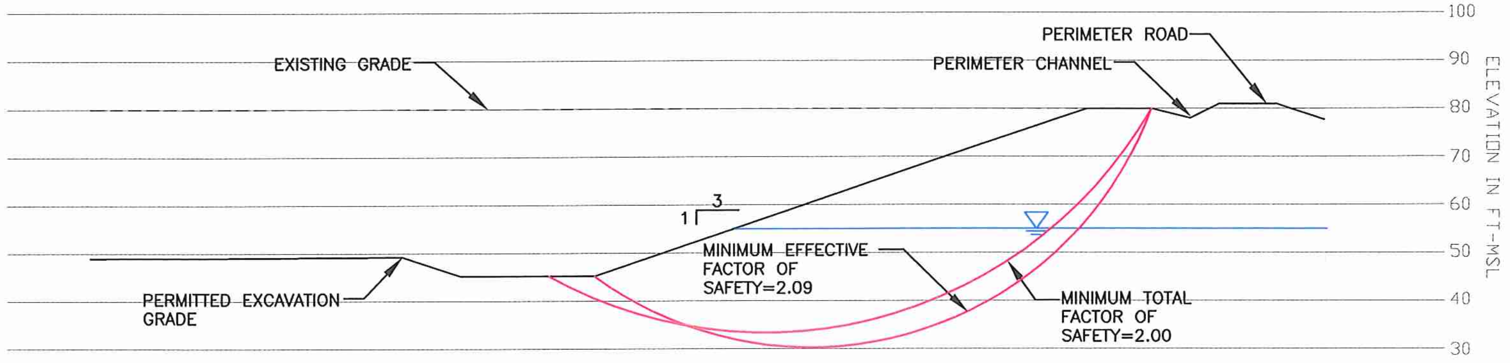

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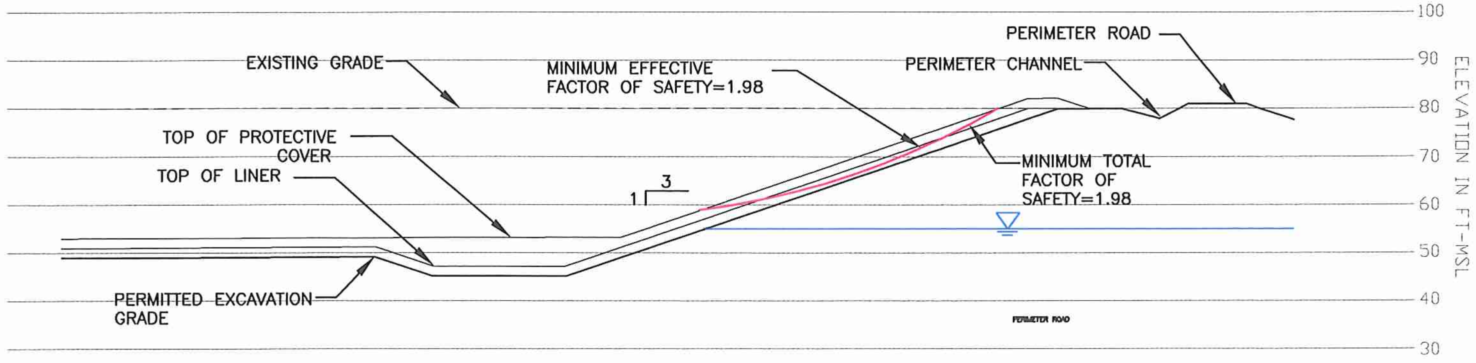

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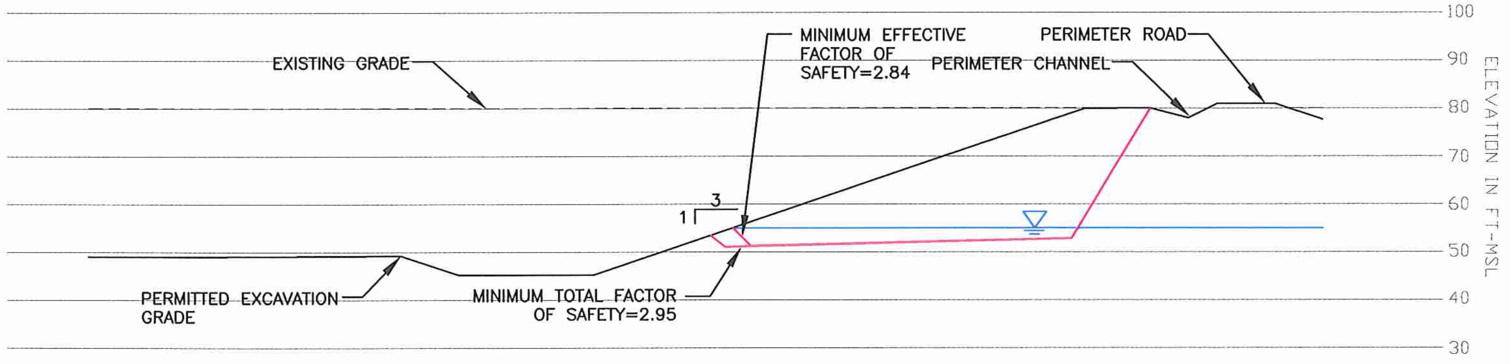

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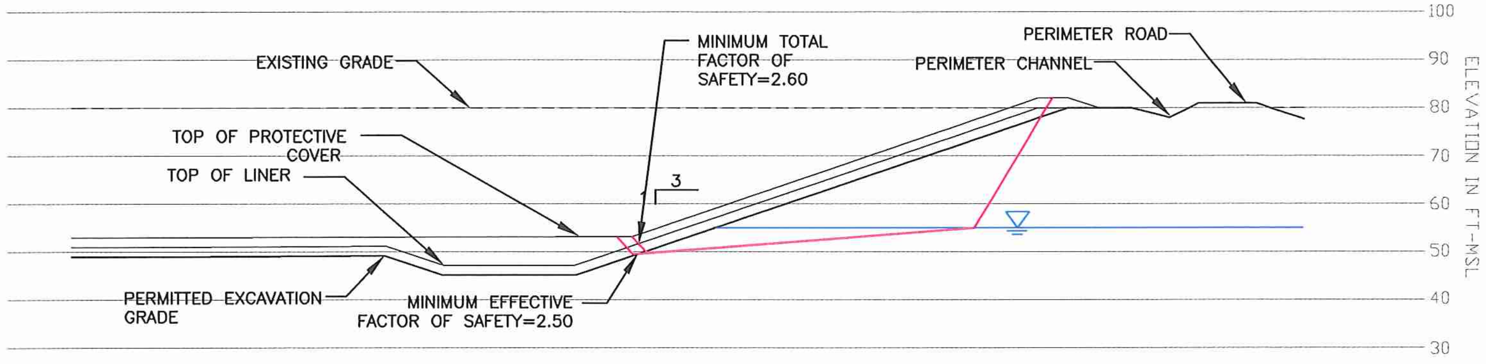

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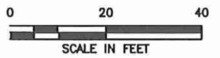

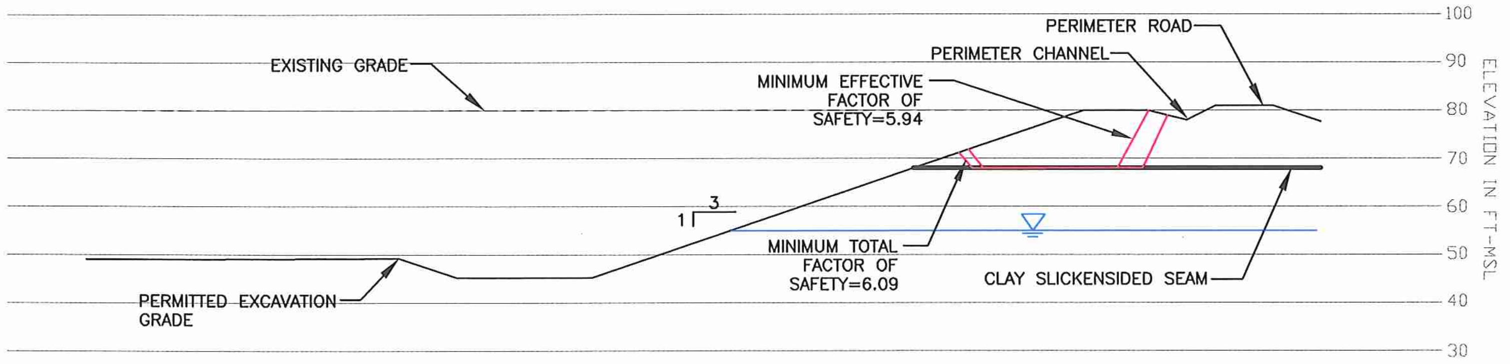

EXCAVATION SECTION E1–3

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MAJOR PERMIT AMENDMENT EXCAVATION STABILITY ANALYSIS SECTIONS

HARDIN COUNTY LANDFILL
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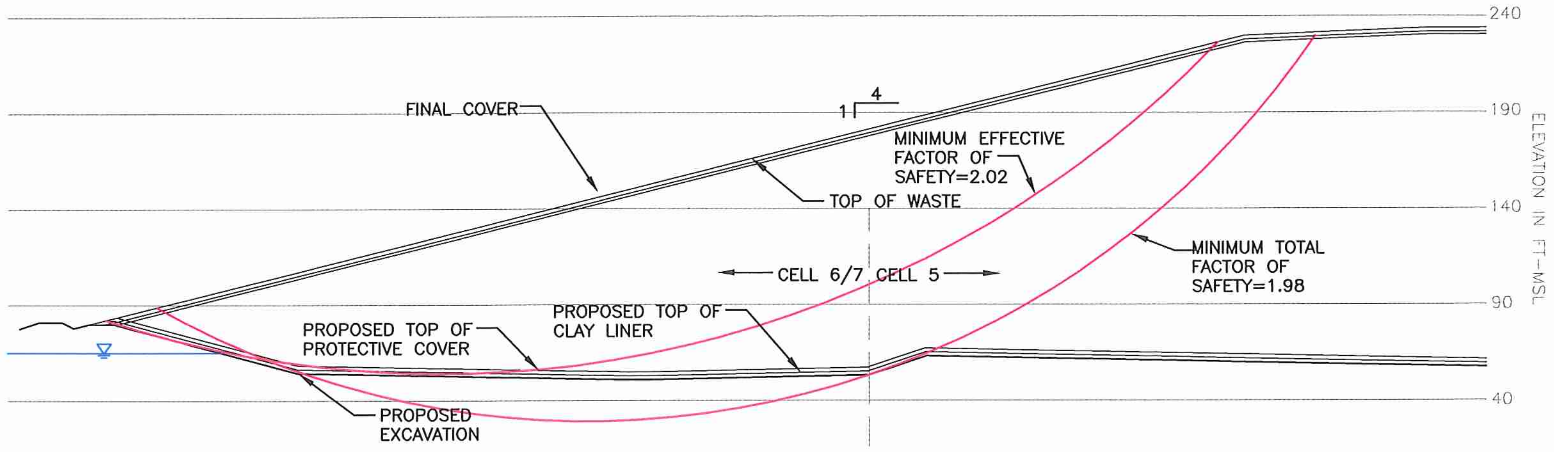
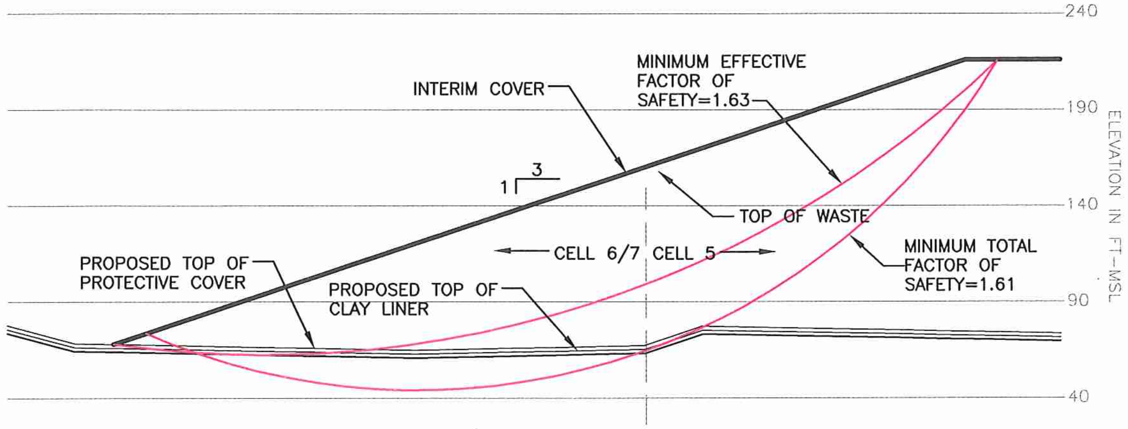
FIGURE IIIE–A–1–3

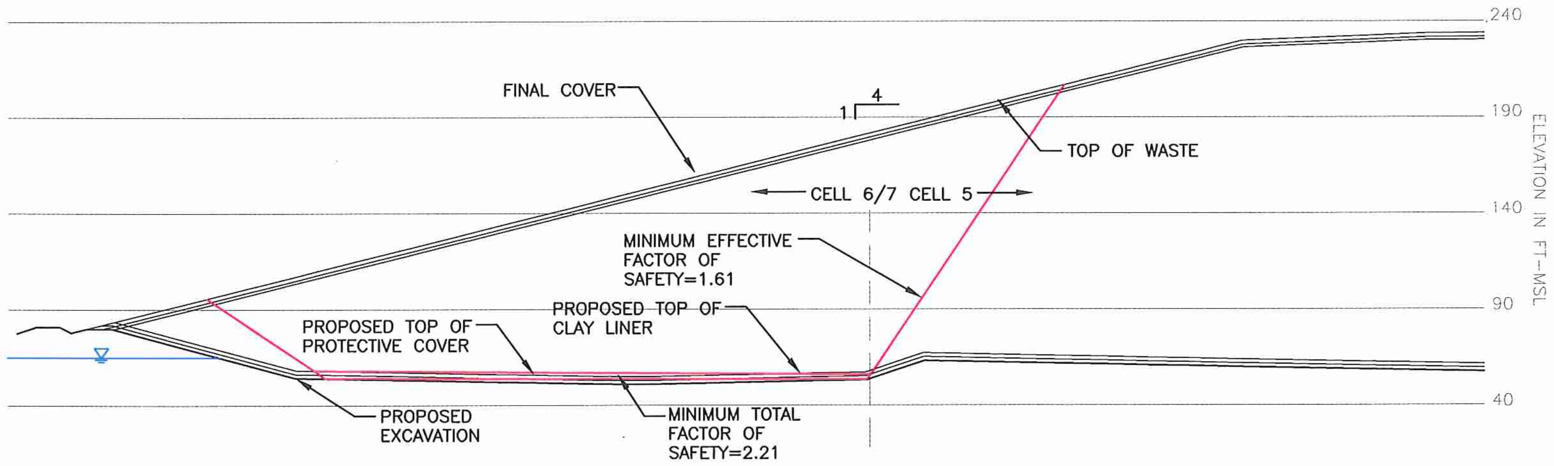
O:\0120\758\2214B EXPANSION\IIIE\IIIE-A-3 EXCAVATION STABILITY ANALYSIS SECTIONS.dwg, 11/15/2017 8:34:13 AM, rsellers, 1:2



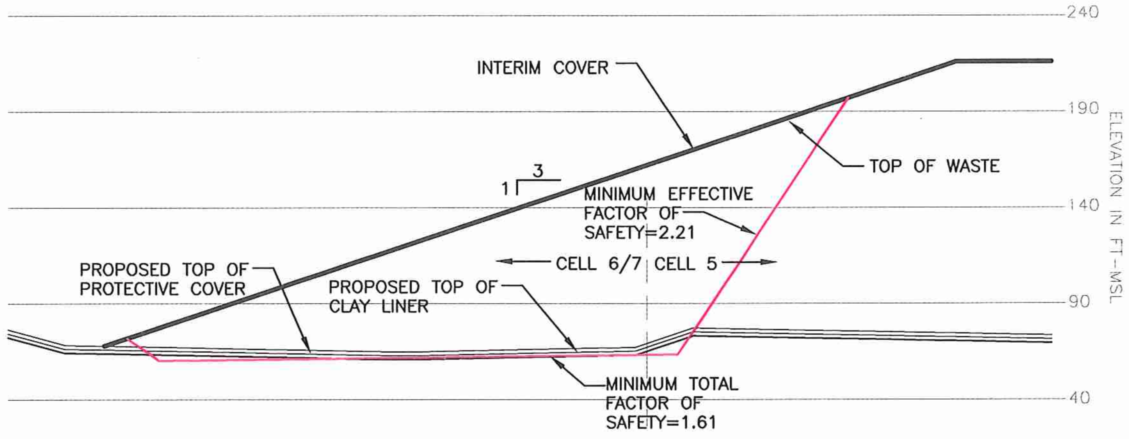
FINAL COVER SECTION E3–8



INTERIM COVER SECTION E2–6



FINAL COVER SECTION E3–9



INTERIM COVER SECTION E2–7

0 50 100
SCALE IN FEET

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CAD: IIIE-A-1-4 STABILITY ANALYSIS.DWG

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DESIGN BY: CCJ
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FINAL AND INTERIM STABILITY ANALYSIS SECTIONS

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HARDIN COUNTY, TEXAS

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FIGURE IIIE-A-1-4

O:\0120\758\2214B EXPANSION\IIIE\IIIE-A-4 FINAL AND INTERIM STABILITY ANALYSIS SECTIONS.dwg, 11/15/2017 8:34:37 AM, rsellers, 1:2

APPENDIX IIIE-A-2

SLOPE STABILITY ANALYSIS
RESULTS (ALL CONDITIONS, TOTAL AND EFFECTIVE)

Includes pages IIIE-A-2-1 through IIIE-A-2-140

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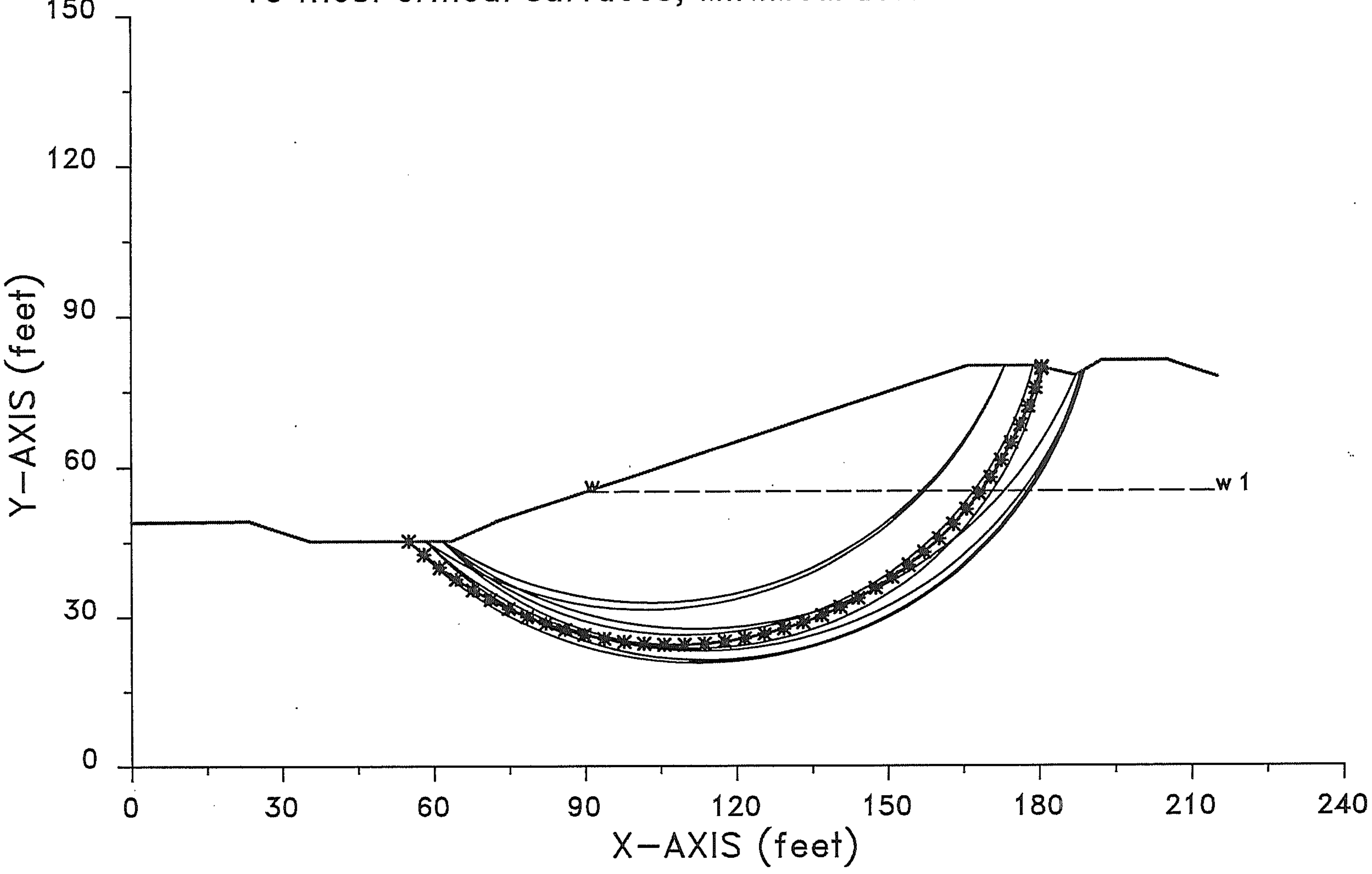
SECTION E1-1 (FIGURE IIIE-A-3)

EXCAVATED SLOPE STABILITY ANALYSIS
BISHOP CIRCULAR METHOD
TOTAL AND EFFECTIVE STRESS MODELS

EX-2-1-T 2-03-17 16:21

HARDIN CO LF INT PER SLOPE ANALYSIS
10 most critical surfaces, MINIMUM BISHOP FOS = 1.998



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XSTABL File: EX-2-1-T   2-03-17   16:21

          ******************************************
          *              X S T A B L               *
          *                                        *
          *        Slope Stability Analysis        *
          *              using the                 *
          *            Method of Slices            *
          *                                        *
          *        Copyright (C) 1992 - 2008       *
          *   Interactive Software Designs, Inc.   *
          *        Moscow, ID 83843, U.S.A.        *
          *                                        *
          *          All Rights Reserved           *
          *                                        *
          *  Ver. 5.208                 96 - 2046  *
          ******************************************


    Problem Description : HARDIN CO LF INT PER SLOPE ANALYSIS



    -----------------------------
    SEGMENT BOUNDARY COORDINATES
    -----------------------------

       10 SURFACE boundary segments

    Segment    x-left    y-left    x-right   y-right    Soil Unit
       No.      (ft)      (ft)      (ft)      (ft)    Below Segment

       1          .0      49.0      23.4      49.2          1
       2        23.4      49.2      35.4      45.2          1
       3        35.4      45.2      63.4      45.2          1
       4        63.4      45.2      72.5      49.2          1
       5        72.5      49.2     165.8      80.0          1
       6       165.8      80.0     179.3      80.0          1
       7       179.3      80.0     187.3      78.0          1
       8       187.3      78.0     192.5      81.0          1
       9       192.5      81.0     205.3      81.0          1
      10       205.3      81.0     215.2      77.7          1


    --------------------------
    ISOTROPIC Soil Parameters
    --------------------------

     1 Soil unit(s) specified

    Soil   Unit Weight   Cohesion  Friction   Pore Pressure     Water
    Unit  Moist    Sat.  Intercept  Angle   Parameter Constant Surface
     No.  (pcf)   (pcf)    (psf)    (deg)       Ru      (psf)     No.

      1   108.7   130.0     718.0   11.40      .000        .0      1


     1 Water surface(s) have been specified

    Unit weight of water =     62.40  (pcf)


    Water Surface No.  1 specified by  2 coordinate points

    **********************************
             PHREATIC SURFACE,
    **********************************

      Point       x-water      y-water
       No.          (ft)         (ft)

        1          90.00        55.00
        2         215.00        55.00


    A critical failure surface searching method, using a random
    technique for generating CIRCULAR surfaces has been specified.


      100 trial surfaces will be generated and analyzed.


      10 Surfaces initiate from each of  10 points equally spaced
    along the ground surface between   x =      55.0 ft
                                     and  x =      85.0 ft


    Each surface terminates between    x =     160.0 ft
                                    and  x =     190.0 ft


    Unless further limitations were imposed, the minimum elevation
```

```
   at which a surface extends is  y =       .0 ft


   * * * * *  DEFAULT SEGMENT LENGTH SELECTED BY XSTABL  * * * * *

       4.0 ft line segments define each trial failure surface.


   ---------------------
   ANGULAR RESTRICTIONS
   ---------------------

     The first segment of each failure surface will be inclined
     within the angular range defined by :

             Lower angular limit :=   -45.0 degrees
             Upper angular limit :=  (slope angle - 5.0) degrees



***********************************************************************
    -- WARNING  --  WARNING  --  WARNING  --  WARNING  --  (# 48)
***********************************************************************
Negative effective stresses were calculated at the base of a slice.
This warning is usually reported for cases where slices have low self
weight and a relatively high "c" shear strength parameter.  In such
cases, this effect can only be eliminated by reducing the "c" value.
***********************************************************************

    -----------------------------------------------------------
    USER SELECTED option to maintain strength greater than zero
    -----------------------------------------------------------


    Factors of safety have been calculated by the :

    * * * * *   SIMPLIFIED BISHOP METHOD   * * * * *


      The most critical circular failure surface
      is specified by 41 coordinate points

           Point       x-surf       y-surf
            No.         (ft)         (ft)

             1          55.00        45.20
             2          57.97        42.53
             3          61.09        40.01
             4          64.32        37.66
             5          67.68        35.49
             6          71.15        33.49
             7          74.72        31.68
             8          78.37        30.06
             9          82.11        28.64
            10          85.92        27.41
            11          89.78        26.38
            12          93.70        25.56
            13          97.65        24.95
            14         101.63        24.54
            15         105.62        24.34
            16         109.62        24.36
            17         113.62        24.58
            18         117.59        25.01
            19         121.54        25.65
            20         125.45        26.50
            21         129.31        27.55
            22         133.11        28.80
            23         136.84        30.25
            24         140.49        31.89
            25         144.04        33.73
            26         147.50        35.74
            27         150.84        37.94
            28         154.06        40.31
            29         157.16        42.84
            30         160.12        45.53
            31         162.93        48.38
            32         165.59        51.37
            33         168.09        54.49
            34         170.42        57.74
            35         172.57        61.11
            36         174.55        64.59
            37         176.34        68.16
            38         177.94        71.83
            39         179.35        75.57
            40         180.56        79.38
            41         180.63        79.67

    ****  Simplified BISHOP FOS =    1.998  ****
```

```
The following is a summary of the TEN most critical surfaces

Problem Description : HARDIN CO LF INT PER SLOPE ANALYSIS

        FOS      Circle Center    Radius   Initial  Terminal   Resisting
      (BISHOP)  x-coord  y-coord            x-coord  x-coord     Moment
                  (ft)     (ft)      (ft)     (ft)     (ft)      (ft-lb)

  1.   1.998    107.38   100.48     76.16    55.00    180.63   1.366E+07
  2.   2.013    112.42   100.33     79.60    55.00    189.12   1.604E+07
  3.   2.030    113.24   102.43     79.31    58.33    188.97   1.515E+07
  4.   2.035    112.24   108.69     81.17    61.67    187.37   1.404E+07
  5.   2.043    114.05    98.10     76.83    58.33    188.35   1.508E+07
  6.   2.047    114.32    98.44     77.26    58.33    189.09   1.527E+07
  7.   2.059    109.98    97.43     71.15    61.67    178.94   1.188E+07
  8.   2.072    112.27    93.84     70.19    61.67    180.99   1.247E+07
  9.   2.075    102.08   107.86     76.42    58.33    173.21   1.118E+07
 10.   2.102    103.13   108.43     75.62    61.67    173.18   1.061E+07

                    * * *  END OF FILE  * * *
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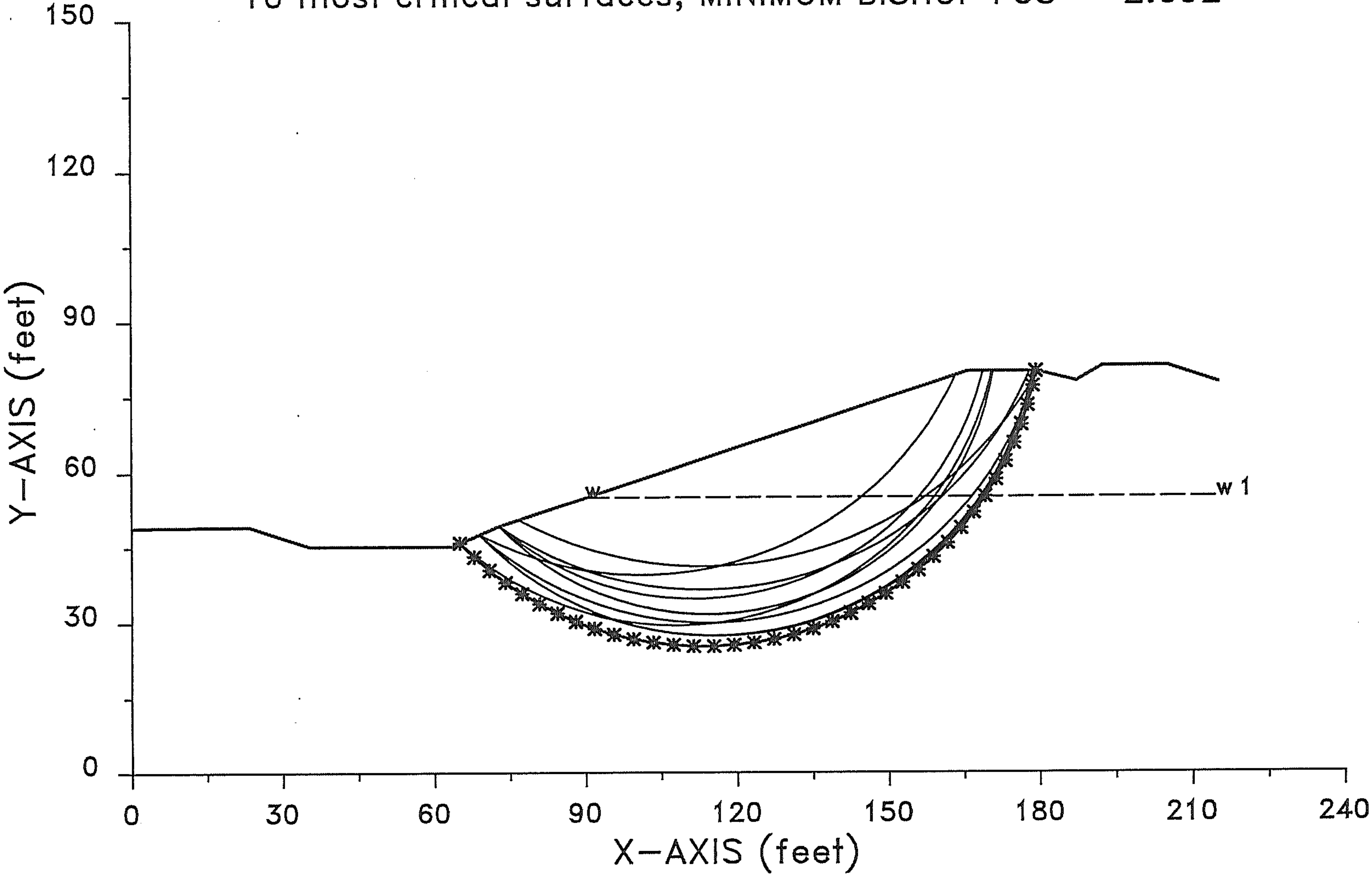
EX-2-1-E 2-03-17 16:20

HARDIN CO LF INT PER SLOPE ANALYSIS
10 most critical surfaces, MINIMUM BISHOP FOS = 2.092



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             *                                        *
             *        Slope Stability Analysis        *
             *              using the                 *
             *           Method of Slices             *
             *                                        *
             *        Copyright (C) 1992 - 2008       *
             *   Interactive Software Designs, Inc.   *
             *        Moscow, ID 83843, U.S.A.        *
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             *          All Rights Reserved           *
             *                                        *
             *  Ver. 5.208                96 - 2046   *
             ******************************************


    Problem Description : HARDIN CO LF INT PER SLOPE ANALYSIS



    -----------------------------
    SEGMENT BOUNDARY COORDINATES
    -----------------------------

       10 SURFACE boundary segments

    Segment    x-left    y-left    x-right   y-right     Soil Unit
       No.      (ft)      (ft)      (ft)      (ft)     Below Segment

       1         .0      49.0      23.4      49.2          1
       2       23.4      49.2      35.4      45.2          1
       3       35.4      45.2      63.4      45.2          1
       4       63.4      45.2      72.5      49.2          1
       5       72.5      49.2     165.8      80.0          1
       6      165.8      80.0     179.3      80.0          1
       7      179.3      80.0     187.3      78.0          1
       8      187.3      78.0     192.5      81.0          1
       9      192.5      81.0     205.3      81.0          1
      10      205.3      81.0     215.2      77.7          1

    --------------------------
    ISOTROPIC Soil Parameters
    --------------------------

     1 Soil unit(s) specified

    Soil   Unit Weight   Cohesion  Friction    Pore Pressure      Water
    Unit  Moist    Sat.  Intercept   Angle  Parameter  Constant  Surface
     No.  (pcf)   (pcf)    (psf)     (deg)      Ru       (psf)      No.

      1   108.7   130.0     644.0    12.60      .000         .0      1


     1 Water surface(s) have been specified

    Unit weight of water =    62.40  (pcf)


    Water Surface No.  1 specified by  2 coordinate points

    **********************************
             PHREATIC SURFACE,
    **********************************

      Point       x-water      y-water
       No.          (ft)         (ft)

        1          90.00        55.00
        2         215.00        55.00


    A critical failure surface searching method, using a random
    technique for generating CIRCULAR surfaces has been specified.


      100 trial surfaces will be generated and analyzed.


      10 Surfaces initiate from each of  10 points equally spaced
    along the ground surface between  x =      65.0 ft
                                    and  x =     100.0 ft


    Each surface terminates between   x =     150.0 ft
                                    and  x =     180.0 ft


    Unless further limitations were imposed, the minimum elevation
```

```
    at which a surface extends is  y =      .0 ft


    * * * * *  DEFAULT SEGMENT LENGTH SELECTED BY XSTABL  * * * * *

        4.0 ft line segments define each trial failure surface.


    ---------------------
    ANGULAR RESTRICTIONS
    ---------------------

      The first segment of each failure surface will be inclined
      within the angular range defined by :

              Lower angular limit :=   -45.0 degrees
              Upper angular limit :=  (slope angle - 5.0) degrees



************************************************************************
    -- WARNING -- WARNING -- WARNING -- WARNING -- (# 48)
************************************************************************
Negative effective stresses were calculated at the base of a slice.
This warning is usually reported for cases where slices have low self
weight and a relatively high "c" shear strength parameter.  In such
cases, this effect can only be eliminated by reducing the "c" value.
************************************************************************

    ------------------------------------------------------------
    USER SELECTED option to maintain strength greater than zero
    ------------------------------------------------------------



    Factors of safety have been calculated by the :

    * * * * *   SIMPLIFIED BISHOP METHOD   * * * * *



      The most critical circular failure surface
      is specified by 38 coordinate points

           Point      x-surf      y-surf
            No.        (ft)        (ft)

             1         65.00       45.90
             2         67.85       43.10
             3         70.86       40.46
             4         74.03       38.02
             5         77.33       35.76
             6         80.76       33.71
             7         84.31       31.87
             8         87.97       30.24
             9         91.71       28.83
            10         95.53       27.65
            11         99.42       26.69
            12        103.35       25.98
            13        107.32       25.49
            14        111.31       25.25
            15        115.31       25.24
            16        119.31       25.48
            17        123.28       25.95
            18        127.22       26.65
            19        131.10       27.59
            20        134.93       28.77
            21        138.68       30.16
            22        142.34       31.78
            23        145.89       33.62
            24        149.33       35.66
            25        152.64       37.90
            26        155.81       40.34
            27        158.83       42.96
            28        161.69       45.76
            29        164.38       48.73
            30        166.88       51.85
            31        169.19       55.11
            32        171.31       58.50
            33        173.22       62.02
            34        174.91       65.64
            35        176.39       69.36
            36        177.64       73.16
            37        178.66       77.03
            38        179.26       80.00

    ****  Simplified BISHOP FOS =   2.092  ****
```

```
The following is a summary of the TEN most critical surfaces

Problem Description : HARDIN CO LF INT PER SLOPE ANALYSIS

        FOS     Circle Center    Radius   Initial Terminal  Resisting
      (BISHOP)  x-coord  y-coord          x-coord  x-coord    Moment
                  (ft)     (ft)     (ft)     (ft)     (ft)     (ft-lb)

 1.    2.092    113.41    92.21    66.99    65.00   179.26  1.107E+07
 2.    2.104    107.87    93.62    64.15    65.00   170.52  9.041E+06
 3.    2.149    114.26    97.12    67.15    68.89   179.17  1.000E+07
 4.    2.184    115.84    91.85    64.50    68.89   179.21  1.014E+07
 5.    2.186    115.58    91.48    64.06    68.89   178.57  1.001E+07
 6.    2.230    113.17   106.62    70.13    72.78   178.02  8.929E+06
 7.    2.319    111.58    93.37    58.72    72.78   168.73  7.138E+06
 8.    2.325    114.18    88.89    57.29    72.78   170.76  7.608E+06
 9.    2.347    113.39   117.24    76.12    76.67   179.69  8.792E+06
10.    2.362    101.25   107.91    68.44    68.89   163.36  7.032E+06

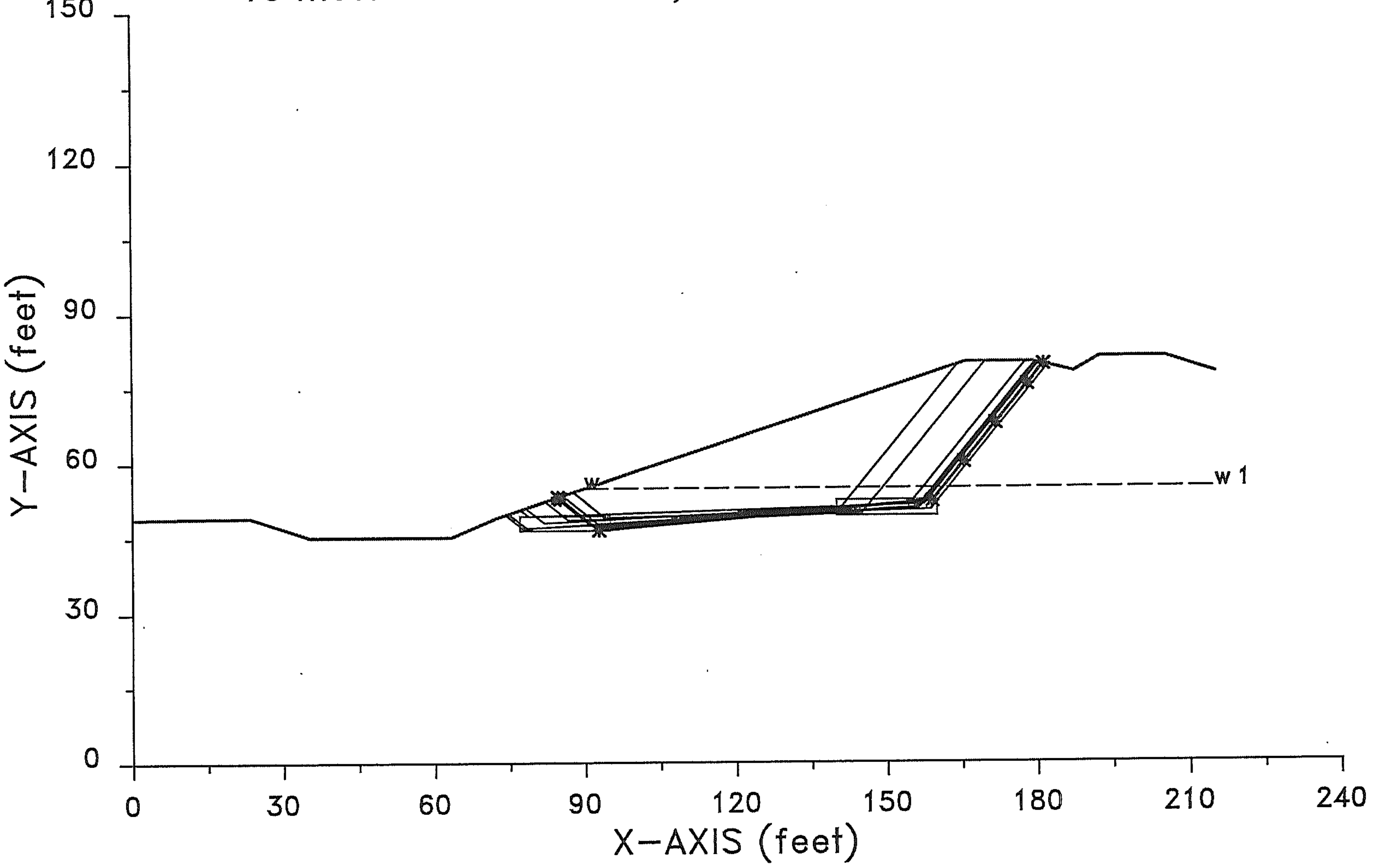
                     * * *  END OF FILE  * * *
```

SECTION E1-2 (FIGURE IIIE-A-3)

EXCAVATED SLOPE STABILITY ANALYSIS
RANKINE BLOCK METHOD
TOTAL AND EFFECTIVE STRESS MODELS

EX-2-2BT 2-06-17 9:16

HARDIN CO LF INT PER SLOPE ANAYLSIS
10 most critical surfaces, MINIMUM JANBU FOS = 2.946



```
XSTABL File: EX-2-2BT   2-06-17    9:16

          ******************************************
          *             X S T A B L                *
          *                                        *
          *        Slope Stability Analysis        *
          *              using the                 *
          *           Method of Slices             *
          *                                        *
          *        Copyright (C) 1992 - 2008       *
          *   Interactive Software Designs, Inc.   *
          *        Moscow, ID 83843, U.S.A.        *
          *                                        *
          *          All Rights Reserved           *
          *                                        *
          *  Ver. 5.208                96 - 2046   *
          ******************************************


    Problem Description : HARDIN CO LF INT PER SLOPE ANAYLSIS



    -----------------------------
    SEGMENT BOUNDARY COORDINATES
    -----------------------------

       10 SURFACE boundary segments

    Segment    x-left    y-left    x-right   y-right    Soil Unit
      No.       (ft)      (ft)       (ft)      (ft)    Below Segment

       1          .0      49.0       23.4      49.2          1
       2        23.4      49.2       35.4      45.2          1
       3        35.4      45.2       63.4      45.2          1
       4        63.4      45.2       72.5      49.2          1
       5        72.5      49.2      165.8      80.0          1
       6       165.8      80.0      179.3      80.0          1
       7       179.3      80.0      187.3      78.0          1
       8       187.3      78.0      192.5      81.0          1
       9       192.5      81.0      205.3      81.0          1
      10       205.3      81.0      215.2      77.7          1


    --------------------------
    ISOTROPIC Soil Parameters
    --------------------------

     1 Soil unit(s) specified

    Soil   Unit Weight    Cohesion  Friction    Pore Pressure      Water
    Unit  Moist   Sat.   Intercept   Angle   Parameter Constant  Surface
     No.  (pcf)   (pcf)    (psf)     (deg)       Ru      (psf)      No.

      1   108.7   130.0    718.0     11.40      .000          .0     1


     1 Water surface(s) have been specified

    Unit weight of water =    62.40  (pcf)
```

```
Water Surface No.  1 specified by  2 coordinate points

**********************************
         PHREATIC SURFACE,
**********************************

  Point        x-water      y-water
   No.          (ft)         (ft)

    1           90.00        55.00
    2          215.20        55.00



A critical failure surface searching method, using a random
technique for generating sliding BLOCK surfaces, has been
specified.


The active and passive portions of the sliding surfaces
are generated according to the Rankine theory.


   10 trial surfaces will be generated and analyzed.


  2 boxes specified for generation of central block base


Length of line segments for active and passive portions of
sliding block is     10.0 ft


Box          x-left     y-left    x-right    y-right      Width
no.           (ft)       (ft)       (ft)       (ft)        (ft)

 1            77.0       48.0       95.0       48.0         3.0
 2           140.0       51.0      160.0       51.0         3.0

************************************************************************
  --  WARNING  --  WARNING  --  WARNING  --  WARNING  --  (# 48)
************************************************************************
Negative effective stresses were calculated at the base of a slice.
This warning is usually reported for cases where slices have low self
weight and a relatively high "c" shear strength parameter.  In such
cases, this effect can only be eliminated by reducing the "c" value.
************************************************************************

    ------------------------------------------------------------
    USER SELECTED option to maintain strength greater than zero
    ------------------------------------------------------------



    Factors of safety have been calculated by the :

    * * * * *   SIMPLIFIED JANBU METHOD    * * * * *



    The 10 most critical of all the failure surfaces examined
```

```
are displayed below - the most critical first

Failure surface No. 1 specified by  8 coordinate points

      Point       x-surf       y-surf
       No.         (ft)         (ft)

        1          84.67        53.22
        2          85.09        52.87
        3          92.83        46.54
        4         159.07        52.31
        5         165.40        60.05
        6         171.73        67.79
        7         178.07        75.53
        8         181.32        79.50

 **  Corrected JANBU FOS =   2.946  **    (Fo factor = 1.073)

Failure surface No. 2 specified by  7 coordinate points

      Point       x-surf       y-surf
       No.         (ft)         (ft)

        1          84.41        53.13
        2          91.31        47.48
        3         156.60        51.13
        4         162.93        58.87
        5         169.27        66.61
        6         175.60        74.35
        7         180.07        79.81

 **  Corrected JANBU FOS =   2.958  **    (Fo factor = 1.075)

Failure surface No. 3 specified by  7 coordinate points

      Point       x-surf       y-surf
       No.         (ft)         (ft)

        1          84.82        53.27
        2          92.41        47.06
        3         157.56        52.01
        4         163.89        59.75
        5         170.22        67.48
        6         176.56        75.22
        7         180.27        79.76

 **  Corrected JANBU FOS =   2.965  **    (Fo factor = 1.074)

Failure surface No. 4 specified by  7 coordinate points

      Point       x-surf       y-surf
       No.         (ft)         (ft)

        1          74.79        49.96
        2          78.49        46.93
        3         154.65        51.76
        4         160.98        59.50
        5         167.31        67.24
        6         173.65        74.98
        7         177.76        80.00
```

```
**  Corrected JANBU FOS =   2.988  **    (Fo factor = 1.070)

Failure surface No. 5 specified by  7 coordinate points

       Point      x-surf      y-surf
        No.        (ft)        (ft)

         1         82.12       52.38
         2         86.69       48.64
         3        156.16       50.67
         4        162.50       58.41
         5        168.83       66.15
         6        175.16       73.89
         7        180.02       79.82

**  Corrected JANBU FOS =   3.009  **    (Fo factor = 1.075)

Failure surface No. 6 specified by  7 coordinate points

       Point      x-surf      y-surf
        No.        (ft)        (ft)

         1         78.24       51.10
         2         81.97       48.05
         3        156.81       51.96
         4        163.14       59.70
         5        169.48       67.44
         6        175.81       75.17
         7        179.68       79.90

**  Corrected JANBU FOS =   3.039  **    (Fo factor = 1.071)

Failure surface No. 7 specified by  7 coordinate points

       Point      x-surf      y-surf
        No.        (ft)        (ft)

         1         87.74       54.23
         2         93.99       49.12
         3        158.80       50.65
         4        165.14       58.39
         5        171.47       66.13
         6        177.80       73.87
         7        182.22       79.27

**  Corrected JANBU FOS =   3.062  **    (Fo factor = 1.076)

Failure surface No. 8 specified by  7 coordinate points

       Point      x-surf      y-surf
        No.        (ft)        (ft)

         1         85.62       53.53
         2         92.53       47.88
         3        145.13       49.95
         4        151.47       57.69
         5        157.80       65.43
         6        164.13       73.17
```

```
      7         169.73        80.00

  **  Corrected JANBU FOS =   3.105  **     (Fo factor = 1.080)


 Failure surface No. 9 specified by  7 coordinate points

       Point       x-surf       y-surf
        No.         (ft)         (ft)

         1          77.21        50.75
         2          78.84        49.41
         3         157.38        51.60
         4         163.71        59.34
         5         170.04        67.07
         6         176.38        74.81
         7         180.40        79.73

  **  Corrected JANBU FOS =   3.132  **     (Fo factor = 1.071)


 Failure surface No.10 specified by  7 coordinate points

       Point       x-surf       y-surf
        No.         (ft)         (ft)

         1          74.17        49.75
         2          77.60        46.94
         3         140.18        50.08
         4         146.52        57.82
         5         152.85        65.56
         6         159.19        73.30
         7         164.26        79.49

  **  Corrected JANBU FOS =   3.258  **     (Fo factor = 1.075)



 The following is a summary of the TEN most critical surfaces

 Problem Description : HARDIN CO LF INT PER SLOPE ANAYLSIS

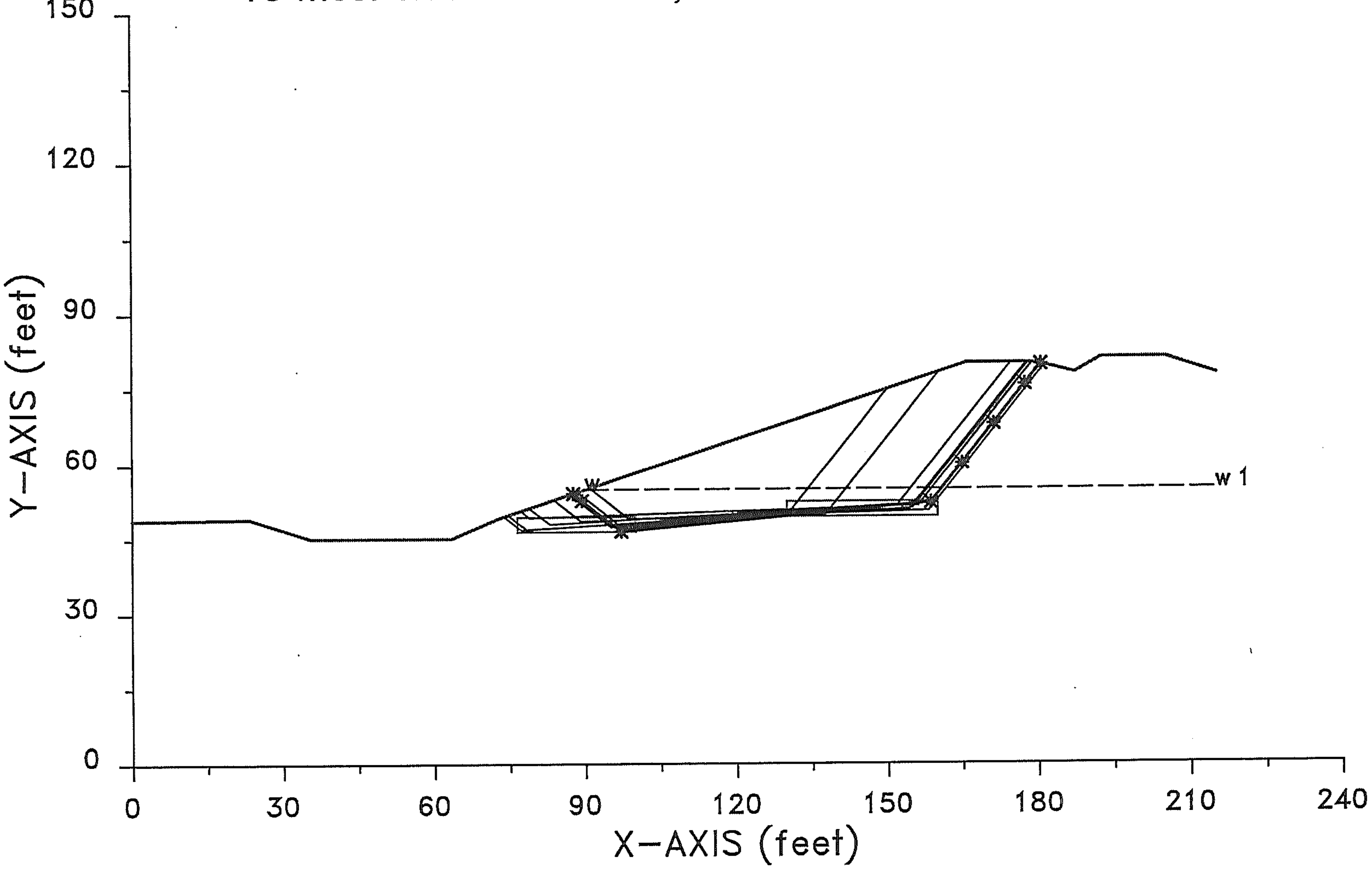
       Modified      Correction    Initial     Terminal    Available
      JANBU FOS        Factor      x-coord      x-coord     Strength
                                     (ft)         (ft)        (lb)

  1.     2.946         1.073        84.67       181.32     1.114E+05
  2.     2.958         1.075        84.41       180.07     1.101E+05
  3.     2.965         1.074        84.82       180.27     1.096E+05
  4.     2.988         1.070        74.79       177.76     1.133E+05
  5.     3.009         1.075        82.12       180.02     1.111E+05
  6.     3.039         1.071        78.24       179.68     1.125E+05
  7.     3.062         1.076        87.74       182.22     1.099E+05
  8.     3.105         1.080        85.62       169.73     9.550E+04
  9.     3.132         1.071        77.21       180.40     1.133E+05
 10.     3.258         1.075        74.17       164.26     9.662E+04


                       * * *  END OF FILE  * * *
```

EX-2-2BE 2-03-17 16:25

HARDIN CO LF INT PER SLOPE ANALYSIS
10 most critical surfaces, MINIMUM JANBU FOS = 2.840



```
XSTABL File: EX-2-2BE   2-03-17   16:25

           ******************************************
           *             X S T A B L                *
           *                                        *
           *        Slope Stability Analysis        *
           *               using the                *
           *            Method of Slices            *
           *                                        *
           *        Copyright (C) 1992 - 2008       *
           *   Interactive Software Designs, Inc.   *
           *        Moscow, ID 83843, U.S.A.        *
           *                                        *
           *          All Rights Reserved           *
           *                                        *
           *  Ver. 5.208                 96 - 2046  *
           ******************************************


    Problem Description : HARDIN CO LF INT PER SLOPE ANALYSIS



    -----------------------------
    SEGMENT BOUNDARY COORDINATES
    -----------------------------

       10 SURFACE boundary segments

    Segment     x-left    y-left     x-right    y-right     Soil Unit
      No.        (ft)      (ft)        (ft)       (ft)     Below Segment

       1           .0      49.0        23.4       49.2           1
       2         23.4      49.2        35.4       45.2           1
       3         35.4      45.2        63.4       45.2           1
       4         63.4      45.2        72.5       49.2           1
       5         72.5      49.2       165.8       80.0           1
       6        165.8      80.0       179.3       80.0           1
       7        179.3      80.0       187.3       78.0           1
       8        187.3      78.0       192.5       81.0           1
       9        192.5      81.0       205.3       81.0           1
      10        205.3      81.0       215.2       77.7           1


    --------------------------
    ISOTROPIC Soil Parameters
    --------------------------

     1 Soil unit(s) specified

    Soil   Unit Weight    Cohesion  Friction    Pore Pressure      Water
    Unit  Moist    Sat.  Intercept   Angle   Parameter Constant  Surface
     No.  (pcf)   (pcf)    (psf)     (deg)       Ru      (psf)      No.

      1   108.7   130.0    644.0     12.60      .000         .0      1


     1 Water surface(s) have been specified

    Unit weight of water =    62.40  (pcf)
```

```
Water Surface No.  1 specified by  2 coordinate points

**********************************
         PHREATIC SURFACE,
**********************************

  Point       x-water      y-water
   No.         (ft)         (ft)

    1          90.00        55.00
    2         215.00        55.00


A critical failure surface searching method, using a random
technique for generating sliding BLOCK surfaces, has been
specified.


The active and passive portions of the sliding surfaces
are generated according to the Rankine theory.


   10 trial surfaces will be generated and analyzed.


  2 boxes specified for generation of central block base


Length of line segments for active and passive portions of
sliding block is     10.0 ft


Box         x-left     y-left    x-right    y-right       Width
no.          (ft)       (ft)       (ft)       (ft)         (ft)

 1           76.5       48.0      100.0       48.0          3.0
 2          130.0       51.0      160.0       51.0          3.0


Factors of safety have been calculated by the :

* * * * *   SIMPLIFIED JANBU METHOD   * * * * *


The 10 most critical of all the failure surfaces examined
are displayed below - the most critical first

Failure surface No. 1 specified by  8 coordinate points

       Point       x-surf       y-surf
        No.         (ft)         (ft)

         1          87.62        54.19
         2          89.36        52.79
         3          97.17        46.54
         4         158.60        52.31
         5         164.85        60.12
         6         171.10        67.92
         7         177.36        75.72
```

```
         8         180.54        79.69

  **   Corrected JANBU FOS =    2.840   **     (Fo factor = 1.075)


 Failure surface No. 2 specified by  8 coordinate points

       Point       x-surf        y-surf
        No.         (ft)          (ft)

         1          87.04         54.00
         2          87.38         53.73
         3          95.19         47.48
         4         154.90         51.13
         5         161.15         58.94
         6         167.41         66.74
         7         173.66         74.55
         8         178.03         80.00

  **   Corrected JANBU FOS =    2.851   **     (Fo factor = 1.077)


 Failure surface No. 3 specified by  8 coordinate points

       Point       x-surf        y-surf
        No.         (ft)          (ft)

         1          87.68         54.21
         2          88.81         53.31
         3          96.61         47.06
         4         156.34         52.01
         5         162.59         59.81
         6         168.84         67.62
         7         175.09         75.42
         8         178.76         80.00

  **   Corrected JANBU FOS =    2.857   **     (Fo factor = 1.076)


 Failure surface No. 4 specified by  7 coordinate points

       Point       x-surf        y-surf
        No.         (ft)          (ft)

         1          74.70         49.93
         2          78.44         46.93
         3         151.97         51.76
         4         158.22         59.57
         5         164.47         67.37
         6         170.73         75.18
         7         174.59         80.00

  **   Corrected JANBU FOS =    2.862   **     (Fo factor = 1.071)


 Failure surface No. 5 specified by  7 coordinate points

       Point       x-surf        y-surf
        No.         (ft)          (ft)

         1          83.79         52.93
         2          89.15         48.64
         3         154.25         50.67
```

```
          4          160.50          58.47
          5          166.75          66.28
          6          173.00          74.08
          7          177.74          80.00

  **  Corrected JANBU FOS =   2.872  **     (Fo factor = 1.076)


 Failure surface No. 6 specified by  7 coordinate points

        Point        x-surf         y-surf
         No.          (ft)           (ft)

          1           78.91          51.31
          2           82.98          48.05
          3          155.21          51.96
          4          161.47          59.76
          5          167.72          67.57
          6          173.97          75.37
          7          177.68          80.00

  **  Corrected JANBU FOS =   2.893  **     (Fo factor = 1.072)


 Failure surface No. 7 specified by  7 coordinate points

        Point        x-surf         y-surf
         No.          (ft)           (ft)

          1           90.97          55.30
          2           98.69          49.12
          3          158.20          50.65
          4          164.46          58.46
          5          170.71          66.26
          6          176.96          74.07
          7          181.31          79.50

  **  Corrected JANBU FOS =   2.947  **     (Fo factor = 1.079)


 Failure surface No. 8 specified by  7 coordinate points

        Point        x-surf         y-surf
         No.          (ft)           (ft)

          1           77.23          50.76
          2           78.91          49.41
          3          156.07          51.60
          4          162.32          59.40
          5          168.57          67.21
          6          174.82          75.01
          7          178.82          80.00

  **  Corrected JANBU FOS =   2.986  **     (Fo factor = 1.072)


 Failure surface No. 9 specified by  8 coordinate points

        Point        x-surf         y-surf
         No.          (ft)           (ft)

          1           88.52          54.49
          2           88.96          54.14
```

```
          3          96.77        47.88
          4         137.70        49.95
          5         143.95        57.76
          6         150.21        65.56
          7         156.46        73.36
          8         160.33        78.19

  **  Corrected JANBU FOS =   3.277  **    (Fo factor = 1.083)


 Failure surface No.10 specified by  7 coordinate points

        Point      x-surf       y-surf
         No.        (ft)         (ft)

          1          73.89        49.66
          2          77.29        46.94
          3         130.28        50.08
          4         136.53        57.89
          5         142.78        65.69
          6         149.03        73.49
          7         150.09        74.81

  **  Corrected JANBU FOS =   3.437  **    (Fo factor = 1.075)




 The following is a summary of the TEN most critical surfaces

 Problem Description : HARDIN CO LF INT PER SLOPE ANALYSIS

        Modified     Correction    Initial     Terminal    Available
       JANBU FOS       Factor      x-coord      x-coord     Strength
                                    (ft)         (ft)         (lb)

  1.     2.840          1.075        87.62       180.54     1.037E+05
  2.     2.851          1.077        87.04       178.03     1.008E+05
  3.     2.857          1.076        87.68       178.76     1.010E+05
  4.     2.862          1.071        74.70       174.59     1.038E+05
  5.     2.872          1.076        83.79       177.74     1.020E+05
  6.     2.893          1.072        78.91       177.68     1.043E+05
  7.     2.947          1.079        90.97       181.31     1.021E+05
  8.     2.986          1.072        77.23       178.82     1.060E+05
  9.     3.277          1.083        88.52       160.33     7.664E+04
 10.     3.437          1.075        73.89       150.09     7.420E+04

                        * * *  END OF FILE  * * *
```

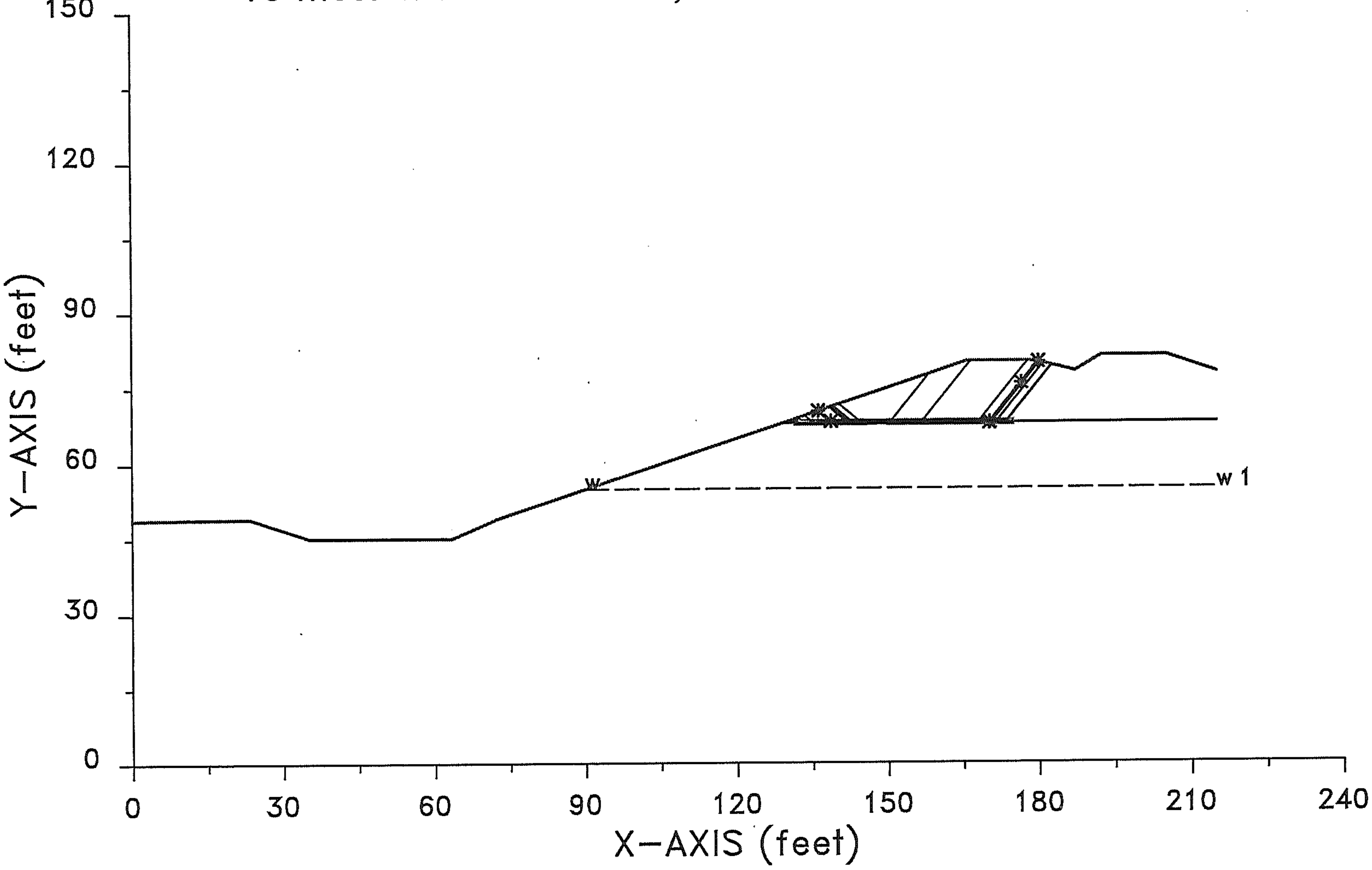
SECTION E1-3 (FIGURE IIIE-A-3)

EXCAVATED SLOPE STABILITY ANALYSIS
RANKINE BLOCK METHOD
TOTAL AND EFFECTIVE STRESS MODELS

Note: Section E1-3 includes a clay slickensided seam through which the analysis failure plane was projected.

EX-2-3BT 2-06-17 9:29

HARDIN CO LF INT PER SLOPE ANAYLSIS
10 most critical surfaces, MINIMUM JANBU FOS = 6.091



```
XSTABL File: EX-2-3BT   2-06-17    9:29

              ******************************************
              *               X S T A B L              *
              *                                        *
              *         Slope Stability Analysis       *
              *               using the                *
              *            Method of Slices            *
              *                                        *
              *         Copyright (C) 1992 - 2008      *
              *    Interactive Software Designs, Inc.  *
              *         Moscow, ID 83843, U.S.A.       *
              *                                        *
              *           All Rights Reserved          *
              *                                        *
              *  Ver. 5.208                96 - 2046   *
              ******************************************


    Problem Description : HARDIN CO LF INT PER SLOPE ANAYLSIS



    -----------------------------
    SEGMENT BOUNDARY COORDINATES
    -----------------------------

       10 SURFACE boundary segments

    Segment    x-left    y-left    x-right    y-right    Soil Unit
      No.       (ft)      (ft)       (ft)       (ft)    Below Segment

       1          .0      49.0       23.4       49.2         1
       2        23.4      49.2       35.4       45.2         1
       3        35.4      45.2       63.4       45.2         1
       4        63.4      45.2       72.5       49.2         1
       5        72.5      49.2      165.8       80.0         1
       6       165.8      80.0      179.3       80.0         1
       7       179.3      80.0      187.3       78.0         1
       8       187.3      78.0      192.5       81.0         1
       9       192.5      81.0      205.3       81.0         1
      10       205.3      81.0      215.2       77.7         1

        2 SUBSURFACE boundary segments

    Segment    x-left    y-left    x-right    y-right    Soil Unit
      No.       (ft)      (ft)       (ft)       (ft)    Below Segment

       1       129.4      68.0      215.2       68.0         2
       2       129.1      67.9      215.2       67.9         1


    --------------------------
    ISOTROPIC Soil Parameters
    --------------------------

     2 Soil unit(s) specified

    Soil   Unit Weight   Cohesion  Friction    Pore Pressure      Water
    Unit  Moist    Sat.  Intercept  Angle   Parameter  Constant  Surface
     No.  (pcf)   (pcf)    (psf)    (deg)      Ru        (psf)      No.
```

```
   1   108.7   130.0      718.0     11.40       .000          .0      0
   2   108.7   130.0         .0       .00       .000          .0      0


  1 Water surface(s) have been specified

 Unit weight of water =    62.40  (pcf)


 Water Surface No.  1 specified by  2 coordinate points

 **********************************
          PHREATIC SURFACE,
 **********************************

   Point       x-water     y-water
    No.          (ft)        (ft)

     1           90.00       55.00
     2          215.20       55.00

 -- WARNING -----------------------------------------------
      Water surface number 1 has been defined but is not
      used by any soil unit.  The analysis will IGNORE
      water surface # 1.  Please make sure that this
      assumption is consistent with your subsurface model.
 ----------------------------------------------------------



 A critical failure surface searching method, using a random
 technique for generating sliding BLOCK surfaces, has been
 specified.


 The active and passive portions of the sliding surfaces
 are generated according to the Rankine theory.


    10 trial surfaces will be generated and analyzed.


   2 boxes specified for generation of central block base


 Length of line segments for active and passive portions of
 sliding block is     10.0 ft


 Box         x-left      y-left     x-right     y-right       Width
 no.          (ft)        (ft)        (ft)        (ft)         (ft)

  1          131.5        68.0       145.0        68.0          1.0
  2          150.0        68.0       175.0        68.0          1.0


 ************************************************************
 **      Factor of safety calculation for surface #      2    **
 **      failed to converge within FIFTY iterations           **
 **                                                           **
 **    The last calculated value of the FOS was    7.4850     **
```

```
**    This will be ignored for final summary of results     **
************************************************************

The trial failure surface in question is
defined by the following  7 coordinate points

       Point      x-surf      y-surf
        No.        (ft)        (ft)

         1       140.32       71.59
         2       144.25       68.37
         3       173.50       67.88
         4       173.51       67.90
         5       173.61       68.00
         6       179.95       75.74
         7       182.73       79.14


************************************************************
**     Factor of safety calculation for surface #     4     **
**     failed to converge within FIFTY iterations           **
**                                                          **
**    The last calculated value of the FOS was    6.4245    **
**    This will be ignored for final summary of results     **
************************************************************

The trial failure surface in question is
defined by the following  7 coordinate points

       Point      x-surf      y-surf
        No.        (ft)        (ft)

         1       138.89       71.12
         2       142.69       68.00
         3       142.79       67.90
         4       143.05       67.69
         5       171.95       68.34
         6       178.28       76.07
         7       181.12       79.54


************************************************************
**     Factor of safety calculation for surface #     5     **
**     failed to converge within FIFTY iterations           **
**                                                          **
**    The last calculated value of the FOS was    4.6923    **
**    This will be ignored for final summary of results     **
************************************************************

The trial failure surface in question is
defined by the following  7 coordinate points

       Point      x-surf      y-surf
        No.        (ft)        (ft)

         1       138.42       70.96
         2       142.04       68.00
         3       142.14       67.90
         4       142.23       67.83
         5       170.75       68.04
         6       177.08       75.78
         7       180.33       79.74
```

```
*************************************************************
**     Factor of safety calculation for surface #     7    **
**     failed to converge within FIFTY iterations          **
**                                                         **
**    The last calculated value of the FOS was   7.0676    **
**    This will be ignored for final summary of results    **
*************************************************************

The trial failure surface in question is
defined by the following  8 coordinate points

       Point       x-surf       y-surf
        No.         (ft)         (ft)

         1         139.18        71.21
         2         143.11        68.00
         3         143.14        67.96
         4         156.42        67.65
         5         156.62        67.90
         6         156.72        68.00
         7         163.05        75.74
         8         166.54        80.00


*************************************************************
**     Factor of safety calculation for surface #     8    **
**     failed to converge within FIFTY iterations          **
**                                                         **
**    The last calculated value of the FOS was   7.6021    **
**    This will be ignored for final summary of results    **
*************************************************************

The trial failure surface in question is
defined by the following  7 coordinate points

       Point       x-surf       y-surf
        No.         (ft)         (ft)

         1         138.96        71.14
         2         142.80        68.00
         3         142.90        67.90
         4         143.37        67.51
         5         173.83        68.44
         6         180.17        76.18
         7         182.62        79.17


*************************************************************
**     Factor of safety calculation for surface #     9    **
**     failed to converge within FIFTY iterations          **
**                                                         **
**    The last calculated value of the FOS was  10.1759    **
**    This will be ignored for final summary of results    **
*************************************************************

The trial failure surface in question is
defined by the following  9 coordinate points

       Point       x-surf       y-surf
        No.         (ft)         (ft)

         1         130.94        68.49
```

```
          2          131.54        68.00
          3          131.64        67.90
          4          131.95        67.65
          5          150.23        67.69
          6          150.40        67.90
          7          150.50        68.00
          8          156.83        75.74
          9          158.29        77.52


Factors of safety have been calculated by the :

* * * * *   SIMPLIFIED JANBU METHOD   * * * * *


The 10 most critical of all the failure surfaces examined
are displayed below - the most critical first

Failure surface No. 1 specified by  7 coordinate points

       Point        x-surf        y-surf
        No.          (ft)          (ft)

          1          136.27        70.25
          2          138.76        68.21
          3          170.20        67.89
          4          170.21        67.90
          5          170.31        68.00
          6          176.65        75.74
          7          179.99        79.83

 **  Corrected JANBU FOS =    6.091  **     (Fo factor = 1.075)

Failure surface No. 2 specified by  7 coordinate points

       Point        x-surf        y-surf
        No.          (ft)          (ft)

          1          131.41        68.65
          2          132.20        68.00
          3          132.30        67.90
          4          132.62        67.64
          5          168.31        68.25
          6          174.64        75.99
          7          177.92        80.00

 **  Corrected JANBU FOS =    6.357  **     (Fo factor = 1.070)

Failure surface No. 3 specified by  5 coordinate points

       Point        x-surf        y-surf
        No.          (ft)          (ft)

          1          133.58        69.36
          2          135.22        68.02
          3          171.01        68.32
          4          177.34        76.06
          5          180.36        79.74
```

```
 **  Corrected JANBU FOS =   7.524  **    (Fo factor = 1.070)

Failure surface No. 4 specified by  5 coordinate points

       Point        x-surf       y-surf
        No.          (ft)         (ft)

         1          132.31        68.94
         2          132.88        68.47
         3          171.72        68.20
         4          178.06        75.94
         5          181.03        79.57

 **  Corrected JANBU FOS =   7.878  **    (Fo factor = 1.068)

Failure surface No. 5 specified by  7 coordinate points

       Point        x-surf       y-surf
        No.          (ft)         (ft)

         1          140.32        71.59
         2          144.25        68.37
         3          173.50        67.88
         4          173.51        67.90
         5          173.61        68.00
         6          179.95        75.74
         7          182.73        79.14

 **  Corrected JANBU FOS = 500.000  **    (Fo factor = 1.076)

Failure surface No. 6 specified by  7 coordinate points

       Point        x-surf       y-surf
        No.          (ft)         (ft)

         1          138.89        71.12
         2          142.69        68.00
         3          142.79        67.90
         4          143.05        67.69
         5          171.95        68.34
         6          178.28        76.07
         7          181.12        79.54

 **  Corrected JANBU FOS = 500.000  **    (Fo factor = 1.075)

Failure surface No. 7 specified by  7 coordinate points

       Point        x-surf       y-surf
        No.          (ft)         (ft)

         1          138.42        70.96
         2          142.04        68.00
         3          142.14        67.90
         4          142.23        67.83
         5          170.75        68.04
         6          177.08        75.78
         7          180.33        79.74

 **  Corrected JANBU FOS = 500.000  **    (Fo factor = 1.076)
```

```
Failure surface No. 8 specified by  8 coordinate points

       Point      x-surf      y-surf
        No.        (ft)        (ft)

         1        139.18       71.21
         2        143.11       68.00
         3        143.14       67.96
         4        156.42       67.65
         5        156.62       67.90
         6        156.72       68.00
         7        163.05       75.74
         8        166.54       80.00

 **  Corrected JANBU FOS = 500.000  **    (Fo factor = 1.087)

Failure surface No. 9 specified by  7 coordinate points

       Point      x-surf      y-surf
        No.        (ft)        (ft)

         1        138.96       71.14
         2        142.80       68.00
         3        142.90       67.90
         4        143.37       67.51
         5        173.83       68.44
         6        180.17       76.18
         7        182.62       79.17

 **  Corrected JANBU FOS = 500.000  **    (Fo factor = 1.072)

Failure surface No.10 specified by  9 coordinate points

       Point      x-surf      y-surf
        No.        (ft)        (ft)

         1        130.94       68.49
         2        131.54       68.00
         3        131.64       67.90
         4        131.95       67.65
         5        150.23       67.69
         6        150.40       67.90
         7        150.50       68.00
         8        156.83       75.74
         9        158.29       77.52

 **  Corrected JANBU FOS = 500.000  **    (Fo factor = 1.079)


**********************************************************************
**                                                                  **
**  Out of the     10 surfaces generated and analyzed by XSTABL,    **
**        6 surfaces were found to have MISLEADING FOS values.      **
**                                                                  **
**********************************************************************
```

```
The following is a summary of the TEN most critical surfaces

Problem Description : HARDIN CO LF INT PER SLOPE ANAYLSIS

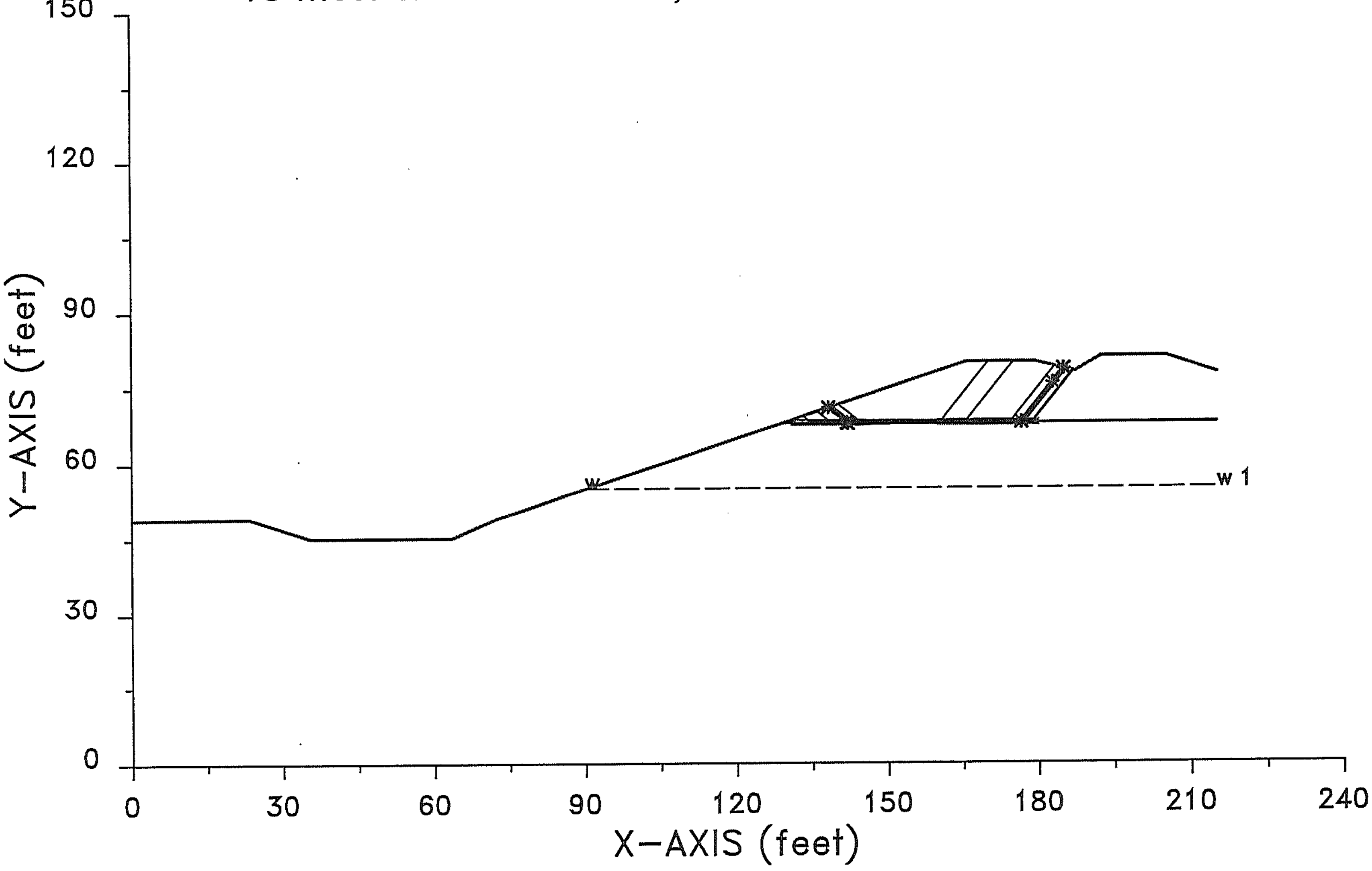
       Modified     Correction   Initial    Terminal    Available
      JANBU FOS      Factor      x-coord     x-coord     Strength
                                  (ft)        (ft)        (lb)

 1.      6.091        1.075      136.27      179.99     3.374E+04
 2.      6.357        1.070      131.41      177.92     3.939E+04
 3.      7.524        1.070      133.58      180.36     4.529E+04
 4.      7.878        1.068      132.31      181.03     4.656E+04
 5.    500.000        1.076      140.32      182.73     3.690E+04
 6.    500.000        1.075      138.89      181.12     3.848E+04
 7.    500.000        1.076      138.42      180.33     3.009E+04
 8.    500.000        1.087      139.18      166.54     2.584E+04
 9.    500.000        1.072      138.96      182.62     4.075E+04
10.    500.000        1.079      130.94      158.29     2.527E+04

                     * * *  END OF FILE  * * *
```

EX-2-3BE 2-08-17 8:34

HARDIN CO LF INT PER SLOPE ANALYSIS
10 most critical surfaces, MINIMUM JANBU FOS = 5.942



```
XSTABL File: EX-2-3BE   2-08-17    8:34

          ******************************************
          *               X S T A B L              *
          *                                        *
          *        Slope Stability Analysis        *
          *               using the                *
          *            Method of Slices            *
          *                                        *
          *        Copyright (C) 1992 - 2008       *
          *   Interactive Software Designs, Inc.   *
          *        Moscow, ID 83843, U.S.A.        *
          *                                        *
          *          All Rights Reserved           *
          *                                        *
          * Ver. 5.208                 96 - 2046   *
          ******************************************


     Problem Description : HARDIN CO LF INT PER SLOPE ANALYSIS



     -----------------------------
     SEGMENT BOUNDARY COORDINATES
     -----------------------------

        10 SURFACE boundary segments

     Segment     x-left     y-left     x-right     y-right     Soil Unit
       No.        (ft)       (ft)        (ft)        (ft)    Below Segment

        1           .0       49.0        23.4        49.2          1
        2         23.4       49.2        35.4        45.2          1
        3         35.4       45.2        63.4        45.2          1
        4         63.4       45.2        72.5        49.2          1
        5         72.5       49.2       165.8        80.0          1
        6        165.8       80.0       179.3        80.0          1
        7        179.3       80.0       187.3        78.0          1
        8        187.3       78.0       192.5        81.0          1
        9        192.5       81.0       205.3        81.0          1
       10        205.3       81.0       215.2        77.7          1

         2 SUBSURFACE boundary segments

     Segment     x-left     y-left     x-right     y-right     Soil Unit
       No.        (ft)       (ft)        (ft)        (ft)    Below Segment

        1        129.4       68.0       215.2        68.0          2
        2        129.1       67.9       215.2        67.9          1

     --------------------------
     ISOTROPIC Soil Parameters
     --------------------------

       2 Soil unit(s) specified

     Soil   Unit Weight   Cohesion  Friction    Pore Pressure      Water
     Unit  Moist   Sat.  Intercept   Angle   Parameter Constant   Surface
      No.  (pcf)  (pcf)    (psf)     (deg)       Ru      (psf)       No.
```

```
 1   108.7   130.0      644.0     12.60        .000          .0      1
 2   108.7   130.0         .0       .00        .000          .0      0


 1 Water surface(s) have been specified

Unit weight of water =     62.40  (pcf)


Water Surface No.  1 specified by  2 coordinate points

**********************************
         PHREATIC SURFACE,
**********************************

  Point        x-water      y-water
   No.           (ft)         (ft)

    1           90.00        55.00
    2          215.20        55.00


A critical failure surface searching method, using a random
technique for generating sliding BLOCK surfaces, has been
specified.


The active and passive portions of the sliding surfaces
are generated according to the Rankine theory.


   10 trial surfaces will be generated and analyzed.


  2 boxes specified for generation of central block base


Length of line segments for active and passive portions of
sliding block is     10.0 ft


Box         x-left      y-left    x-right     y-right       Width
no.          (ft)        (ft)       (ft)        (ft)         (ft)

 1          131.0        68.0      145.0        68.0          1.0
 2          160.0        68.0      180.0        68.0          1.0

*************************************************************
**      Factor of safety calculation for surface #     2    **
**      failed to converge within FIFTY iterations          **
**                                                          **
**     The last calculated value of the FOS was   9.0222    **
**     This will be ignored for final summary of results    **
*************************************************************

The trial failure surface in question is
defined by the following  7 coordinate points

       Point      x-surf      y-surf
```

```
         No.          (ft)          (ft)

          1          140.24         71.56
          2          144.22         68.37
          3          178.80         67.88
          4          178.81         67.90
          5          178.91         68.00
          6          185.17         75.80
          7          186.99         78.08


**************************************************************
**      Factor of safety calculation for surface #     7    **
**      failed to converge within FIFTY iterations          **
**                                                          **
**     The last calculated value of the FOS was   5.1400    **
**     This will be ignored for final summary of results    **
**************************************************************

The trial failure surface in question is
defined by the following  8 coordinate points

        Point       x-surf       y-surf
         No.         (ft)         (ft)

          1          139.07        71.18
          2          143.04        68.00
          3          143.08        67.96
          4          165.13        67.65
          5          165.33        67.90
          6          165.43        68.00
          7          171.69        75.80
          8          175.05        80.00


**************************************************************
**      Factor of safety calculation for surface #     8    **
**      failed to converge within FIFTY iterations          **
**                                                          **
**     The last calculated value of the FOS was   8.6839    **
**     This will be ignored for final summary of results    **
**************************************************************

The trial failure surface in question is
defined by the following  7 coordinate points

        Point       x-surf       y-surf
         No.         (ft)         (ft)

          1          138.85        71.10
          2          142.73        68.00
          3          142.83        67.90
          4          143.31        67.51
          5          179.07        68.44
          6          185.32        76.24
          7          186.82        78.12


Factors of safety have been calculated by the :

* * * * *   SIMPLIFIED JANBU METHOD   * * * * *
```

```
The 10 most critical of all the failure surfaces examined
are displayed below - the most critical first

Failure surface No. 1 specified by  7 coordinate points

       Point      x-surf      y-surf
        No.        (ft)        (ft)

         1        138.29       70.92
         2        141.94       68.00
         3        142.04       67.90
         4        142.13       67.83
         5        176.60       68.04
         6        182.85       75.85
         7        185.03       78.57

 **  Corrected JANBU FOS =   5.942  **    (Fo factor = 1.070)

Failure surface No. 2 specified by  9 coordinate points

       Point      x-surf      y-surf
        No.        (ft)        (ft)

         1        130.58       68.37
         2        131.05       68.00
         3        131.15       67.90
         4        131.47       67.65
         5        160.18       67.69
         6        160.35       67.90
         7        160.45       68.00
         8        166.70       75.80
         9        170.06       80.00

 **  Corrected JANBU FOS =   6.002  **    (Fo factor = 1.076)

Failure surface No. 3 specified by  7 coordinate points

       Point      x-surf      y-surf
        No.        (ft)        (ft)

         1        136.07       70.19
         2        138.53       68.21
         3        176.16       67.89
         4        176.17       67.90
         5        176.27       68.00
         6        182.52       75.80
         7        184.79       78.63

 **  Corrected JANBU FOS =   6.444  **    (Fo factor = 1.068)

Failure surface No. 4 specified by  7 coordinate points

       Point      x-surf      y-surf
        No.        (ft)        (ft)

         1        131.07       68.53
         2        131.74       68.00
         3        131.84       67.90
```

```
          4          132.16         67.64
          5          174.65         68.25
          6          180.90         76.06
          7          183.26         79.01

  **   Corrected JANBU FOS =    6.592   **     (Fo factor = 1.064)


 Failure surface No. 5 specified by  7 coordinate points

        Point        x-surf        y-surf
         No.          (ft)          (ft)

          1          138.77         71.08
          2          142.61         68.00
          3          142.71         67.90
          4          142.98         67.69
          5          177.56         68.34
          6          183.81         76.14
          7          185.63         78.42

  **   Corrected JANBU FOS =    7.751   **     (Fo factor = 1.068)


 Failure surface No. 6 specified by  5 coordinate points

        Point        x-surf        y-surf
         No.          (ft)          (ft)

          1          133.30         69.27
          2          134.86         68.02
          3          176.81         68.32
          4          183.06         76.12
          5          185.02         78.57

  **   Corrected JANBU FOS =    8.176   **     (Fo factor = 1.063)


 Failure surface No. 7 specified by  5 coordinate points

        Point        x-surf        y-surf
         No.          (ft)          (ft)

          1          131.98         68.84
          2          132.43         68.47
          3          177.38         68.20
          4          183.63         76.00
          5          185.57         78.43

  **   Corrected JANBU FOS =    8.739   **     (Fo factor = 1.062)


 Failure surface No. 8 specified by  7 coordinate points

        Point        x-surf        y-surf
         No.          (ft)          (ft)

          1          140.24         71.56
          2          144.22         68.37
          3          178.80         67.88
          4          178.81         67.90
          5          178.91         68.00
          6          185.17         75.80
```

```
      7         186.99        78.08

 **  Corrected JANBU FOS = 500.000  **     (Fo factor = 1.070)


Failure surface No. 9 specified by  8 coordinate points

     Point      x-surf      y-surf
      No.        (ft)        (ft)

      1         139.07        71.18
      2         143.04        68.00
      3         143.08        67.96
      4         165.13        67.65
      5         165.33        67.90
      6         165.43        68.00
      7         171.69        75.80
      8         175.05        80.00

 **  Corrected JANBU FOS = 500.000  **     (Fo factor = 1.083)


Failure surface No.10 specified by  7 coordinate points

     Point      x-surf      y-surf
      No.        (ft)        (ft)

      1         138.85        71.10
      2         142.73        68.00
      3         142.83        67.90
      4         143.31        67.51
      5         179.07        68.44
      6         185.32        76.24
      7         186.82        78.12

 **  Corrected JANBU FOS = 500.000  **     (Fo factor = 1.066)


*******************************************************************
**                                                               **
**  Out of the     10 surfaces generated and analyzed by XSTABL, **
**        3 surfaces were found to have MISLEADING FOS values.   **
**                                                               **
*******************************************************************



The following is a summary of the TEN most critical surfaces

Problem Description : HARDIN CO LF INT PER SLOPE ANALYSIS

       Modified     Correction   Initial    Terminal    Available
      JANBU FOS       Factor     x-coord     x-coord     Strength
                                   (ft)        (ft)        (lb)

 1.     5.942         1.070      138.29      185.03     2.951E+04
 2.     6.002         1.076      130.58      170.06     3.479E+04
 3.     6.444         1.068      136.07      184.79     3.390E+04
 4.     6.592         1.064      131.07      183.26     4.075E+04
 5.     7.751         1.068      138.77      185.63     3.924E+04
```

```
  6.     8.176          1.063      133.30      185.02     4.662E+04
  7.     8.739          1.062      131.98      185.57     4.765E+04
  8.   500.000          1.070      140.24      186.99     3.720E+04
  9.   500.000          1.083      139.07      175.05     3.086E+04
 10.   500.000          1.066      138.85      186.82     4.134E+04

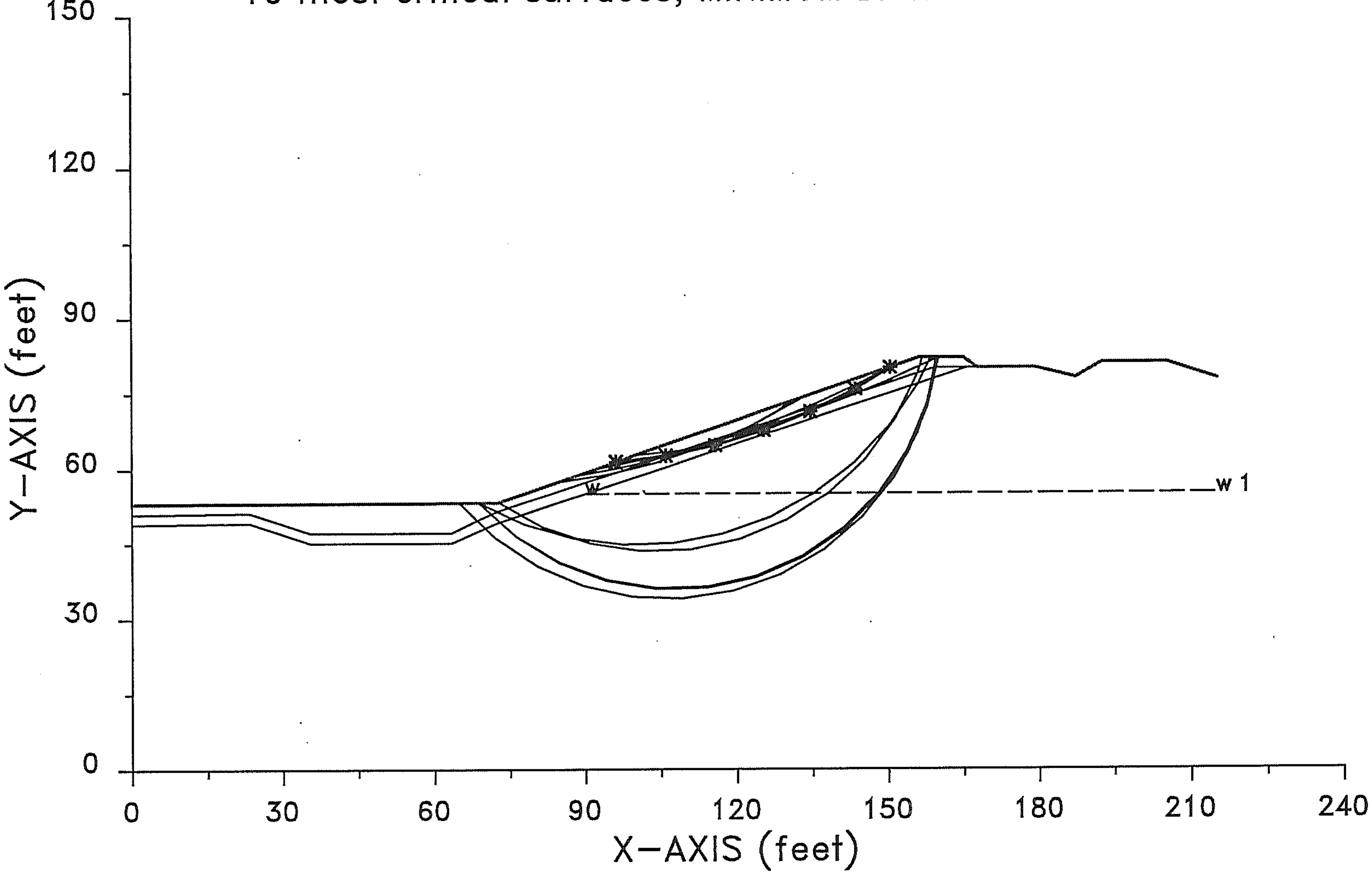
                   * * *  END OF FILE  * * *
```

SECTION E1-4 (FIGURE IIIE-A-3)

CONSTRUCTED LINER SLOPE STABILITY ANALYSIS
BISHOP CIRCULAR METHOD
TOTAL AND EFFECTIVE STRESS MODELS

EX-2-4-T 2-10-17 8:18

HARDIN CO LF INT PER SLOPE ANALYSIS
10 most critical surfaces, MINIMUM BISHOP FOS = 1.977



```
XSTABL File: EX-2-4-T   2-10-17    8:18


          *******************************************
          *              X S T A B L                *
          *                                         *
          *        Slope Stability Analysis         *
          *               using the                 *
          *            Method of Slices             *
          *                                         *
          *        Copyright (C) 1992 - 2008        *
          *   Interactive Software Designs, Inc.    *
          *        Moscow, ID 83843, U.S.A.         *
          *                                         *
          *          All Rights Reserved            *
          *                                         *
          *  Ver. 5.208                96 - 2046    *
          *******************************************


  Problem Description : HARDIN CO LF INT PER SLOPE ANALYSIS


  -----------------------------
  SEGMENT BOUNDARY COORDINATES
  -----------------------------

      9 SURFACE boundary segments

  Segment   x-left   y-left   x-right   y-right     Soil Unit
    No.      (ft)     (ft)     (ft)      (ft)    Below Segment

     1         .0     53.0      72.5      53.2          3
     2       72.5     53.2     156.1      82.0          3
     3      156.1     82.0     165.0      82.0          3
     4      165.0     82.0     167.8      80.0          3
     5      167.8     80.0     179.3      80.0          1
     6      179.3     80.0     187.3      78.0          1
```

```
        7      187.3      78.0      192.5      81.0          1

        8      192.5      81.0      205.3      81.0          1

        9      205.3      81.0      215.2      77.7          1



      14 SUBSURFACE boundary segments


   Segment    x-left    y-left    x-right   y-right      Soil Unit

     No.       (ft)      (ft)       (ft)      (ft)     Below Segment


      1          .0      51.0       23.4      51.2          2

      2        23.4      51.2       27.4      49.9          2

      3        27.4      49.9       35.4      47.2          2

      4        35.4      47.2       63.4      47.2          2

      5        63.4      47.2       68.9      49.9          2

      6        68.9      49.9       72.5      51.5          2

      7        72.5      51.5      159.4      80.0          2

      8       159.4      80.0      165.8      80.0          2

      9          .0      49.0       23.4      49.2          1

     10        23.4      49.2       35.4      45.2          1

     11        35.4      45.2       63.4      45.2          1

     12        63.4      45.2       72.5      49.2          1

     13        72.5      49.2      165.8      80.0          1

     14       165.8      80.0      167.8      80.0          1



  --------------------------

  ISOTROPIC Soil Parameters

  --------------------------


   3 Soil unit(s) specified


  Soil  Unit Weight  Cohesion  Friction   Pore Pressure      Water

  Unit  Moist   Sat. Intercept   Angle  Parameter Constant  Surface

   No.  (pcf)  (pcf)   (psf)     (deg)     Ru      (psf)      No.


    1   108.7  130.0    718.0    11.40    .000         .0      1

    2   120.0  120.0    100.0    16.00    .000         .0      0

    3   116.0  116.0    100.0    16.00    .000         .0      0
```

```
 1 Water surface(s) have been specified


Unit weight of water =    62.40  (pcf)



Water Surface No.  1 specified by  2 coordinate points


**********************************

        PHREATIC SURFACE,

**********************************


  Point      x-water     y-water

   No.         (ft)        (ft)


    1          90.00       55.00

    2         215.00       55.00




A critical failure surface searching method, using a random

technique for generating CIRCULAR surfaces has been specified.



  100 trial surfaces will be generated and analyzed.



  10 Surfaces initiate from each of  10 points equally spaced

along the ground surface between  x =     65.0 ft

                           and  x =    100.0 ft



Each surface terminates between   x =    130.0 ft

                          and   x =    160.0 ft



Unless further limitations were imposed, the minimum elevation
```

```
  at which a surface extends is  y =     30.0 ft



    10.0 ft line segments define each trial failure surface.




  ---------------------
  ANGULAR RESTRICTIONS
  ---------------------


    The first segment of each failure surface will be inclined
    within the angular range defined by :


            Lower angular limit :=   -45.0 degrees
            Upper angular limit :=  (slope angle - 5.0) degrees




**********************************************************************
    --  WARNING  --  WARNING  --  WARNING  --  WARNING  --  (# 48)
**********************************************************************
Negative effective stresses were calculated at the base of a slice.
This warning is usually reported for cases where slices have low self
weight and a relatively high "c" shear strength parameter.  In such
cases, this effect can only be eliminated by reducing the "c" value.
**********************************************************************


    -----------------------------------------------------------
    USER SELECTED option to maintain strength greater than zero
    -----------------------------------------------------------




    Factors of safety have been calculated by the :


    * * * * *   SIMPLIFIED BISHOP METHOD   * * * * *
```

```
The most critical circular failure surface

is specified by  7 coordinate points


     Point      x-surf      y-surf

      No.        (ft)        (ft)


       1          96.11       61.33

       2         106.04       62.53

       3         115.83       64.59

       4         125.39       67.50

       5         134.67       71.23

       6         143.58       75.76

       7         150.48       80.06



****  Simplified BISHOP FOS =    1.977  ****






The following is a summary of the TEN most critical surfaces


Problem Description : HARDIN CO LF INT PER SLOPE ANALYSIS


        FOS     Circle Center    Radius   Initial Terminal  Resisting

      (BISHOP)  x-coord  y-coord          x-coord  x-coord    Moment

                 (ft)     (ft)     (ft)     (ft)     (ft)     (ft-lb)


  1.   1.977     87.45   175.15   114.14    96.11   150.48  1.171E+06

  2.   2.067     48.80   309.14   252.90    92.22   159.95  3.098E+06

  3.   2.293     57.18   250.19   194.05    88.33   150.26  2.020E+06

  4.   2.302     71.32   174.93   118.34    84.44   133.14  9.744E+05

  5.   2.377    106.26    87.73    53.82    65.00   159.54  6.999E+06

  6.   2.421    107.95    88.25    52.49    68.89   160.06  6.492E+06

  7.   2.422    107.71    88.35    52.37    68.89   159.64  6.417E+06

  8.   2.431    100.67   108.51    63.80    68.89   158.49  6.130E+06
```

```
    9.   2.464   100.44   111.90    49.23   100.00   130.50  2.516E+05

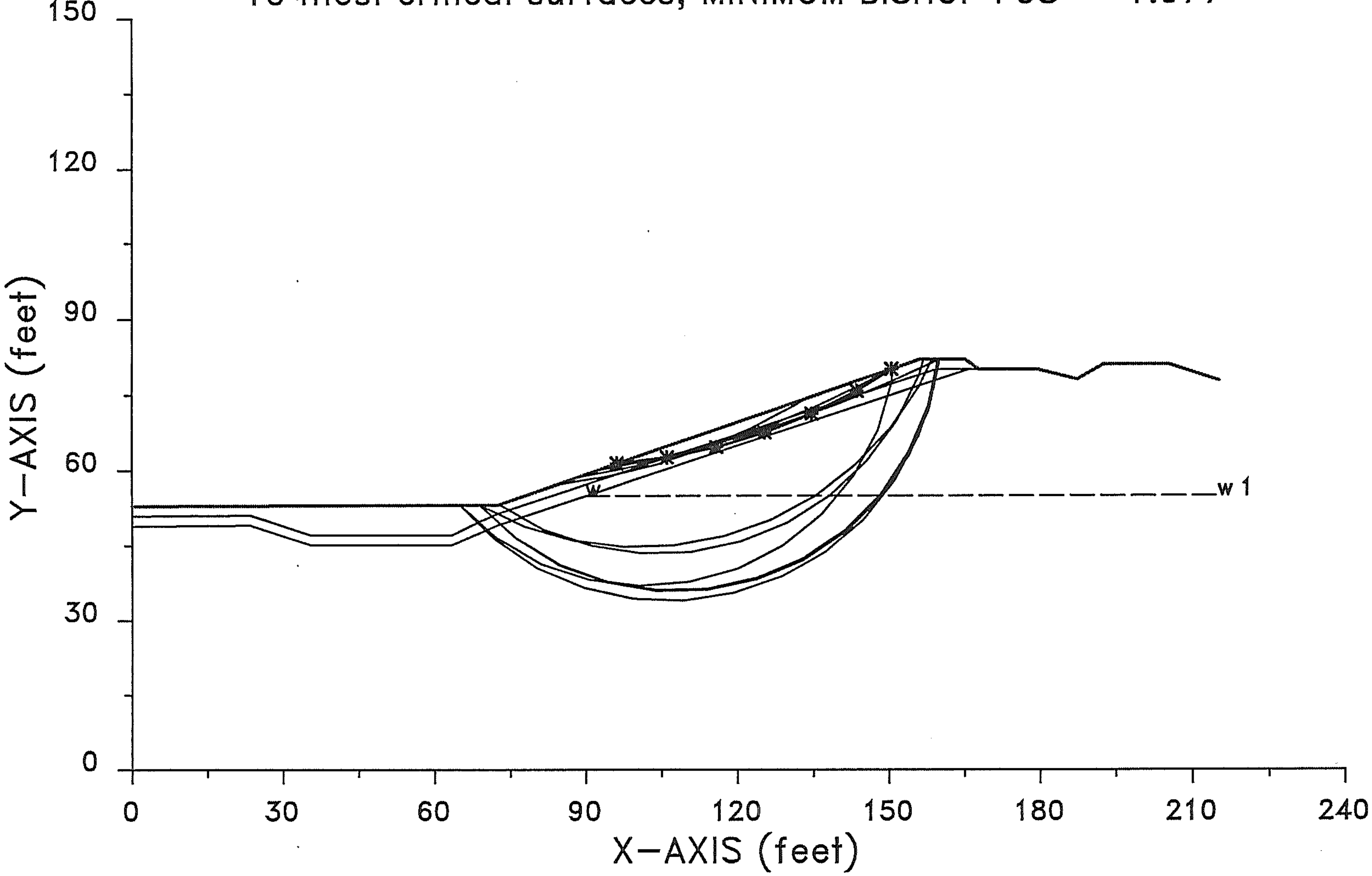
   10.   2.475   104.33    98.71    55.30    72.78   156.83  5.344E+06



                   * * *  END OF FILE  * * *
```

EX-2-4-E 2-10-17 8:18

HARDIN CO LF INT PER SLOPE ANALYSIS
10 most critical surfaces, MINIMUM BISHOP FOS = 1.977



```
XSTABL File: EX-2-4-E   2-10-17    8:18

          *****************************************
          *              X S T A B L              *
          *                                       *
          *        Slope Stability Analysis       *
          *               using the               *
          *            Method of Slices           *
          *                                       *
          *        Copyright (C) 1992 - 2008      *
          *   Interactive Software Designs, Inc.  *
          *        Moscow, ID 83843, U.S.A.       *
          *                                       *
          *          All Rights Reserved          *
          *                                       *
          * Ver. 5.208                96 - 2046   *
          *****************************************


    Problem Description : HARDIN CO LF INT PER SLOPE ANALYSIS


    -----------------------------
    SEGMENT BOUNDARY COORDINATES
    -----------------------------

        9 SURFACE boundary segments

    Segment   x-left   y-left   x-right   y-right    Soil Unit
      No.      (ft)     (ft)      (ft)      (ft)    Below Segment

       1          .0     53.0      72.5      53.2         3
       2        72.5     53.2     156.1      82.0         3
       3       156.1     82.0     165.0      82.0         3
       4       165.0     82.0     167.8      80.0         3
       5       167.8     80.0     179.3      80.0         1
       6       179.3     80.0     187.3      78.0         1
```

```
     7      187.3      78.0      192.5      81.0         1

     8      192.5      81.0      205.3      81.0         1

     9      205.3      81.0      215.2      77.7         1



    14 SUBSURFACE boundary segments


  Segment    x-left    y-left    x-right   y-right     Soil Unit

     No.      (ft)      (ft)      (ft)      (ft)     Below Segment


     1         .0      51.0       23.4      51.2          2

     2       23.4      51.2       27.4      49.9          2

     3       27.4      49.9       35.4      47.2          2

     4       35.4      47.2       63.4      47.2          2

     5       63.4      47.2       68.9      49.9          2

     6       68.9      49.9       72.5      51.5          2

     7       72.5      51.5      159.4      80.0          2

     8      159.4      80.0      165.8      80.0          2

     9         .0      49.0       23.4      49.2          1

    10       23.4      49.2       35.4      45.2          1

    11       35.4      45.2       63.4      45.2          1

    12       63.4      45.2       72.5      49.2          1

    13       72.5      49.2      165.8      80.0          1

    14      165.8      80.0      167.8      80.0          1



  --------------------------

  ISOTROPIC Soil Parameters

  --------------------------


   3 Soil unit(s) specified


  Soil   Unit Weight   Cohesion  Friction    Pore Pressure      Water

  Unit  Moist    Sat.  Intercept   Angle  Parameter  Constant  Surface

   No.  (pcf)   (pcf)    (psf)     (deg)      Ru       (psf)      No.


    1   108.7   130.0    644.0    12.60      .000         .0       1

    2   120.0   120.0    100.0    16.00      .000         .0       0

    3   116.0   116.0    100.0    16.00      .000         .0       0
```

```
 1 Water surface(s) have been specified

Unit weight of water =    62.40  (pcf)

Water Surface No.  1 specified by  2 coordinate points

**********************************
        PHREATIC SURFACE,
**********************************

  Point      x-water     y-water
   No.        (ft)        (ft)

    1          90.00       55.00
    2         215.00       55.00

A critical failure surface searching method, using a random
technique for generating CIRCULAR surfaces has been specified.

  100 trial surfaces will be generated and analyzed.

  10 Surfaces initiate from each of  10 points equally spaced
along the ground surface between  x =      65.0 ft
                             and  x =     100.0 ft

Each surface terminates between   x =     130.0 ft
                            and   x =     160.0 ft

Unless further limitations were imposed, the minimum elevation
```

```
      at which a surface extends is  y =     30.0 ft



         10.0 ft line segments define each trial failure surface.




      ---------------------

      ANGULAR RESTRICTIONS

      ---------------------


        The first segment of each failure surface will be inclined

        within the angular range defined by :


                Lower angular limit :=   -45.0 degrees

                Upper angular limit :=  (slope angle - 5.0) degrees





  ************************************************************************

      --  WARNING  --  WARNING  --  WARNING  --  WARNING  --  (# 48)

  ************************************************************************

  Negative effective stresses were calculated at the base of a slice.

  This warning is usually reported for cases where slices have low self

  weight and a relatively high "c" shear strength parameter.  In such

  cases, this effect can only be eliminated by reducing the "c" value.

  ************************************************************************


      ------------------------------------------------------------

      USER SELECTED option to maintain strength greater than zero

      ------------------------------------------------------------




      Factors of safety have been calculated by the :


      * * * * *   SIMPLIFIED BISHOP METHOD   * * * * *
```

```
The most critical circular failure surface

is specified by  7 coordinate points


     Point      x-surf      y-surf

      No.        (ft)        (ft)


       1         96.11       61.33

       2        106.04       62.53

       3        115.83       64.59

       4        125.39       67.50

       5        134.67       71.23

       6        143.58       75.76

       7        150.48       80.06



****  Simplified BISHOP FOS =    1.977  ****






The following is a summary of the TEN most critical surfaces


Problem Description : HARDIN CO LF INT PER SLOPE ANALYSIS


        FOS     Circle Center    Radius  Initial Terminal  Resisting

      (BISHOP)  x-coord  y-coord           x-coord  x-coord    Moment

                  (ft)     (ft)     (ft)     (ft)     (ft)     (ft-lb)


  1.    1.977    87.45   175.15   114.14    96.11   150.48  1.171E+06

  2.    2.067    48.80   309.14   252.90    92.22   159.95  3.098E+06

  3.    2.293    57.18   250.19   194.05    88.33   150.26  2.020E+06

  4.    2.302    71.32   174.93   118.34    84.44   133.14  9.744E+05

  5.    2.317   106.26    87.73    53.82    65.00   159.54  6.823E+06

  6.    2.332   100.67   108.51    63.80    68.89   158.49  5.881E+06

  7.    2.355   107.95    88.25    52.49    68.89   160.06  6.317E+06

  8.    2.356   107.71    88.35    52.37    68.89   159.64  6.242E+06
```

```
    9.   2.381    104.33    98.71    55.30    72.78   156.83  5.142E+06

   10.   2.416    101.73    86.57    49.64    65.00   150.75  5.261E+06

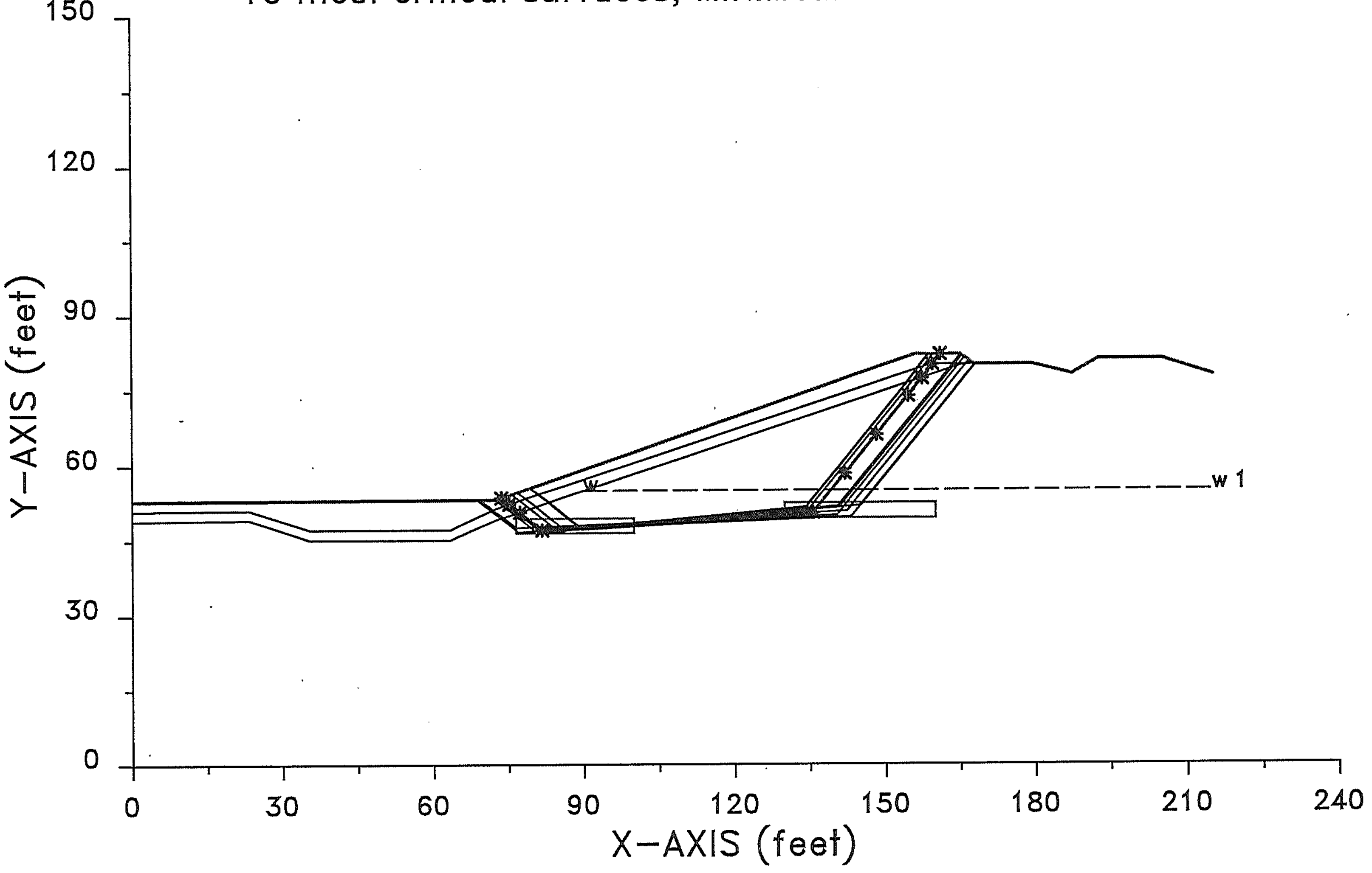

                     * * *  END OF FILE  * * *
```

SECTION E1-5 (FIGURE IIIE-A-3)

CONSTRUCTED LINER SLOPE STABILITY ANALYSIS
RANKINE BLOCK METHOD
TOTAL AND EFFECTIVE STRESS MODELS

EX-2-5-T 2-10-17 8:22

HARDIN CO LF INT PER SLOPE ANALYSIS
10 most critical surfaces, MINIMUM JANBU FOS = 2.604



```
XSTABL File: EX-2-5-T   2-10-17    8:22


          ******************************************
          *              X S T A B L               *
          *                                        *
          *        Slope Stability Analysis        *
          *               using the                *
          *           Method of Slices             *
          *                                        *
          *        Copyright (C) 1992 - 2008       *
          *   Interactive Software Designs, Inc.   *
          *        Moscow, ID 83843, U.S.A.        *
          *                                        *
          *          All Rights Reserved           *
          *                                        *
          * Ver. 5.208                 96 - 2046   *
          ******************************************



    Problem Description : HARDIN CO LF INT PER SLOPE ANALYSIS



    -----------------------------
    SEGMENT BOUNDARY COORDINATES
    -----------------------------

        9 SURFACE boundary segments

    Segment   x-left   y-left   x-right   y-right    Soil Unit
      No.      (ft)     (ft)      (ft)      (ft)    Below Segment

      1          .0     53.0      72.5      53.2         3
      2        72.5     53.2     156.1      82.0         3
      3       156.1     82.0     165.0      82.0         3
      4       165.0     82.0     167.8      80.0         3
      5       167.8     80.0     179.3      80.0         1
      6       179.3     80.0     187.3      78.0         1
```

```
    7     187.3     78.0     192.5     81.0        1

    8     192.5     81.0     205.3     81.0        1

    9     205.3     81.0     215.2     77.7        1



  14 SUBSURFACE boundary segments


Segment   x-left   y-left   x-right  y-right    Soil Unit

  No.      (ft)     (ft)      (ft)     (ft)    Below Segment


   1         .0     51.0      23.4     51.2        2

   2       23.4     51.2      27.4     49.9        2

   3       27.4     49.9      35.4     47.2        2

   4       35.4     47.2      63.4     47.2        2

   5       63.4     47.2      68.9     49.9        2

   6       68.9     49.9      72.5     51.5        2

   7       72.5     51.5     159.4     80.0        2

   8      159.4     80.0     165.8     80.0        2

   9         .0     49.0      23.4     49.2        1

  10       23.4     49.2      35.4     45.2        1

  11       35.4     45.2      63.4     45.2        1

  12       63.4     45.2      72.5     49.2        1

  13       72.5     49.2     165.8     80.0        1

  14      165.8     80.0     167.8     80.0        1



--------------------------

ISOTROPIC Soil Parameters

--------------------------


 3 Soil unit(s) specified


Soil   Unit Weight   Cohesion  Friction   Pore Pressure      Water

Unit  Moist    Sat.  Intercept  Angle  Parameter  Constant  Surface

 No.  (pcf)   (pcf)    (psf)    (deg)     Ru       (psf)      No.


  1   108.7   130.0    718.0    11.40    .000         .0       1

  2   120.0   120.0    100.0    16.00    .000         .0       0

  3   116.0   116.0    100.0    16.00    .000         .0       0
```

```
 1 Water surface(s) have been specified

Unit weight of water =    62.40  (pcf)

Water Surface No.  1 specified by  2 coordinate points

**********************************
        PHREATIC SURFACE,
**********************************

  Point     x-water     y-water
   No.        (ft)        (ft)

    1         90.00       55.00
    2        215.00       55.00

A critical failure surface searching method, using a random
technique for generating sliding BLOCK surfaces, has been
specified.

The active and passive portions of the sliding surfaces
are generated according to the Rankine theory.

  100 trial surfaces will be generated and analyzed.

  2 boxes specified for generation of central block base

Length of line segments for active and passive portions of
sliding block is     10.0 ft
```

```
    Box        x-left     y-left    x-right    y-right      Width

    no.         (ft)       (ft)       (ft)       (ft)        (ft)


     1          76.5       48.0      100.0       48.0         3.0

     2         130.0       51.0      160.0       51.0         3.0



************************************************************************

    --  WARNING  --  WARNING  --  WARNING  --  WARNING  --  (# 48)

************************************************************************

Negative effective stresses were calculated at the base of a slice.

This warning is usually reported for cases where slices have low self

weight and a relatively high "c" shear strength parameter.  In such

cases, this effect can only be eliminated by reducing the "c" value.

************************************************************************


    ------------------------------------------------------------

    USER SELECTED option to maintain strength greater than zero

    ------------------------------------------------------------



************************************************************************

ERROR # 39

************************************************************************

The program calculated a point for the ACTIVE wedge that is outside

the defined slope geometry.  The analysis will continue, but the user

should adjust the search box or slope geometry to allow an active

wedge to be formed from all points within last box.

************************************************************************




    Factors of safety have been calculated by the :


    * * * * *   SIMPLIFIED JANBU METHOD   * * * * *
```

```
The 10 most critical of all the failure surfaces examined

are displayed below - the most critical first


Failure surface No. 1 specified by 11 coordinate points


       Point      x-surf      y-surf

        No.        (ft)        (ft)


         1         73.56       53.56

         2         75.15       52.37

         3         77.26       50.77

         4         81.69       47.15

         5        135.67       50.61

         6        142.00       58.35

         7        148.34       66.09

         8        154.67       73.82

         9        157.47       77.25

        10        159.55       80.00

        11        161.05       82.00

  **  Corrected JANBU FOS =   2.604  **     (Fo factor = 1.080)



Failure surface No. 2 specified by 11 coordinate points


       Point      x-surf      y-surf

        No.        (ft)        (ft)


         1         68.77       53.19

         2         71.56       51.08

         3         73.59       49.56

         4         77.00       46.76

         5        140.47       51.63

         6        146.80       59.37

         7        153.13       67.11

         8        159.47       74.84

         9        162.91       79.04
```

```
         10        163.63        80.00

         11        165.09        81.94


 **  Corrected JANBU FOS =    2.611  **     (Fo factor = 1.076)



Failure surface No. 3 specified by 11 coordinate points


        Point      x-surf       y-surf

         No.        (ft)         (ft)


          1         69.35        53.19

          2         71.93        51.25

          3         73.99        49.69

          4         76.59        47.57

          5        140.49        50.16

          6        146.82        57.90

          7        153.16        65.64

          8        159.49        73.37

          9        164.59        79.60

         10        164.89        80.00

         11        165.91        81.35


 **  Corrected JANBU FOS =    2.613  **     (Fo factor = 1.077)



Failure surface No. 4 specified by 10 coordinate points


        Point      x-surf       y-surf

         No.        (ft)         (ft)


          1         73.09        53.40

          2         74.67        52.21

          3         76.79        50.62

          4         80.78        47.35

          5        142.45        50.94

          6        148.78        58.67

          7        155.12        66.41

          8        161.45        74.15
```

```
          9        166.24       80.00

         10        166.78       80.73


   **  Corrected JANBU FOS =   2.619  **     (Fo factor = 1.077)



  Failure surface No. 5 specified by 11 coordinate points


        Point       x-surf       y-surf

         No.         (ft)         (ft)


          1         76.75        54.66

          2         78.39        53.43

          3         80.50        51.84

          4         86.25        47.13

          5        141.10        51.88

          6        147.44        59.62

          7        153.77        67.36

          8        160.10        75.10

          9        163.49        79.24

         10        164.07        80.00

         11        165.37        81.73

   **  Corrected JANBU FOS =   2.619  **     (Fo factor = 1.078)



  Failure surface No. 6 specified by  9 coordinate points


        Point       x-surf       y-surf

         No.         (ft)         (ft)


          1         73.26        53.46

          2         74.85        52.27

          3         76.97        50.67

          4         81.89        46.65

          5        145.26        52.04

          6        151.59        59.77

          7        157.92        67.51

          8        164.26        75.25
```

```
         9       168.15       80.00


  **  Corrected JANBU FOS =   2.626  **     (Fo factor = 1.074)



 Failure surface No. 7 specified by 11 coordinate points


        Point      x-surf       y-surf

         No.        (ft)         (ft)


         1          72.39        53.20

         2          73.99        51.99

         3          76.11        50.39

         4          80.20        47.05

         5         133.82        51.18

         6         140.15        58.92

         7         146.49        66.65

         8         152.82        74.39

         9         154.31        76.21

        10         156.43        79.03

        11         158.67        82.00

  **  Corrected JANBU FOS =   2.636  **     (Fo factor = 1.079)



 Failure surface No. 8 specified by 11 coordinate points


        Point      x-surf       y-surf

         No.        (ft)         (ft)


         1          79.25        55.53

         2          80.93        54.26

         3          83.03        52.68

         4          89.69        47.23

         5         133.66        49.80

         6         139.99        57.54

         7         146.33        65.27

         8         152.66        73.01

         9         155.63        76.64
```

```
          10        157.75        79.46

          11        159.67        82.00


  **  Corrected JANBU FOS =    2.639  **     (Fo factor = 1.083)



 Failure surface No. 9 specified by  9 coordinate points


        Point       x-surf       y-surf

         No.         (ft)         (ft)


          1          79.36        55.56

          2          81.03        54.30

          3          83.14        52.71

          4          89.47        47.53

          5         143.27        49.75

          6         149.60        57.49

          7         155.94        65.23

          8         162.27        72.97

          9         168.03        80.00


  **  Corrected JANBU FOS =    2.646  **     (Fo factor = 1.081)



 Failure surface No.10 specified by 11 coordinate points


        Point       x-surf       y-surf

         No.         (ft)         (ft)


          1          75.65        54.28

          2          77.27        53.06

          3          79.38        51.47

          4          84.70        47.12

          5         134.40        51.73

          6         140.74        59.47

          7         147.07        67.20

          8         153.41        74.94

          9         154.49        76.27

         10         156.61        79.09
```

```
        11        158.81       82.00


  **  Corrected JANBU FOS =   2.649  **    (Fo factor = 1.080)





The following is a summary of the TEN most critical surfaces


Problem Description : HARDIN CO LF INT PER SLOPE ANALYSIS


      Modified        Correction   Initial    Terminal    Available

     JANBU FOS          Factor     x-coord     x-coord     Strength

                                    (ft)        (ft)        (lb)


  1.     2.604          1.080       73.56      161.05    9.653E+04

  2.     2.611          1.076       68.77      165.09    1.059E+05

  3.     2.613          1.077       69.35      165.91    1.082E+05

  4.     2.619          1.077       73.09      166.78    1.076E+05

  5.     2.619          1.078       76.75      165.37    1.009E+05

  6.     2.626          1.074       73.26      168.15    1.097E+05

  7.     2.636          1.079       72.39      158.67    9.293E+04

  8.     2.639          1.083       79.25      159.67    9.030E+04

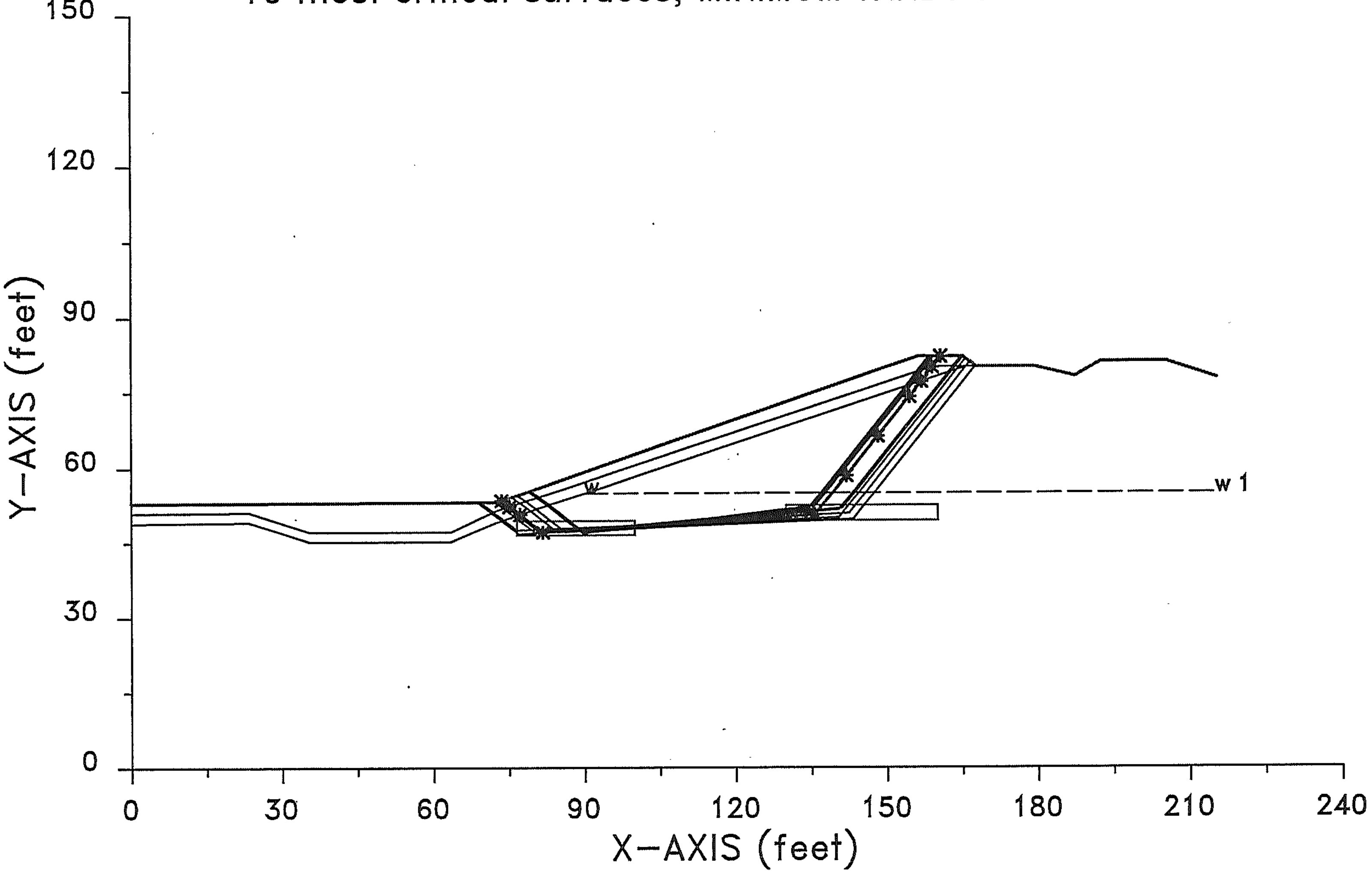
  9.     2.646          1.081       79.36      168.03    1.061E+05

 10.     2.649          1.080       75.65      158.81    9.031E+04


                    * * *  END OF FILE  * * *
```

EX-2-5-E 2-10-17 8:21

HARDIN CO LF INT PER SLOPE ANALYSIS
10 most critical surfaces, MINIMUM JANBU FOS = 2.499



```
XSTABL File: EX-2-5-E   2-10-17    8:21


          ******************************************

          *                X S T A B L             *

          *                                        *

          *         Slope Stability Analysis       *

          *                using the               *

          *             Method of Slices           *

          *                                        *

          *         Copyright (C) 1992 - 2008      *

          *   Interactive Software Designs, Inc.   *

          *         Moscow, ID 83843, U.S.A.       *

          *                                        *

          *           All Rights Reserved          *

          *                                        *

          *  Ver. 5.208               96 - 2046    *

          ******************************************



    Problem Description : HARDIN CO LF INT PER SLOPE ANALYSIS




    -----------------------------

    SEGMENT BOUNDARY COORDINATES

    -----------------------------


        9 SURFACE boundary segments


    Segment   x-left    y-left    x-right   y-right     Soil Unit

       No.     (ft)      (ft)      (ft)      (ft)     Below Segment


       1          .0      53.0      72.5      53.2          3

       2        72.5      53.2     156.1      82.0          3

       3       156.1      82.0     165.0      82.0          3

       4       165.0      82.0     167.8      80.0          3

       5       167.8      80.0     179.3      80.0          1

       6       179.3      80.0     187.3      78.0          1
```

```
      7       187.3      78.0      192.5      81.0         1

      8       192.5      81.0      205.3      81.0         1

      9       205.3      81.0      215.2      77.7         1



     14 SUBSURFACE boundary segments


   Segment    x-left    y-left    x-right   y-right     Soil Unit

     No.       (ft)      (ft)       (ft)      (ft)    Below Segment


      1          .0      51.0       23.4      51.2         2

      2        23.4      51.2       27.4      49.9         2

      3        27.4      49.9       35.4      47.2         2

      4        35.4      47.2       63.4      47.2         2

      5        63.4      47.2       68.9      49.9         2

      6        68.9      49.9       72.5      51.5         2

      7        72.5      51.5      159.4      80.0         2

      8       159.4      80.0      165.8      80.0         2

      9          .0      49.0       23.4      49.2         1

     10        23.4      49.2       35.4      45.2         1

     11        35.4      45.2       63.4      45.2         1

     12        63.4      45.2       72.5      49.2         1

     13        72.5      49.2      165.8      80.0         1

     14       165.8      80.0      167.8      80.0         1



  --------------------------

  ISOTROPIC Soil Parameters

  --------------------------


   3 Soil unit(s) specified


  Soil   Unit Weight   Cohesion  Friction    Pore Pressure     Water

  Unit  Moist    Sat.  Intercept   Angle  Parameter Constant  Surface

   No.  (pcf)   (pcf)    (psf)     (deg)      Ru      (psf)      No.


    1   108.7   130.0    644.0     12.60     .000         .0      1

    2   120.0   120.0    100.0     16.00     .000         .0      0

    3   116.0   116.0    100.0     16.00     .000         .0      0
```

```
 1 Water surface(s) have been specified

Unit weight of water =    62.40  (pcf)


Water Surface No.  1 specified by  2 coordinate points

**********************************
        PHREATIC SURFACE,
**********************************

  Point      x-water     y-water
   No.         (ft)        (ft)

    1          90.00       55.00
    2         215.00       55.00



A critical failure surface searching method, using a random
technique for generating sliding BLOCK surfaces, has been
specified.


The active and passive portions of the sliding surfaces
are generated according to the Rankine theory.


  100 trial surfaces will be generated and analyzed.


  2 boxes specified for generation of central block base


Length of line segments for active and passive portions of
sliding block is     10.0 ft
```

```
 Box        x-left     y-left    x-right    y-right      Width

 no.         (ft)       (ft)      (ft)       (ft)        (ft)


  1           76.5       48.0      100.0       48.0        3.0

  2          130.0       51.0      160.0       51.0        3.0


*************************************************************************

    -- WARNING -- WARNING -- WARNING -- WARNING -- (# 48)

*************************************************************************

Negative effective stresses were calculated at the base of a slice.

This warning is usually reported for cases where slices have low self

weight and a relatively high "c" shear strength parameter.  In such

cases, this effect can only be eliminated by reducing the "c" value.

*************************************************************************


    -------------------------------------------------------------

    USER SELECTED option to maintain strength greater than zero

    -------------------------------------------------------------




    Factors of safety have been calculated by the :


    * * * * *   SIMPLIFIED JANBU METHOD   * * * * *




    The 10 most critical of all the failure surfaces examined

    are displayed below - the most critical first


    Failure surface No. 1 specified by 11 coordinate points


           Point      x-surf      y-surf

            No.        (ft)        (ft)


             1        73.49       53.54
```

```
          2        75.08       52.35

          3        77.20       50.75

          4        81.69       47.15

          5       135.67       50.61

          6       141.92       58.41

          7       148.17       66.22

          8       154.43       74.02

          9       156.85       77.04

         10       158.97       79.86

         11       160.58       82.00


**  Corrected JANBU FOS =   2.499  **    (Fo factor = 1.080)



Failure surface No. 2 specified by 11 coordinate points


        Point     x-surf      y-surf

         No.       (ft)        (ft)


          1        68.69       53.19

          2        71.51       51.06

          3        73.53       49.54

          4        77.00       46.76

          5       140.47       51.63

          6       146.72       59.43

          7       152.97       67.24

          8       159.22       75.04

          9       162.26       78.83

         10       163.14       80.00

         11       164.65       82.00


**  Corrected JANBU FOS =   2.503  **    (Fo factor = 1.076)



Failure surface No. 3 specified by 11 coordinate points


        Point     x-surf      y-surf

         No.       (ft)        (ft)
```

```
          1         69.29        53.19

          2         71.89        51.23

          3         73.95        49.68

          4         76.59        47.57

          5        140.49        50.16

          6        146.74        57.96

          7        153.00        65.77

          8        159.25        73.57

          9        163.89        79.37

         10        164.37        80.00

         11        165.57        81.59


  **  Corrected JANBU FOS =   2.507  **    (Fo factor = 1.078)



 Failure surface No. 4 specified by 11 coordinate points


        Point      x-surf      y-surf

         No.        (ft)        (ft)


          1         76.66        54.63

          2         78.30        53.40

          3         80.41        51.81

          4         86.25        47.13

          5        141.10        51.88

          6        147.35        59.69

          7        153.61        67.49

          8        159.86        75.30

          9        162.85        79.02

         10        163.58        80.00

         11        165.06        81.96

  **  Corrected JANBU FOS =   2.511  **    (Fo factor = 1.079)



 Failure surface No. 5 specified by 11 coordinate points


        Point      x-surf      y-surf

         No.        (ft)        (ft)
```

```
          1          73.03        53.38

          2          74.61        52.19

          3          76.73        50.60

          4          80.78        47.35

          5         142.45        50.94

          6         148.70        58.74

          7         154.95        66.54

          8         161.21        74.35

          9         165.71        79.97

         10         165.73        80.00

         11         166.46        80.96


 **  Corrected JANBU FOS =   2.513  **    (Fo factor = 1.077)



Failure surface No. 6 specified by 11 coordinate points


        Point      x-surf       y-surf

         No.        (ft)         (ft)


          1          72.30        53.20

          2          73.93        51.97

          3          76.05        50.37

          4          80.20        47.05

          5         133.82        51.18

          6         140.07        58.98

          7         146.33        66.79

          8         152.58        74.59

          9         153.72        76.01

         10         155.84        78.83

         11         158.23        82.00


 **  Corrected JANBU FOS =   2.529  **    (Fo factor = 1.079)



Failure surface No. 7 specified by 11 coordinate points


        Point      x-surf       y-surf
```

```
       No.        (ft)       (ft)


        1        79.15      55.49

        2        80.82      54.23

        3        82.93      52.64

        4        89.69      47.23

        5       133.66      49.80

        6       139.91      57.60

        7       146.16      65.41

        8       152.42      73.21

        9       155.00      76.43

       10       157.12      79.25

       11       159.19      82.00


 **  Corrected JANBU FOS =    2.536  **    (Fo factor = 1.084)



Failure surface No. 8 specified by 11 coordinate points


      Point      x-surf      y-surf

       No.        (ft)        (ft)


        1        75.57       54.26

        2        77.19       53.04

        3        79.30       51.44

        4        84.70       47.12

        5       134.40       51.73

        6       140.66       59.53

        7       146.91       67.34

        8       153.16       75.14

        9       153.91       76.08

       10       156.04       78.90

       11       158.38       82.00


 **  Corrected JANBU FOS =    2.541  **    (Fo factor = 1.080)



Failure surface No. 9 specified by 10 coordinate points
```

```
        Point     x-surf      y-surf

         No.       (ft)        (ft)


          1        79.26       55.53

          2        80.94       54.27

          3        83.04       52.68

          4        89.47       47.53

          5       143.27       49.75

          6       149.52       57.56

          7       155.78       65.36

          8       162.03       73.17

          9       167.50       80.00

         10       167.61       80.14


**  Corrected JANBU FOS =    2.542  **     (Fo factor = 1.081)



Failure surface No.10 specified by 11 coordinate points


        Point     x-surf      y-surf

         No.       (ft)        (ft)


          1        78.86       55.39

          2        80.53       54.13

          3        82.64       52.55

          4        89.69       46.90

          5       135.38       52.31

          6       141.63       60.11

          7       147.89       67.91

          8       154.14       75.72

          9       154.61       76.31

         10       156.73       79.13

         11       158.90       82.00


**  Corrected JANBU FOS =    2.544  **     (Fo factor = 1.081)
```

```
The following is a summary of the TEN most critical surfaces


Problem Description : HARDIN CO LF INT PER SLOPE ANALYSIS


       Modified     Correction   Initial    Terminal   Available

       JANBU FOS      Factor     x-coord    x-coord     Strength

                                   (ft)       (ft)        (lb)


  1.     2.499        1.080       73.49      160.58    9.204E+04

  2.     2.503        1.076       68.69      164.65    1.011E+05

  3.     2.507        1.078       69.29      165.57    1.034E+05

  4.     2.511        1.079       76.66      165.06    9.641E+04

  5.     2.513        1.077       73.03      166.46    1.030E+05

  6.     2.529        1.079       72.30      158.23    8.846E+04

  7.     2.536        1.084       79.15      159.19    8.616E+04

  8.     2.541        1.080       75.57      158.38    8.602E+04

  9.     2.542        1.081       79.26      167.61    1.018E+05

 10.     2.544        1.081       78.86      158.90    8.432E+04

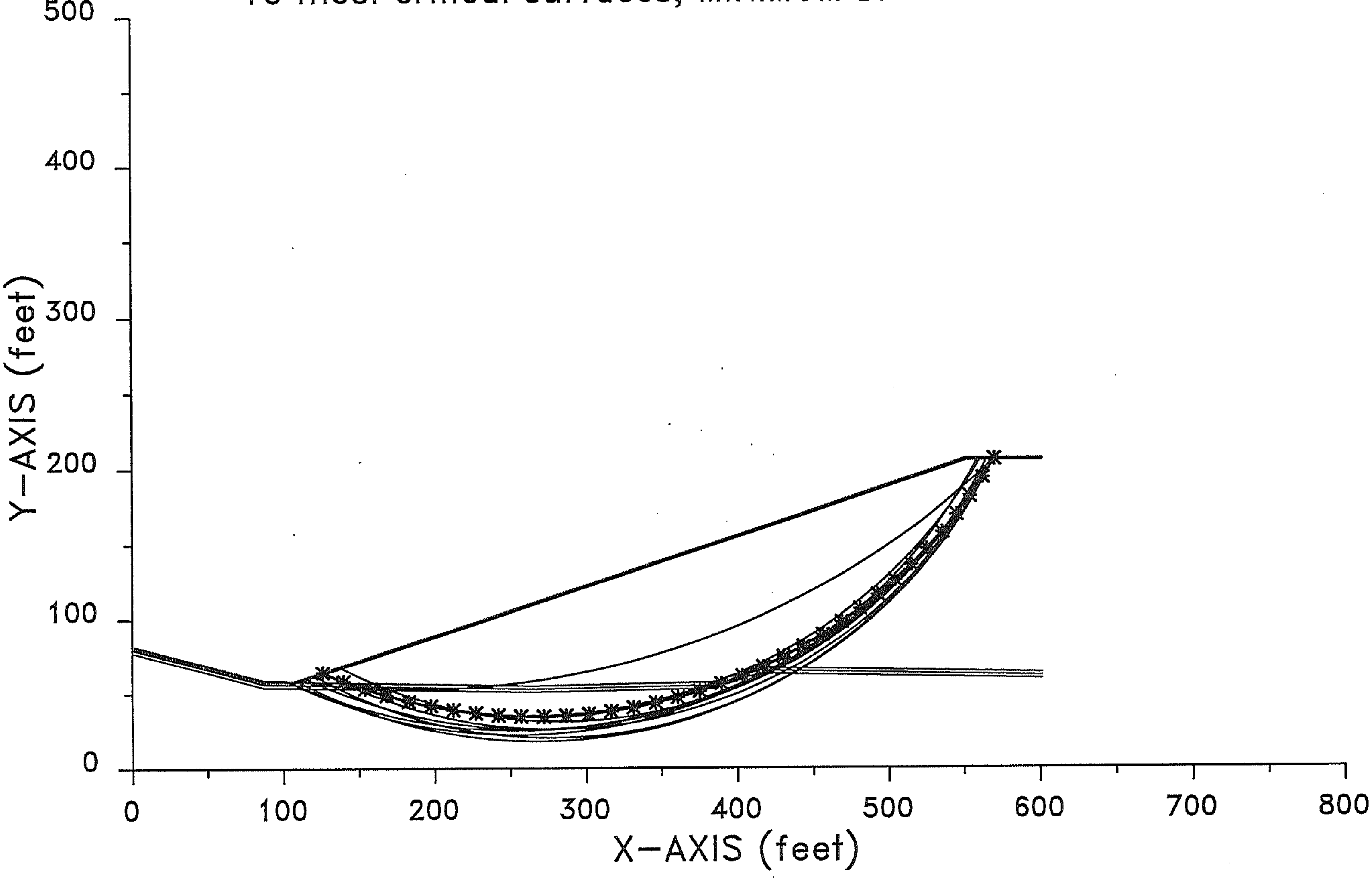

                 * * *  END OF FILE  * * *
```

SECTION E2-6 (FIGURE IIIE-A-4)

INTERIM WASTE SLOPE (3H:1V) STABILITY ANALYSIS
BISHOP CIRCULAR METHOD
TOTAL AND EFFECTIVE STRESS MODELS

HIB-1T 2-03-17 11:27

Hardin Interim Slope (3:1) Total
10 most critical surfaces, MINIMUM BISHOP FOS = 1.607



```
XSTABL File: HIB-1T     2-03-17   11:27

          ******************************************
          *                X S T A B L             *
          *                                        *
          *          Slope Stability Analysis      *
          *                using the               *
          *             Method of Slices           *
          *                                        *
          *          Copyright (C) 1992 - 2008     *
          *     Interactive Software Designs, Inc.  *
          *          Moscow, ID 83843, U.S.A.       *
          *                                        *
          *            All Rights Reserved          *
          *                                        *
          * Ver. 5.208                  96 - 2046  *
          ******************************************


     Problem Description : Hardin Interim Slope (3:1) Total



     -----------------------------
     SEGMENT BOUNDARY COORDINATES
     -----------------------------

         4 SURFACE boundary segments

     Segment     x-left     y-left     x-right    y-right     Soil Unit
       No.        (ft)       (ft)       (ft)       (ft)     Below Segment

        1           .0        81.8       87.4       58.4          4
        2         87.4        58.4      107.3       58.0          4
        3        107.3        58.0      551.3      206.0          2
        4        551.3       206.0      601.3      206.0          2


        17 SUBSURFACE boundary segments

     Segment     x-left     y-left     x-right    y-right     Soil Unit
       No.        (ft)       (ft)       (ft)       (ft)     Below Segment

        1        107.3        58.0      110.3       58.0          4
        2        110.3        58.0      551.4      205.0          5
        3        551.4       205.0      601.3      205.0          5
        4        110.3        58.0      264.4       55.0          4
        5        264.4        55.0      383.6       57.3          4
        6        383.6        57.3      413.7       67.3          4
        7        413.7        67.3      601.3       63.6          4
        8           .0        79.8       87.1       56.4          3
        9         87.1        56.4      264.4       53.0          3
       10        264.4        53.0      383.9       55.3          3
       11        383.9        55.3      414.0       65.3          3
       12        414.0        65.3      601.3       61.6          3
       13           .0        77.7       86.9       54.4          1
       14         86.9        54.4      264.4       51.0          1
       15        264.4        51.0      384.3       53.3          1
       16        384.3        53.3      414.3       63.3          1
       17        414.3        63.3      601.3       59.6          1
```

```
--------------------------
ISOTROPIC Soil Parameters
--------------------------

 5 Soil unit(s) specified

Soil   Unit Weight   Cohesion  Friction    Pore Pressure      Water
Unit  Moist    Sat.  Intercept   Angle  Parameter  Constant  Surface
 No.  (pcf)   (pcf)    (psf)     (deg)      Ru       (psf)     No.

  1   108.7   130.0     718.0    11.40     .000          .0      0
  2   116.0   116.0     100.0    16.00     .000          .0      0
  3   120.0   120.0     100.0    16.00     .000          .0      0
  4   116.0   116.0     100.0    16.00     .000          .0      0
  5    66.0    66.0     288.0    23.00     .000          .0      0


A critical failure surface searching method, using a random
technique for generating CIRCULAR surfaces has been specified.


  100 trial surfaces will be generated and analyzed.


  10 Surfaces initiate from each of  10 points equally spaced
along the ground surface between  x =     98.0 ft
                              and  x =    148.0 ft


Each surface terminates between    x =    520.0 ft
                            and    x =    570.0 ft


Unless further limitations were imposed, the minimum elevation
at which a surface extends is  y =       .0 ft


* * * * *  DEFAULT SEGMENT LENGTH SELECTED BY XSTABL  * * * * *

   15.0 ft line segments define each trial failure surface.


---------------------
ANGULAR RESTRICTIONS
---------------------

  The first segment of each failure surface will be inclined
  within the angular range defined by :

          Lower angular limit :=   -45.0 degrees
          Upper angular limit :=  (slope angle - 5.0) degrees



Factors of safety have been calculated by the :

* * * * *   SIMPLIFIED BISHOP METHOD   * * * * *
```

```
The most critical circular failure surface
is specified by 35 coordinate points

     Point       x-surf       y-surf
      No.         (ft)         (ft)

       1         125.78        64.16
       2         139.65        58.45
       3         153.75        53.33
       4         168.05        48.81
       5         182.53        44.90
       6         197.17        41.62
       7         211.93        38.96
       8         226.79        36.92
       9         241.73        35.52
      10         256.71        34.76
      11         271.71        34.63
      12         286.70        35.14
      13         301.65        36.29
      14         316.55        38.07
      15         331.35        40.48
      16         346.04        43.51
      17         360.59        47.17
      18         374.97        51.44
      19         389.15        56.32
      20         403.12        61.80
      21         416.84        67.87
      22         430.28        74.51
      23         443.44        81.72
      24         456.28        89.48
      25         468.77        97.77
      26         480.90       106.60
      27         492.65       115.92
      28         503.99       125.74
      29         514.90       136.03
      30         525.37       146.78
      31         535.37       157.96
      32         544.89       169.55
      33         553.90       181.54
      34         562.40       193.90
      35         569.98       206.00

****  Simplified BISHOP FOS =   1.607  ****



The following is a summary of the TEN most critical surfaces

Problem Description : Hardin Interim Slope (3:1) Total

        FOS      Circle Center    Radius   Initial  Terminal  Resisting
      (BISHOP)  x-coord  y-coord           x-coord  x-coord    Moment
                 (ft)     (ft)      (ft)     (ft)     (ft)     (ft-lb)

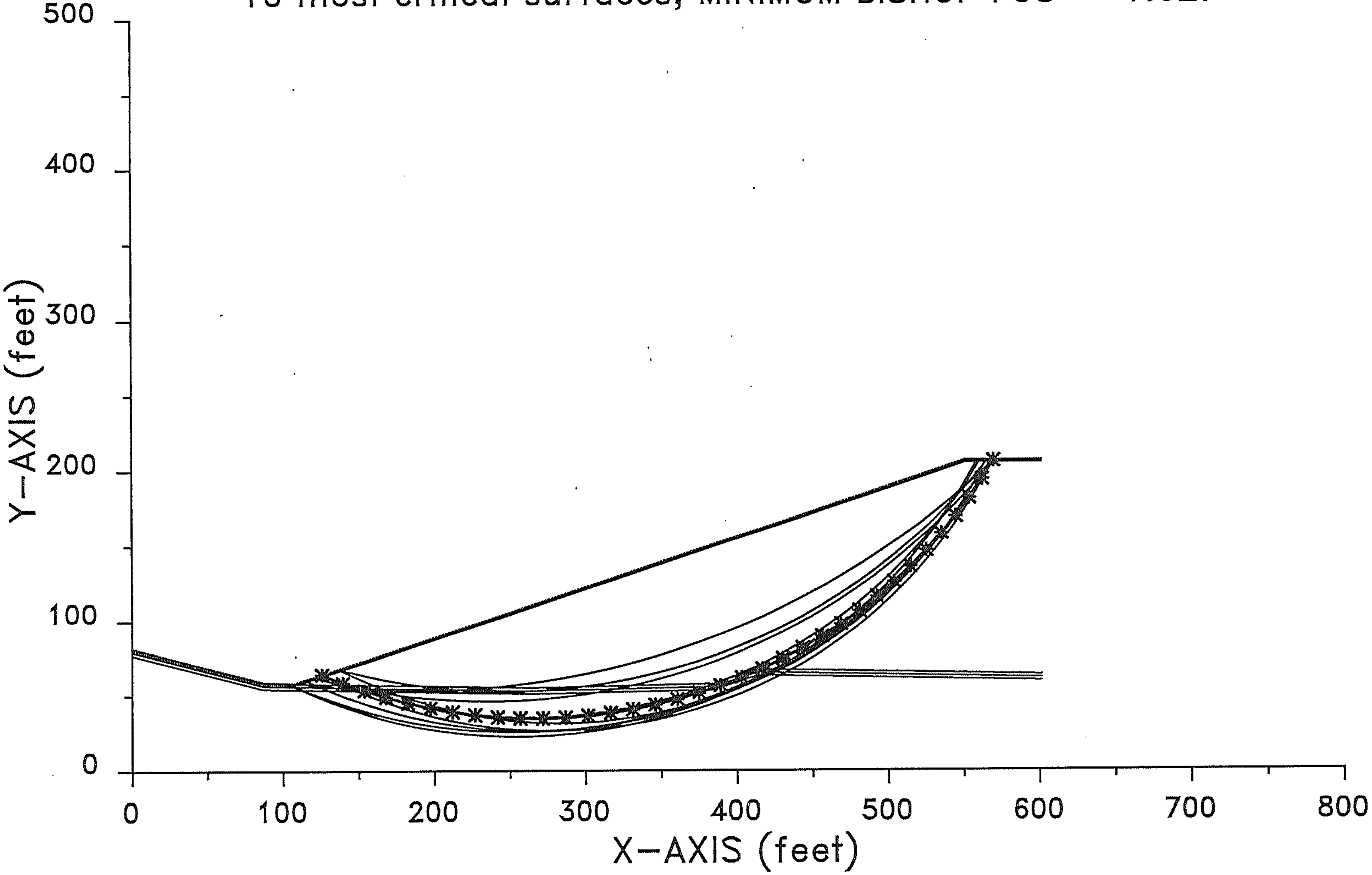
 1.    1.607    267.21   387.76   353.15   125.78   569.98   2.968E+08
 2.    1.618    254.56   390.64   365.25   103.56   569.63   3.416E+08
 3.    1.621    264.70   372.36   346.11   114.67   568.21   3.183E+08
 4.    1.631    278.18   351.73   325.46   125.78   569.10   2.998E+08
```

```
  5.   1.638    283.55   345.57   314.05   136.89   564.79  2.696E+08
  6.   1.638    269.01   352.14   334.26   109.11   569.63  3.372E+08
  7.   1.638    255.74   364.23   341.89   103.56   558.74  3.186E+08
  8.   1.642    187.80   606.86   553.87   109.11   569.94  3.503E+08
  9.   1.648    263.92   378.55   343.41   125.78   560.73  2.827E+08
 10.   1.650    277.54   335.49   315.24   120.22   564.93  3.051E+08

                      * * *  END OF FILE  * * *
```

HIB-1E 2-03-17 11:25

Hardin Interim Slope (3:1) Effective
10 most critical surfaces, MINIMUM BISHOP FOS = 1.627



```
XSTABL File: HIB-1E     2-03-17   11:25

                 ******************************************
                 *              X S T A B L               *
                 *                                        *
                 *        Slope Stability Analysis        *
                 *               using the                *
                 *            Method of Slices            *
                 *                                        *
                 *        Copyright (C) 1992 - 2008       *
                 *   Interactive Software Designs, Inc.   *
                 *        Moscow, ID 83843, U.S.A.        *
                 *                                        *
                 *          All Rights Reserved           *
                 *                                        *
                 * Ver. 5.208                 96 - 2046   *
                 ******************************************


     Problem Description : Hardin Interim Slope (3:1) Effective



     -----------------------------
     SEGMENT BOUNDARY COORDINATES
     -----------------------------

         4 SURFACE boundary segments

     Segment    x-left    y-left    x-right    y-right     Soil Unit
       No.       (ft)      (ft)      (ft)       (ft)     Below Segment

        1          .0      81.8       87.4       58.4          4
        2        87.4      58.4      107.3       58.0          4
        3       107.3      58.0      551.3      206.0          2
        4       551.3     206.0      601.3      206.0          2

        17 SUBSURFACE boundary segments

     Segment    x-left    y-left    x-right    y-right     Soil Unit
       No.       (ft)      (ft)      (ft)       (ft)     Below Segment

        1       107.3      58.0      110.3       58.0          4
        2       110.3      58.0      551.4      205.0          5
        3       551.4     205.0      601.3      205.0          5
        4       110.3      58.0      264.4       55.0          4
        5       264.4      55.0      383.6       57.3          4
        6       383.6      57.3      413.7       67.3          4
        7       413.7      67.3      601.3       63.6          4
        8          .0      79.8       87.1       56.4          3
        9        87.1      56.4      264.4       53.0          3
       10       264.4      53.0      383.9       55.3          3
       11       383.9      55.3      414.0       65.3          3
       12       414.0      65.3      601.3       61.6          3
       13          .0      77.7       86.9       54.4          1
       14        86.9      54.4      264.4       51.0          1
       15       264.4      51.0      384.3       53.3          1
       16       384.3      53.3      414.3       63.3          1
       17       414.3      63.3      601.3       59.6          1
```

```
--------------------------
ISOTROPIC Soil Parameters
--------------------------

 5 Soil unit(s) specified

Soil   Unit Weight    Cohesion  Friction     Pore Pressure      Water
Unit  Moist    Sat.  Intercept    Angle   Parameter  Constant  Surface
 No.   (pcf)   (pcf)   (psf)      (deg)       Ru       (psf)      No.

  1   108.7   130.0     644.0    12.60      .000          .0       0
  2   116.0   116.0     100.0    16.00      .000          .0       0
  3   120.0   120.0     100.0    16.00      .000          .0       0
  4   116.0   116.0     100.0    16.00      .000          .0       0
  5    66.0    66.0     288.0    23.00      .000          .0       0


A critical failure surface searching method, using a random
technique for generating CIRCULAR surfaces has been specified.


  100 trial surfaces will be generated and analyzed.


  10 Surfaces initiate from each of  10 points equally spaced
along the ground surface between  x =     98.0 ft
                             and  x =    148.0 ft


Each surface terminates between    x =    520.0 ft
                            and    x =    570.0 ft


Unless further limitations were imposed, the minimum elevation
at which a surface extends is  y =       .0 ft


* * * * *  DEFAULT SEGMENT LENGTH SELECTED BY XSTABL  * * * * *

   15.0 ft line segments define each trial failure surface.


---------------------
ANGULAR RESTRICTIONS
---------------------

  The first segment of each failure surface will be inclined
  within the angular range defined by :

          Lower angular limit :=   -45.0 degrees
          Upper angular limit :=  (slope angle - 5.0) degrees



Factors of safety have been calculated by the :

* * * * *   SIMPLIFIED BISHOP METHOD   * * * * *
```

```
The most critical circular failure surface
is specified by 35 coordinate points

      Point       x-surf      y-surf
       No.         (ft)        (ft)

        1         125.78      64.16
        2         139.65      58.45
        3         153.75      53.33
        4         168.05      48.81
        5         182.53      44.90
        6         197.17      41.62
        7         211.93      38.96
        8         226.79      36.92
        9         241.73      35.52
       10         256.71      34.76
       11         271.71      34.63
       12         286.70      35.14
       13         301.65      36.29
       14         316.55      38.07
       15         331.35      40.48
       16         346.04      43.51
       17         360.59      47.17
       18         374.97      51.44
       19         389.15      56.32
       20         403.12      61.80
       21         416.84      67.87
       22         430.28      74.51
       23         443.44      81.72
       24         456.28      89.48
       25         468.77      97.77
       26         480.90     106.60
       27         492.65     115.92
       28         503.99     125.74
       29         514.90     136.03
       30         525.37     146.78
       31         535.37     157.96
       32         544.89     169.55
       33         553.90     181.54
       34         562.40     193.90
       35         569.98     206.00

****  Simplified BISHOP FOS =   1.627   ****



The following is a summary of the TEN most critical surfaces

Problem Description : Hardin Interim Slope (3:1) Effective

        FOS      Circle Center     Radius   Initial  Terminal  Resisting
      (BISHOP)  x-coord  y-coord             x-coord  x-coord    Moment
                  (ft)     (ft)      (ft)     (ft)     (ft)      (ft-lb)

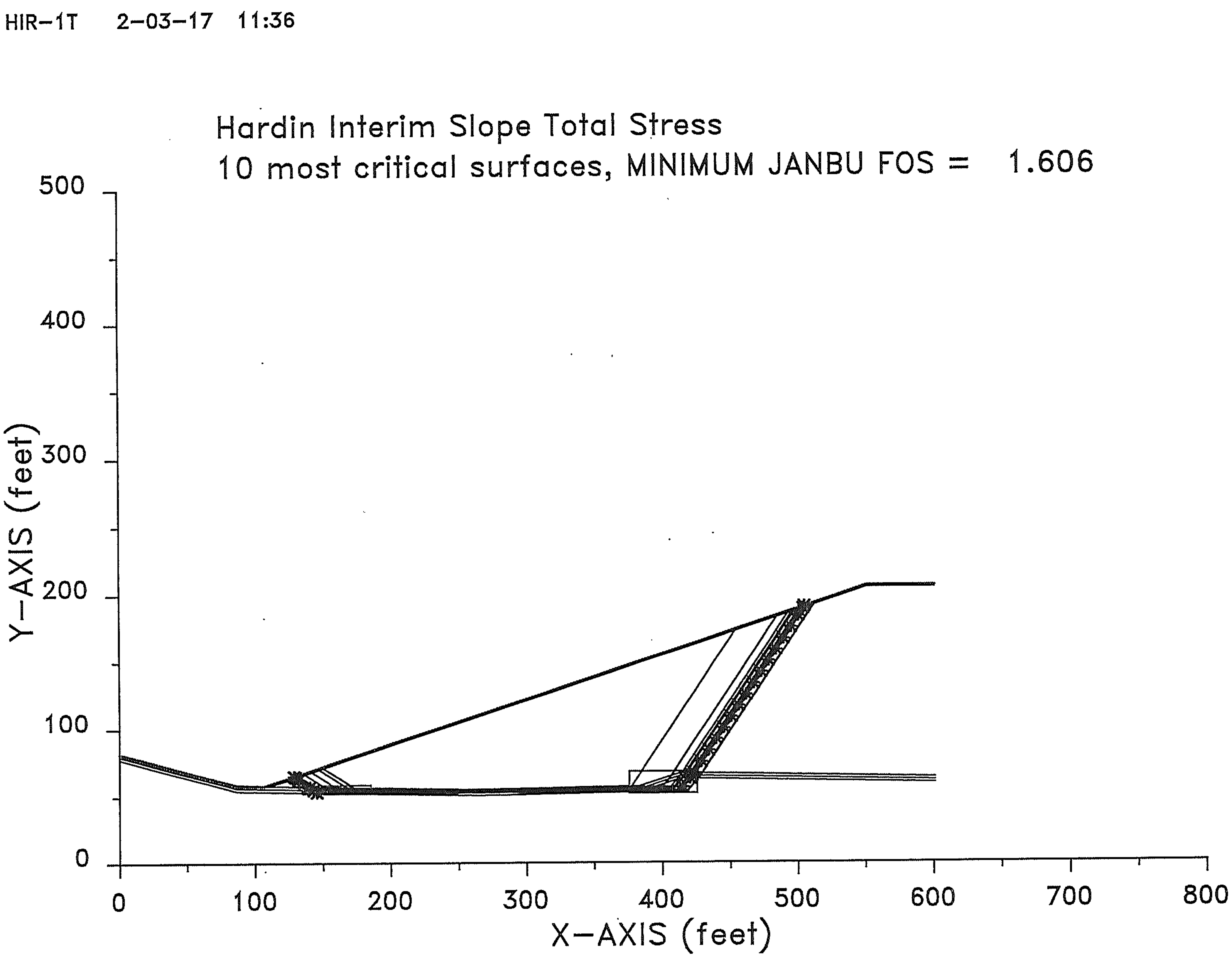
 1.    1.627    267.21   387.76    353.15   125.78   569.98   3.005E+08
 2.    1.642    187.80   606.86    553.87   109.11   569.94   3.503E+08
 3.    1.648    254.56   390.64    365.25   103.56   569.63   3.479E+08
 4.    1.652    264.70   372.36    346.11   114.67   568.21   3.245E+08
```

```
  5.   1.659    247.53   459.37   406.84   136.89   565.79  2.781E+08
  6.   1.665    278.18   351.73   325.46   125.78   569.10  3.061E+08
  7.   1.667    263.92   378.55   343.41   125.78   560.73  2.860E+08
  8.   1.668    283.55   345.57   314.05   136.89   564.79  2.746E+08
  9.   1.670    237.01   472.61   426.60   120.22   569.95  3.240E+08
 10.   1.671    255.74   364.23   341.89   103.56   558.74  3.251E+08

                   * * *  END OF FILE  * * *
```

SECTION E2-7 (FIGURE IIIE-A-4)

ITERIM WASTE SLOPE (3H:1V) STABILITY ANALYSIS
RANKINE BLOCK METHOD
TOTAL AND EFFECTIVE STRESS MODELS


HIR-1T 2-03-17 11:36
Hardin Interim Slope Total Stress
10 most critical surfaces, MINIMUM JANBU FOS = 1.606
Y-AXIS (feet)
500
400
300
200
100
0
X-AXIS (feet)
0
100
200
300
400
500
600
700
800

```
XSTABL File: HIR-1T     2-03-17    11:36

            ******************************************
            *             X S T A B L                *
            *                                        *
            *        Slope Stability Analysis        *
            *              using the                 *
            *           Method of Slices             *
            *                                        *
            *        Copyright (C) 1992 - 2008       *
            *    Interactive Software Designs, Inc.  *
            *        Moscow, ID 83843, U.S.A.        *
            *                                        *
            *          All Rights Reserved           *
            *                                        *
            *  Ver. 5.208                96 - 2046   *
            ******************************************


     Problem Description : Hardin Interim Slope Total Stress



     -----------------------------
     SEGMENT BOUNDARY COORDINATES
     -----------------------------

         4 SURFACE boundary segments

     Segment    x-left    y-left    x-right    y-right     Soil Unit
       No.       (ft)      (ft)       (ft)       (ft)    Below Segment

        1          .0      81.8       87.4       58.4          4
        2        87.4      58.4      107.3       58.0          4
        3       107.3      58.0      551.3      206.0          2
        4       551.3     206.0      601.3      206.0          2

        17 SUBSURFACE boundary segments

     Segment    x-left    y-left    x-right    y-right     Soil Unit
       No.       (ft)      (ft)       (ft)       (ft)    Below Segment

        1       107.3      58.0      110.3       58.0          4
        2       110.3      58.0      551.4      205.0          5
        3       551.4     205.0      601.3      205.0          5
        4       110.3      58.0      264.4       55.0          4
        5       264.4      55.0      383.6       57.3          4
        6       383.6      57.3      413.7       67.3          4
        7       413.7      67.3      601.3       63.6          4
        8          .0      79.8       87.1       56.4          3
        9        87.1      56.4      264.4       53.0          3
       10       264.4      53.0      383.9       55.3          3
       11       383.9      55.3      414.0       65.3          3
       12       414.0      65.3      601.3       61.6          3
       13          .0      77.7       86.9       54.4          1
       14        86.9      54.4      264.4       51.0          1
       15       264.4      51.0      384.3       53.3          1
       16       384.3      53.3      414.3       63.3          1
       17       414.3      63.3      601.3       59.6          1
```

```
--------------------------
ISOTROPIC Soil Parameters
--------------------------

 5 Soil unit(s) specified

Soil   Unit Weight    Cohesion  Friction    Pore Pressure        Water
Unit  Moist     Sat.  Intercept   Angle   Parameter  Constant   Surface
 No.  (pcf)    (pcf)    (psf)     (deg)       Ru       (psf)      No.

  1   108.7    130.0     718.0    11.40      .000         .0        0
  2   116.0    116.0     100.0    16.00      .000         .0        0
  3   120.0    120.0     100.0    16.00      .000         .0        0
  4   116.0    116.0     100.0    16.00      .000         .0        0
  5    66.0     66.0     288.0    23.00      .000         .0        0


A critical failure surface searching method, using a random
technique for generating sliding BLOCK surfaces, has been
specified.


The active and passive portions of the sliding surfaces
are generated according to the Rankine theory.


  100 trial surfaces will be generated and analyzed.


  2 boxes specified for generation of central block base


Length of line segments for active and passive portions of
sliding block is     10.0 ft


Box          x-left     y-left    x-right    y-right      Width
no.           (ft)       (ft)       (ft)       (ft)        (ft)

 1           135.1       55.5      185.1       55.5         6.0
 2           375.7       60.3      425.7       60.3        16.0

***********************************************************************
   -- WARNING  --  WARNING  --  WARNING  --  WARNING  --  (# 48)
***********************************************************************
Negative effective stresses were calculated at the base of a slice.
This warning is usually reported for cases where slices have low self
weight and a relatively high "c" shear strength parameter.  In such
cases, this effect can only be eliminated by reducing the "c" value.
***********************************************************************

 -----------------------------------------------------------
 USER SELECTED option to maintain strength greater than zero
 -----------------------------------------------------------


 Factors of safety have been calculated by the :

 * * * * *   SIMPLIFIED JANBU METHOD   * * * * *
```

```
The 10 most critical of all the failure surfaces examined
are displayed below - the most critical first

Failure surface No. 1 specified by 26 coordinate points

       Point      x-surf      y-surf
        No.        (ft)        (ft)

         1        128.88       65.19
         2        129.80       64.50
         3        132.17       62.93
         4        140.51       57.41
         5        143.29       55.32
         6        145.60       53.58
         7        415.17       56.72
         8        420.46       63.18
         9        421.94       65.14
        10        423.42       67.11
        11        428.94       75.45
        12        434.46       83.79
        13        439.98       92.12
        14        445.50      100.46
        15        451.02      108.80
        16        456.54      117.14
        17        462.06      125.48
        18        467.58      133.82
        19        473.10      142.16
        20        478.62      150.50
        21        484.13      158.84
        22        489.65      167.17
        23        495.17      175.51
        24        500.69      183.85
        25        504.30      189.30
        26        505.34      190.68

  **  Corrected JANBU FOS =   1.606  **    (Fo factor = 1.081)

Failure surface No. 2 specified by 26 coordinate points

       Point      x-surf      y-surf
        No.        (ft)        (ft)

         1        129.70       65.47
         2        130.62       64.77
         3        133.44       62.91
         4        141.78       57.39
         5        144.09       55.65
         6        406.80       52.94
         7        413.13       60.68
         8        415.26       63.28
         9        416.74       65.25
        10        418.23       67.21
        11        423.74       75.55
        12        429.26       83.89
        13        434.78       92.23
        14        440.30      100.57
        15        445.82      108.91
        16        451.34      117.24
        17        456.86      125.58
        18        462.38      133.92
```

```
        19          467.90         142.26
        20          473.42         150.60
        21          478.94         158.94
        22          484.46         167.28
        23          489.98         175.62
        24          495.50         183.95
        25          497.55         187.05
        26          498.58         188.43

  **   Corrected JANBU FOS =    1.627   **     (Fo factor = 1.082)


 Failure surface No. 3 specified by 26 coordinate points

       Point        x-surf         y-surf
        No.          (ft)           (ft)

         1          125.35          64.02
         2          126.27          63.32
         3          126.70          63.04
         4          135.04          57.52
         5          137.18          55.91
         6          406.93          54.11
         7          413.26          61.85
         8          414.45          63.30
         9          415.93          65.26
        10          417.41          67.23
        11          422.93          75.57
        12          428.45          83.90
        13          433.97          92.24
        14          439.49         100.58
        15          445.01         108.92
        16          450.53         117.26
        17          456.05         125.60
        18          461.57         133.94
        19          467.08         142.28
        20          472.60         150.62
        21          478.12         158.95
        22          483.64         167.29
        23          489.16         175.63
        24          494.68         183.97
        25          496.49         186.70
        26          497.52         188.07

  **   Corrected JANBU FOS =    1.627   **     (Fo factor = 1.081)


 Failure surface No. 4 specified by 28 coordinate points

       Point        x-surf         y-surf
        No.          (ft)           (ft)

         1          131.82          66.17
         2          132.74          65.48
         3          136.72          62.84
         4          145.06          57.32
         5          147.83          55.24
         6          150.56          53.18
         7          150.87          52.92
         8          410.07          53.42
         9          416.40          61.16
        10          418.09          63.22
        11          419.57          65.19
```

```
          12        421.05      67.15
          13        426.57      75.49
          14        432.09      83.83
          15        437.61      92.17
          16        443.13     100.51
          17        448.65     108.85
          18        454.17     117.19
          19        459.69     125.53
          20        465.21     133.87
          21        470.73     142.20
          22        476.25     150.54
          23        481.77     158.88
          24        487.29     167.22
          25        492.81     175.56
          26        498.33     183.90
          27        501.22     188.28
          28        502.26     189.65

   **  Corrected JANBU FOS =   1.644  **    (Fo factor = 1.082)


  Failure surface No. 5 specified by 27 coordinate points

         Point      x-surf      y-surf
          No.        (ft)        (ft)

           1        130.67      65.79
           2        131.60      65.10
           3        134.95      62.88
           4        143.29      57.36
           5        146.06      55.27
           6        148.79      53.21
           7        149.40      52.72
           8        405.35      55.40
           9        410.89      62.16
          10        413.01      64.97
          11        414.75      67.28
          12        420.27      75.62
          13        425.79      83.96
          14        431.31      92.30
          15        436.82     100.63
          16        442.34     108.97
          17        447.86     117.31
          18        453.38     125.65
          19        458.90     133.99
          20        464.42     142.33
          21        469.94     150.67
          22        475.46     159.01
          23        480.98     167.35
          24        486.50     175.68
          25        492.02     184.02
          26        493.03     185.55
          27        494.06     186.92

   **  Corrected JANBU FOS =   1.644  **    (Fo factor = 1.082)


  Failure surface No. 6 specified by 28 coordinate points

         Point      x-surf      y-surf
          No.        (ft)        (ft)

           1        146.52      71.07
```

```
            2         147.44        70.38
            3         151.15        67.92
            4         159.49        62.40
            5         167.83        56.88
            6         170.59        54.80
            7         173.20        52.83
            8         419.62        54.44
            9         425.96        62.17
           10         426.68        63.06
           11         428.16        65.02
           12         429.64        66.99
           13         435.16        75.32
           14         440.68        83.66
           15         446.20        92.00
           16         451.72       100.34
           17         457.24       108.68
           18         462.75       117.02
           19         468.27       125.36
           20         473.79       133.70
           21         479.31       142.04
           22         484.83       150.37
           23         490.35       158.71
           24         495.87       167.05
           25         501.39       175.39
           26         506.91       183.73
           27         512.38       192.00
           28         513.42       193.37

  **  Corrected JANBU FOS =   1.647  **    (Fo factor = 1.083)


 Failure surface No. 7 specified by 26 coordinate points

          Point       x-surf       y-surf
           No.         (ft)         (ft)

            1         135.89        67.53
            2         136.81        66.83
            3         143.03        62.72
            4         151.37        57.20
            5         154.13        55.11
            6         154.88        54.55
            7         408.24        55.91
            8         414.30        63.30
            9         415.78        65.26
           10         417.26        67.23
           11         422.78        75.57
           12         428.30        83.91
           13         433.82        92.25
           14         439.33       100.59
           15         444.85       108.92
           16         450.37       117.26
           17         455.89       125.60
           18         461.41       133.94
           19         466.93       142.28
           20         472.45       150.62
           21         477.97       158.96
           22         483.49       167.30
           23         489.01       175.63
           24         494.53       183.97
           25         496.29       186.63
           26         497.33       188.01
```

```
 **  Corrected JANBU FOS =    1.654  **     (Fo factor = 1.082)

Failure surface No. 8 specified by 28 coordinate points

         Point        x-surf        y-surf
          No.          (ft)          (ft)

           1          149.77         72.16
           2          150.69         71.46
           3          156.19         67.82
           4          164.53         62.30
           5          172.87         56.78
           6          175.63         54.70
           7          176.20         54.27
           8          418.36         53.35
           9          424.69         61.09
          10          426.31         63.06
          11          427.79         65.03
          12          429.27         66.99
          13          434.79         75.33
          14          440.31         83.67
          15          445.83         92.01
          16          451.35        100.35
          17          456.87        108.69
          18          462.39        117.03
          19          467.91        125.36
          20          473.43        133.70
          21          478.95        142.04
          22          484.47        150.38
          23          489.98        158.72
          24          495.50        167.06
          25          501.02        175.40
          26          506.54        183.74
          27          511.90        191.84
          28          512.94        193.21

 **  Corrected JANBU FOS =    1.657  **     (Fo factor = 1.084)

Failure surface No. 9 specified by 27 coordinate points

         Point        x-surf        y-surf
          No.          (ft)          (ft)

           1          141.07         69.26
           2          142.00         68.56
           3          142.72         68.08
           4          151.06         62.56
           5          159.40         57.04
           6          162.17         54.96
           7          162.70         54.56
           8          397.82         53.65
           9          402.49         59.36
          10          404.62         62.18
          11          406.73         64.98
          12          412.25         73.32
          13          417.77         81.66
          14          423.29         90.00
          15          428.81         98.34
          16          434.33        106.68
          17          439.84        115.02
          18          445.36        123.36
```

```
        19          450.88        131.69
        20          456.40        140.03
        21          461.92        148.37
        22          467.44        156.71
        23          472.96        165.05
        24          478.48        173.39
        25          484.00        181.73
        26          484.69        182.77
        27          485.72        184.14

  **  Corrected JANBU FOS =    1.666  **     (Fo factor = 1.083)


Failure surface No.10 specified by 23 coordinate points

        Point       x-surf        y-surf
         No.         (ft)          (ft)

          1         122.86         63.19
          2         123.78         62.49
          3         131.19         57.59
          4         133.96         55.50
          5         136.69         53.45
          6         136.78         53.37
          7         376.16         55.40
          8         377.50         57.18
          9         383.02         65.52
         10         388.54         73.86
         11         394.06         82.20
         12         399.58         90.54
         13         405.10         98.88
         14         410.62        107.22
         15         416.14        115.55
         16         421.66        123.89
         17         427.18        132.23
         18         432.70        140.57
         19         438.22        148.91
         20         443.74        157.25
         21         449.25        165.59
         22         453.82        172.48
         23         454.85        173.85

  **  Corrected JANBU FOS =    1.688  **     (Fo factor = 1.081)



The following is a summary of the TEN most critical surfaces

Problem Description : Hardin Interim Slope Total Stress

        Modified      Correction   Initial     Terminal    Available
       JANBU FOS        Factor     x-coord      x-coord     Strength
                                    (ft)         (ft)        (lb)

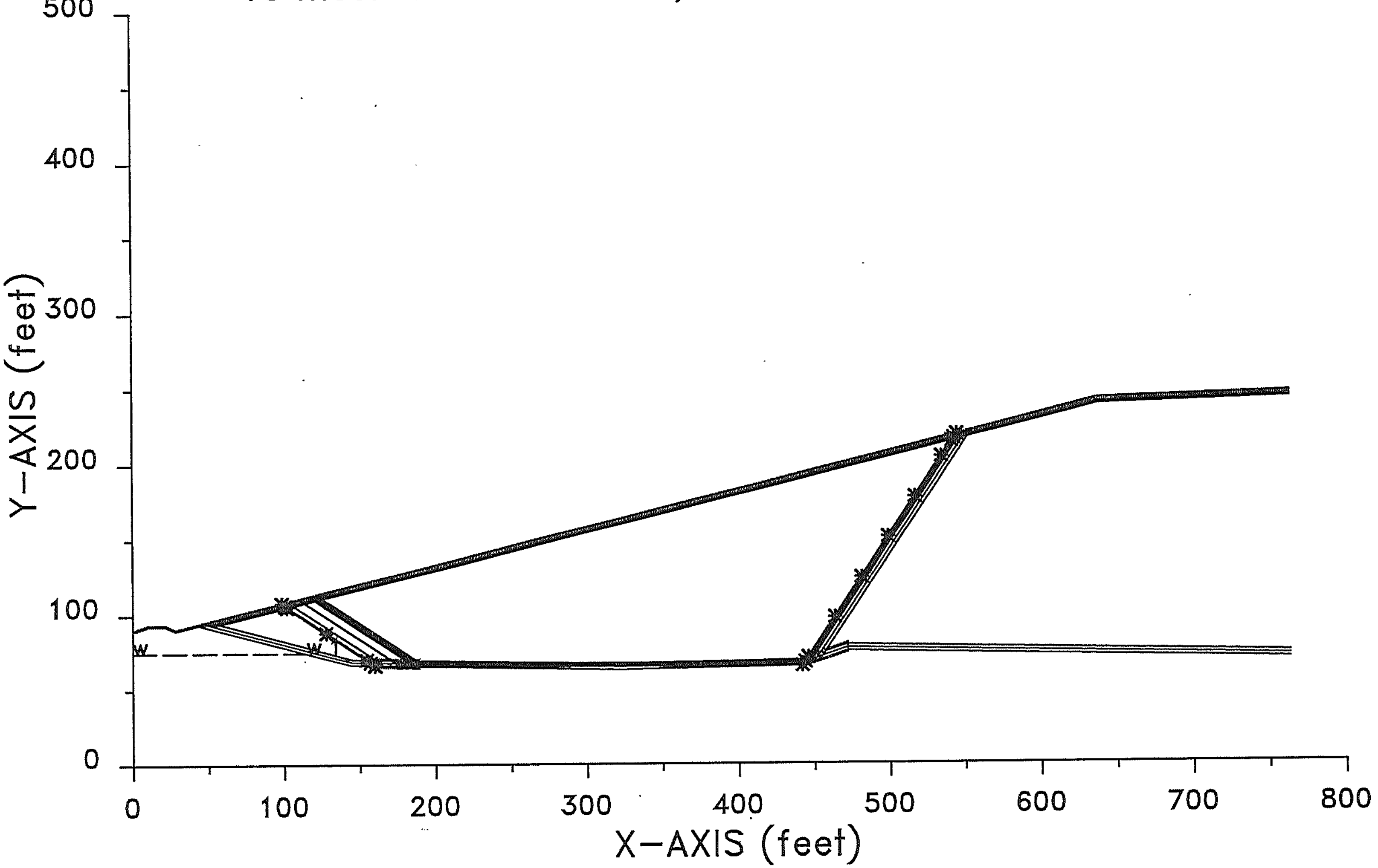
  1.      1.606          1.081      128.88      505.34      5.517E+05
  2.      1.627          1.082      129.70      498.58      5.450E+05
  3.      1.627          1.081      125.35      497.52      5.402E+05
  4.      1.644          1.082      131.82      502.26      5.703E+05
  5.      1.644          1.082      130.67      494.06      5.416E+05
  6.      1.647          1.083      146.52      513.42      5.866E+05
  7.      1.654          1.082      135.89      497.33      5.418E+05
```

```
   8.     1.657          1.084      149.77       512.94     5.839E+05
   9.     1.666          1.083      141.07       485.72     5.127E+05
  10.     1.688          1.081      122.86       454.85     4.441E+05

                      * * *  END OF FILE  * * *
```

HFR-1E 2-03-17 14:50

Hardin Final Cover Rankine Effective
10 most critical surfaces, MINIMUM JANBU FOS = 2.210



```
XSTABL File: HFR-1E      2-03-17   14:50

          ******************************************
          *              X S T A B L               *
          *                                        *
          *        Slope Stability Analysis        *
          *              using the                 *
          *           Method of Slices             *
          *                                        *
          *        Copyright (C) 1992 - 2008       *
          *   Interactive Software Designs, Inc.   *
          *        Moscow, ID 83843, U.S.A.        *
          *                                        *
          *          All Rights Reserved           *
          *                                        *
          *  Ver. 5.208                96 - 2046   *
          ******************************************

    Problem Description : Hardin Final Cover Rankine Effective


    -----------------------------
    SEGMENT BOUNDARY COORDINATES
    -----------------------------

       7 SURFACE boundary segments

    Segment    x-left     y-left     x-right    y-right      Soil Unit
      No.       (ft)       (ft)       (ft)       (ft)     Below Segment

       1          .0       90.0         9.9       93.3          1
       2         9.9       93.3        21.9       93.3          1
       3        21.9       93.3        27.9       90.3          1
       4        27.9       90.3        42.8       94.0          1
       5        42.8       94.0        44.0       94.3          3
       6        44.0       94.3       637.0      242.3          4
       7       637.0      242.3       762.8      246.3          4

      22 SUBSURFACE boundary segments

    Segment    x-left     y-left     x-right    y-right      Soil Unit
      No.       (ft)       (ft)       (ft)       (ft)     Below Segment

       1        43.5       94.3        49.4       94.3          3
       2        49.4       94.3        52.3       94.3          4
       3        52.3       94.3       637.3      240.3          3
       4       637.3      240.3       762.9      244.3          3
       5        52.3       94.3        57.8       94.3          4
       6        57.8       94.3       637.5      238.8          5
       7       637.5      238.8       762.9      242.8          5
       8        57.8       94.3       145.2       70.7          4
       9       145.2       70.7       322.1       67.3          4
      10       322.1       67.3       441.4       69.6          4
      11       441.4       69.6       471.4       79.6          4
      12       471.4       79.6       762.8       73.9          4
      13        49.4       94.3       144.9       68.8          3
      14       144.9       68.8       322.1       65.3          3
      15       322.1       65.3       441.7       67.6          3
      16       441.7       67.6       471.8       77.6          3
```

```
  17       471.8      77.6      762.8      71.9        3
  18        42.8      94.0      144.7      66.8        1
  19       144.7      66.8      322.1      63.3        1
  20       322.1      63.3      442.1      65.6        1
  21       442.1      65.6      472.1      75.6        1
  22       472.1      75.6      762.7      69.9        1


--------------------------
ISOTROPIC Soil Parameters
--------------------------

 5 Soil unit(s) specified

Soil   Unit Weight   Cohesion  Friction   Pore Pressure      Water
Unit  Moist    Sat.  Intercept  Angle   Parameter Constant  Surface
 No.  (pcf)   (pcf)    (psf)    (deg)      Ru       (psf)     No.

  1   108.7   130.0    644.0    12.60     .000        .0        1
  2   116.0   116.0    100.0    16.00     .000        .0        0
  3   120.0   120.0    100.0    16.00     .000        .0        0
  4   116.0   116.0    100.0    16.00     .000        .0        0
  5    66.0    66.0    288.0    23.00     .000        .0        0


 1 Water surface(s) have been specified.

Unit weight of water =    62.40  (pcf)


Water Surface No.  1 specified by  2 coordinate points

**********************************
         PHREATIC SURFACE,
**********************************

  Point       x-water     y-water
   No.         (ft)        (ft)

    1            .00       75.00
    2         115.00       75.00


A critical failure surface searching method, using a random
technique for generating sliding BLOCK surfaces, has been
specified.


The active and passive portions of the sliding surfaces
are generated according to the Rankine theory.


  100 trial surfaces will be generated and analyzed.


  2 boxes specified for generation of central block base


 * * * * *  DEFAULT SEGMENT LENGTH SELECTED BY XSTABL  * * * * *
```

```
Length of line segments for active and passive portions of
sliding block is     32.0 ft

Box         x-left     y-left    x-right    y-right      Width
no.          (ft)       (ft)       (ft)       (ft)        (ft)

 1          159.0       68.5      189.0       67.9         6.0
 2          441.7       67.6      471.8       77.6         6.0


Factors of safety have been calculated by the :

* * * * *   SIMPLIFIED JANBU METHOD   * * * * *


The 10 most critical of all the failure surfaces examined
are displayed below - the most critical first

Failure surface No. 1 specified by 19 coordinate points

       Point      x-surf      y-surf
        No.        (ft)        (ft)

         1         98.49      107.90
         2        100.56      106.34
         3        101.94      105.30
         4        127.80       88.18
         5        154.49       70.52
         6        157.09       68.56
         7        159.82       66.50
         8        160.01       66.35
         9        441.98       65.86
        10        443.82       68.31
        11        445.94       71.11
        12        463.60       97.80
        13        481.26      124.48
        14        498.93      151.17
        15        516.59      177.85
        16        534.25      204.53
        17        541.01      214.75
        18        542.42      216.62
        19        544.35      219.18

 **  Corrected JANBU FOS =    2.210  **    (Fo factor = 1.084)

Failure surface No. 2 specified by 21 coordinate points

       Point      x-surf      y-surf
        No.        (ft)        (ft)

         1        116.96      112.51
         2        119.03      110.95
         3        120.42      109.91
         4        127.32      105.34
         5        154.00       87.68
         6        180.68       70.02
         7        183.31       68.04
         8        186.04       65.98
         9        186.10       65.93
```

```
      10          451.94         68.03
      11          452.86         69.19
      12          454.99         72.02
      13          457.12         74.84
      14          474.78        101.53
      15          492.45        128.21
      16          510.11        154.89
      17          527.77        181.58
      18          545.43        208.26
      19          551.45        217.35
      20          552.86        219.23
      21          554.79        221.78

  **  Corrected JANBU FOS =   2.224  **     (Fo factor = 1.085)


 Failure surface No. 3 specified by 20 coordinate points

      Point       x-surf       y-surf
       No.         (ft)         (ft)

        1         110.20       110.82
        2         112.26       109.26
        3         113.65       108.22
        4         117.72       105.53
        5         144.40        87.86
        6         171.09        70.20
        7         173.70        68.23
        8         175.83        66.62
        9         445.37        66.42
       10         445.67        66.79
       11         447.81        69.63
       12         449.93        72.44
       13         467.59        99.13
       14         485.26       125.81
       15         502.92       152.50
       16         520.58       179.18
       17         538.24       205.87
       18         544.74       215.68
       19         546.15       217.55
       20         548.07       220.11

  **  Corrected JANBU FOS =   2.228  **     (Fo factor = 1.085)


 Failure surface No. 4 specified by 20 coordinate points

      Point       x-surf       y-surf
       No.         (ft)         (ft)

        1         116.95       112.51
        2         119.01       110.95
        3         120.40       109.90
        4         127.29       105.34
        5         153.98        87.68
        6         180.66        70.02
        7         183.29        68.04
        8         186.02        65.98
        9         186.34        65.72
       10         446.27        67.43
       11         447.97        69.68
       12         450.09        72.50
       13         467.75        99.18
```

```
     14          485.41        125.86
     15          503.07        152.55
     16          520.74        179.23
     17          538.40        205.92
     18          544.88        215.71
     19          546.29        217.59
     20          548.22        220.14

**  Corrected JANBU FOS =   2.253  **    (Fo factor = 1.085)

Failure surface No. 5 specified by 20 coordinate points

    Point       x-surf        y-surf
     No.         (ft)          (ft)

      1          117.21        112.57
      2          119.27        111.01
      3          120.66        109.97
      4          127.66        105.34
      5          154.34         87.67
      6          181.03         70.01
      7          183.65         68.03
      8          183.91         67.84
      9          442.72         65.43
     10          443.12         65.94
     11          445.27         68.78
     12          447.38         71.59
     13          465.05         98.28
     14          482.71        124.96
     15          500.37        151.65
     16          518.03        178.33
     17          535.69        205.02
     18          542.36        215.08
     19          543.77        216.96
     20          545.69        219.51

**  Corrected JANBU FOS =   2.263  **    (Fo factor = 1.085)

Failure surface No. 6 specified by 18 coordinate points

    Point       x-surf        y-surf
     No.         (ft)          (ft)

      1          119.79        113.22
      2          121.86        111.66
      3          123.25        110.61
      4          131.33        105.26
      5          158.01         87.60
      6          184.70         69.94
      7          187.04         68.17
      8          441.90         66.10
      9          443.47         68.19
     10          445.59         71.00
     11          463.25         97.68
     12          480.91        124.36
     13          498.57        151.05
     14          516.23        177.73
     15          533.90        204.42
     16          540.68        214.67
     17          542.09        216.54
     18          544.01        219.09
```

```
 **  Corrected JANBU FOS =   2.274  **    (Fo factor = 1.085)

Failure surface No. 7 specified by 18 coordinate points

       Point      x-surf      y-surf
        No.        (ft)        (ft)

         1        110.36      110.86
         2        112.42      109.31
         3        113.81      108.26
         4        117.95      105.52
         5        144.63       87.86
         6        171.32       70.20
         7        172.99       68.94
         8        442.46       66.36
         9        443.96       68.35
        10        446.07       71.16
        11        463.74       97.84
        12        481.40      124.53
        13        499.06      151.21
        14        516.72      177.90
        15        534.38      204.58
        16        541.13      214.78
        17        542.55      216.65
        18        544.47      219.21

 **  Corrected JANBU FOS =   2.301  **    (Fo factor = 1.085)

Failure surface No. 8 specified by 19 coordinate points

       Point      x-surf      y-surf
        No.        (ft)        (ft)

         1        102.85      108.99
         2        104.92      107.43
         3        106.30      106.39
         4        107.31      105.73
         5        133.99       88.06
         6        160.67       70.40
         7        163.28       68.44
         8        165.62       66.67
         9        447.39       68.71
        10        448.17       69.75
        11        450.29       72.56
        12        467.95       99.25
        13        485.62      125.93
        14        503.28      152.62
        15        520.94      179.30
        16        538.60      205.99
        17        545.07      215.76
        18        546.48      217.63
        19        548.41      220.19

 **  Corrected JANBU FOS =   2.327  **    (Fo factor = 1.084)

Failure surface No. 9 specified by 19 coordinate points

       Point      x-surf      y-surf
        No.        (ft)        (ft)
```

```
              1          117.82        112.72
              2          119.89        111.17
              3          121.27        110.12
              4          128.53        105.32
              5          155.21         87.66
              6          181.90         69.99
              7          184.52         68.02
              8          184.88         67.75
              9          449.43         68.07
             10          451.53         70.87
             11          453.66         73.69
             12          471.32        100.37
             13          488.98        127.05
             14          506.64        153.74
             15          524.31        180.42
             16          541.97        207.11
             17          548.21        216.54
             18          549.63        218.42
             19          551.55        220.97

      **  Corrected JANBU FOS =   2.377  **    (Fo factor = 1.085)

     Failure surface No.10 specified by 19 coordinate points

           Point        x-surf        y-surf
            No.          (ft)          (ft)

              1          122.01        113.77
              2          124.07        112.21
              3          125.46        111.17
              4          134.47        105.20
              5          161.15         87.54
              6          187.83         69.88
              7          188.22         69.59
              8          443.47         65.48
              9          444.10         66.27
             10          446.25         69.11
             11          448.36         71.92
             12          466.03         98.61
             13          483.69        125.29
             14          501.35        151.97
             15          519.01        178.66
             16          536.67        205.34
             17          543.27        215.31
             18          544.68        217.19
             19          546.61        219.74

      **  Corrected JANBU FOS =   2.378  **    (Fo factor = 1.086)



     The following is a summary of the TEN most critical surfaces

     Problem Description : Hardin Final Cover Rankine Effective

          Modified      Correction   Initial    Terminal    Available
         JANBU FOS        Factor     x-coord     x-coord     Strength
                                      (ft)        (ft)         (lb)
```

```
  1.     2.210         1.084       98.49       544.35     9.378E+05
  2.     2.224         1.085      116.96       554.79     9.407E+05
  3.     2.228         1.085      110.20       548.07     9.325E+05
  4.     2.253         1.085      116.95       548.22     9.296E+05
  5.     2.263         1.085      117.21       545.69     9.209E+05
  6.     2.274         1.085      119.79       544.01     9.094E+05
  7.     2.301         1.085      110.36       544.47     9.450E+05
  8.     2.327         1.084      102.85       548.41     9.956E+05
  9.     2.377         1.085      117.82       551.55     9.951E+05
 10.     2.378         1.086      122.01       546.61     9.629E+05

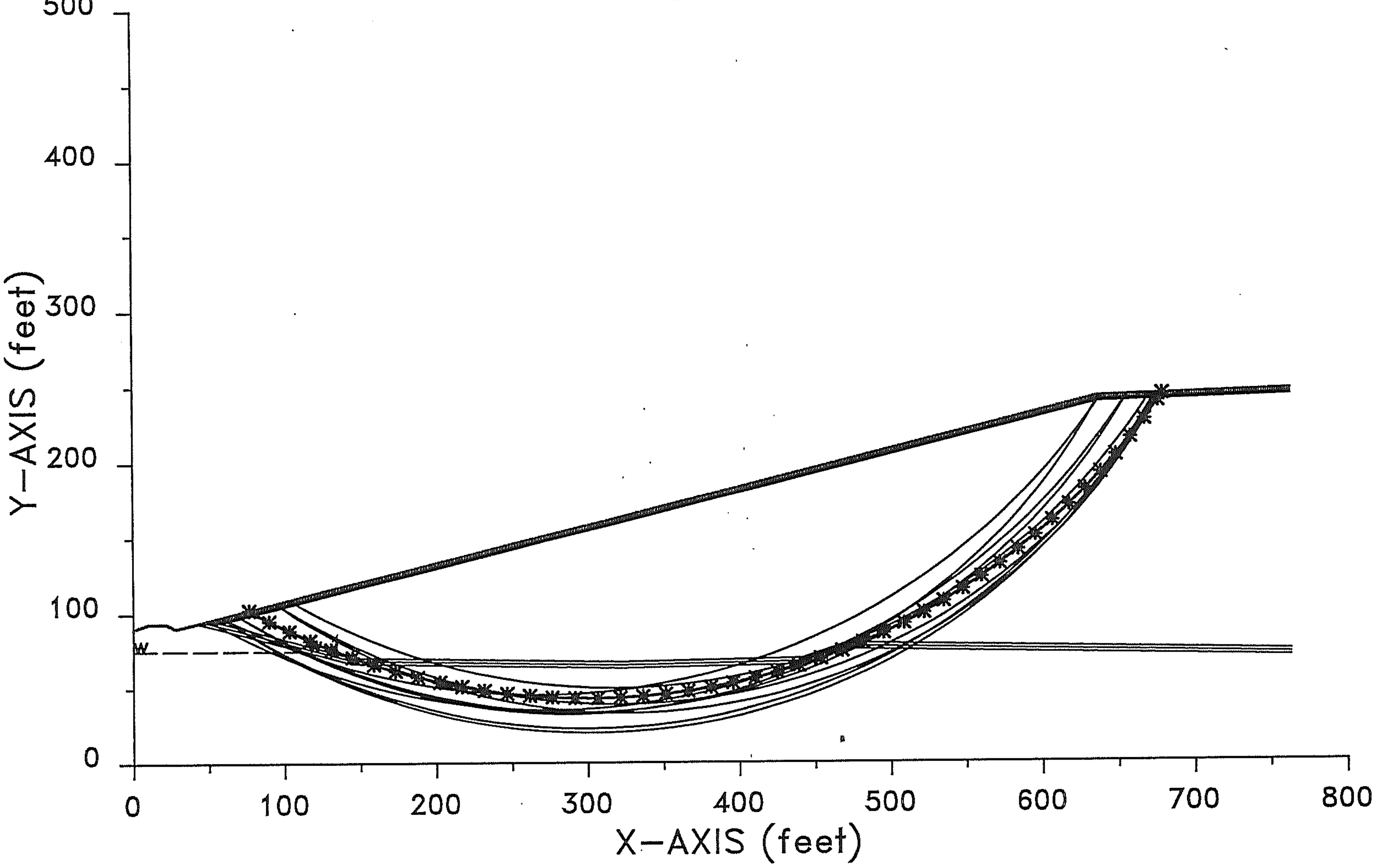
                       * * *  END OF FILE  * * *
```

SECTION E3-8 (FIGURE IIIE-A-4)

FINAL COVER SLOPE STABILITY ANALYSIS
BISHOP CIRCULAR METHOD
TOTAL AND EFFECTIVE STRESS MODELS

HFB-1T   2-03-17   14:44

Hardin Final Cover Bishop Total
10 most critical surfaces, MINIMUM BISHOP FOS = 1.975



```
XSTABL File: HFB-1T     2-03-17   14:44

          ******************************************
          *               X S T A B L              *
          *                                        *
          *        Slope Stability Analysis        *
          *               using the                *
          *            Method of Slices            *
          *                                        *
          *        Copyright (C) 1992 - 2008       *
          *     Interactive Software Designs, Inc. *
          *        Moscow, ID 83843, U.S.A.        *
          *                                        *
          *          All Rights Reserved           *
          *                                        *
          *  Ver. 5.208               96 - 2046    *
          ******************************************


    Problem Description : Hardin Final Cover Bishop Total



    -----------------------------
    SEGMENT BOUNDARY COORDINATES
    -----------------------------

        7 SURFACE boundary segments

    Segment    x-left    y-left    x-right    y-right     Soil Unit
      No.       (ft)      (ft)       (ft)       (ft)    Below Segment

      1          .0       90.0        9.9       93.3          1
      2          9.9      93.3       21.9       93.3          1
      3         21.9      93.3       27.9       90.3          1
      4         27.9      90.3       42.8       94.0          1
      5         42.8      94.0       44.0       94.3          3
      6         44.0      94.3      637.0      242.3          4
      7        637.0     242.3      762.8      246.3          4


       22 SUBSURFACE boundary segments

    Segment    x-left    y-left    x-right    y-right     Soil Unit
      No.       (ft)      (ft)       (ft)       (ft)    Below Segment

      1         43.5      94.3       49.4       94.3          3
      2         49.4      94.3       52.3       94.3          4
      3         52.3      94.3      637.3      240.3          3
      4        637.3     240.3      762.9      244.3          3
      5         52.3      94.3       57.8       94.3          4
      6         57.8      94.3      637.5      238.8          5
      7        637.5     238.8      762.9      242.8          5
      8         57.8      94.3      145.2       70.7          4
      9        145.2      70.7      322.1       67.3          4
     10        322.1      67.3      441.4       69.6          4
     11        441.4      69.6      471.4       79.6          4
     12        471.4      79.6      762.8       73.9          4
     13         49.4      94.3      144.9       68.8          3
     14        144.9      68.8      322.1       65.3          3
     15        322.1      65.3      441.7       67.6          3
     16        441.7      67.6      471.8       77.6          3
```

```
   17        471.8      77.6       762.8       71.9          3
   18         42.8      94.0       144.7       66.8          1
   19        144.7      66.8       322.1       63.3          1
   20        322.1      63.3       442.1       65.6          1
   21        442.1      65.6       472.1       75.6          1
   22        472.1      75.6       762.7       69.9          1


--------------------------
ISOTROPIC Soil Parameters
--------------------------

 5 Soil unit(s) specified

Soil   Unit Weight   Cohesion  Friction    Pore Pressure       Water
Unit  Moist    Sat.  Intercept   Angle   Parameter  Constant  Surface
 No.  (pcf)   (pcf)    (psf)     (deg)       Ru       (psf)      No.

  1   108.7   130.0      718.0    11.40      .000         .0       0
  2   116.0   116.0      100.0    16.00      .000         .0       0
  3   120.0   120.0      100.0    16.00      .000         .0       0
  4   116.0   116.0      100.0    16.00      .000         .0       0
  5    66.0    66.0      288.0    23.00      .000         .0       0


 1 Water surface(s) have been specified

Unit weight of water =     62.40  (pcf)


Water Surface No.  1 specified by  2 coordinate points

**********************************
         PHREATIC SURFACE,
**********************************

  Point       x-water      y-water
   No.          (ft)         (ft)

    1             .00        75.00
    2          115.00        75.00

-- WARNING -----------------------------------------------
     Water surface number 1 has been defined but is not
     used by any soil unit.  The analysis will IGNORE
     water surface # 1.  Please make sure that this
     assumption is consistent with your subsurface model.
----------------------------------------------------------


A critical failure surface searching method, using a random
technique for generating CIRCULAR surfaces has been specified.


  100 trial surfaces will be generated and analyzed.


  10 Surfaces initiate from each of  10 points equally spaced
along the ground surface between  x =     50.0 ft
                             and  x =    130.0 ft
```

```
Each surface terminates between   x =    600.0 ft
                            and   x =    680.0 ft


Unless further limitations were imposed, the minimum elevation
at which a surface extends is  y =       .0 ft


* * * * *  DEFAULT SEGMENT LENGTH SELECTED BY XSTABL  * * * * *

   15.0 ft line segments define each trial failure surface.


---------------------
ANGULAR RESTRICTIONS
---------------------

  The first segment of each failure surface will be inclined
  within the angular range defined by :

          Lower angular limit :=   -45.0 degrees
          Upper angular limit :=  (slope angle - 5.0) degrees



Factors of safety have been calculated by the :

* * * * *   SIMPLIFIED BISHOP METHOD   * * * * *


  The most critical circular failure surface
  is specified by 47 coordinate points

       Point       x-surf       y-surf
        No.         (ft)         (ft)

         1          76.67       102.45
         2          89.83        95.26
         3         103.23        88.51
         4         116.84        82.21
         5         130.65        76.35
         6         144.64        70.95
         7         158.81        66.02
         8         173.13        61.55
         9         187.59        57.56
        10         202.17        54.05
        11         216.86        51.02
        12         231.64        48.47
        13         246.50        46.41
        14         261.42        44.84
        15         276.38        43.76
        16         291.37        43.18
        17         306.37        43.09
        18         321.36        43.49
        19         336.33        44.39
        20         351.27        45.77
        21         366.15        47.65
```

```
         22         380.96        50.02
         23         395.69        52.87
         24         410.31        56.21
         25         424.82        60.02
         26         439.19        64.31
         27         453.42        69.08
         28         467.48        74.30
         29         481.36        79.99
         30         495.04        86.13
         31         508.52        92.72
         32         521.77        99.74
         33         534.79       107.20
         34         547.55       115.09
         35         560.04       123.39
         36         572.26       132.09
         37         584.18       141.20
         38         595.80       150.69
         39         607.09       160.55
         40         618.06       170.79
         41         628.68       181.38
         42         638.95       192.31
         43         648.86       203.58
         44         658.39       215.16
         45         667.53       227.05
         46         676.27       239.24
         47         679.22       243.64

****  Simplified BISHOP FOS =   1.975  ****



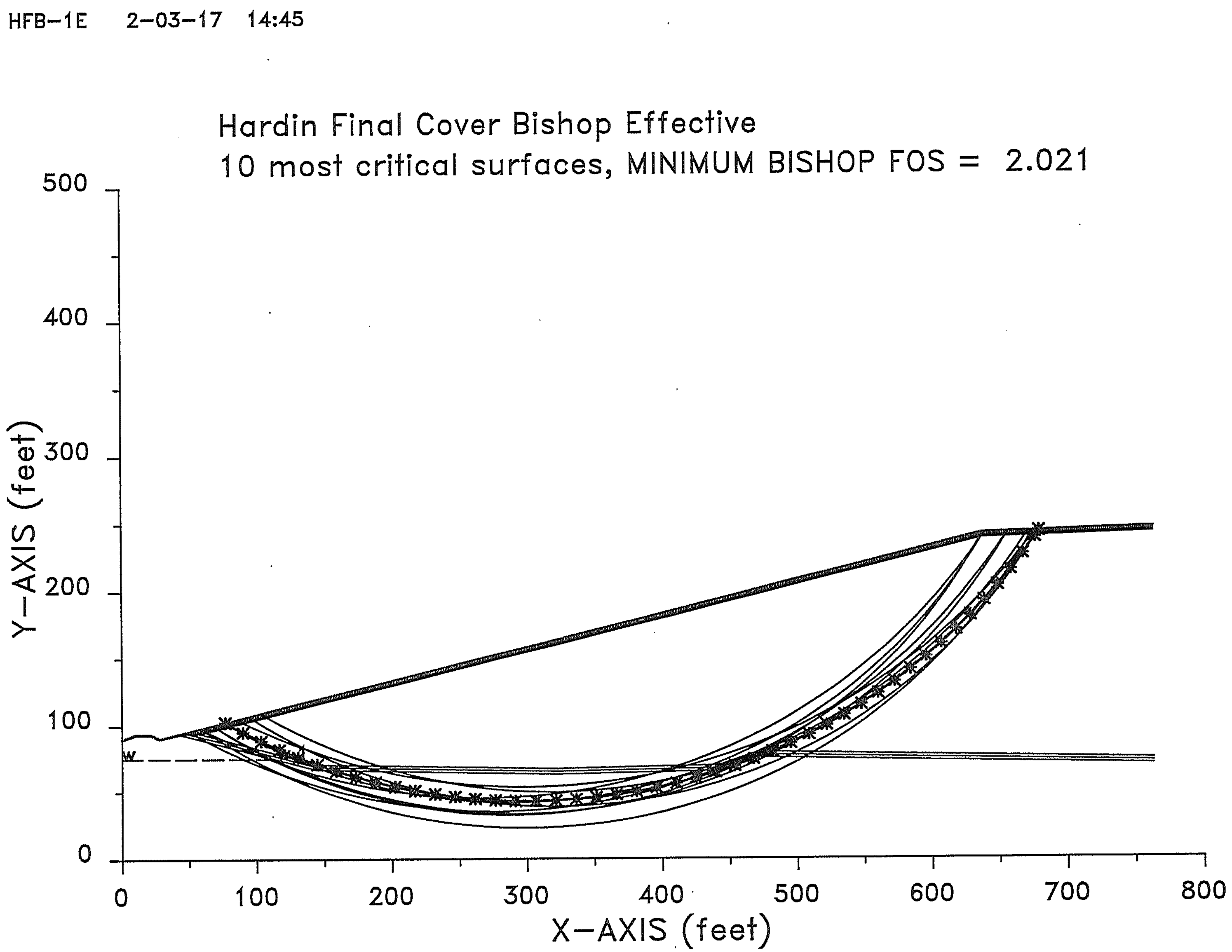

The following is a summary of the TEN most critical surfaces

Problem Description : Hardin Final Cover Bishop Total

        FOS       Circle Center    Radius   Initial  Terminal   Resisting
      (BISHOP)   x-coord  y-coord           x-coord  x-coord     Moment
                  (ft)     (ft)      (ft)     (ft)     (ft)      (ft-lb)

 1.     1.975    301.64   498.86    455.80    76.67   679.22   6.601E+08
 2.     1.990    294.12   451.90    418.21    67.78   656.33   6.114E+08
 3.     2.003    275.79   487.76    452.35    50.00   656.10   6.640E+08
 4.     2.018    300.68   449.82    426.88    58.89   674.31   7.080E+08
 5.     2.019    321.54   452.32    413.39    94.44   678.32   6.129E+08
 6.     2.021    269.05   492.33    446.82    58.89   639.35   5.896E+08
 7.     2.023    288.64   428.53    395.68    67.78   637.74   5.573E+08
 8.     2.032    303.18   441.92    421.83    58.89   675.39   7.158E+08
 9.     2.035    318.14   468.85    419.00   103.33   671.30   5.676E+08
10.     2.037    325.61   433.09    399.81    94.44   677.62   6.152E+08

                      * * *  END OF FILE  * * *
```


HFB-1E 2-03-17 14:45
Hardin Final Cover Bishop Effective
10 most critical surfaces, MINIMUM BISHOP FOS = 2.021
Y-AXIS (feet)
500
400
300
200
100
0
W
0
100
200
300
400
500
600
700
800
X-AXIS (feet)

```
XSTABL File: HFB-1E     2-03-17   14:45

          ******************************************
          *               X S T A B L              *
          *                                        *
          *         Slope Stability Analysis       *
          *                using the               *
          *             Method of Slices           *
          *                                        *
          *         Copyright (C) 1992 - 2008      *
          *    Interactive Software Designs, Inc.  *
          *         Moscow, ID 83843, U.S.A.       *
          *                                        *
          *           All Rights Reserved          *
          *                                        *
          *  Ver. 5.208                96 - 2046   *
          ******************************************


     Problem Description : Hardin Final Cover Bishop Effective



     -----------------------------
     SEGMENT BOUNDARY COORDINATES
     -----------------------------

         7 SURFACE boundary segments

     Segment   x-left   y-left   x-right   y-right    Soil Unit
       No.      (ft)     (ft)      (ft)     (ft)    Below Segment

        1         .0     90.0       9.9     93.3          1
        2        9.9     93.3      21.9     93.3          1
        3       21.9     93.3      27.9     90.3          1
        4       27.9     90.3      42.8     94.0          1
        5       42.8     94.0      44.0     94.3          3
        6       44.0     94.3     637.0    242.3          4
        7      637.0    242.3     762.8    246.3          4


        22 SUBSURFACE boundary segments

     Segment   x-left   y-left   x-right   y-right    Soil Unit
       No.      (ft)     (ft)      (ft)     (ft)    Below Segment

        1       43.5     94.3      49.4     94.3          3
        2       49.4     94.3      52.3     94.3          4
        3       52.3     94.3     637.3    240.3          3
        4      637.3    240.3     762.9    244.3          3
        5       52.3     94.3      57.8     94.3          4
        6       57.8     94.3     637.5    238.8          5
        7      637.5    238.8     762.9    242.8          5
        8       57.8     94.3     145.2     70.7          4
        9      145.2     70.7     322.1     67.3          4
       10      322.1     67.3     441.4     69.6          4
       11      441.4     69.6     471.4     79.6          4
       12      471.4     79.6     762.8     73.9          4
       13       49.4     94.3     144.9     68.8          3
       14      144.9     68.8     322.1     65.3          3
       15      322.1     65.3     441.7     67.6          3
       16      441.7     67.6     471.8     77.6          3
```

```
  17        471.8      77.6      762.8      71.9        3
  18         42.8      94.0      144.7      66.8        1
  19        144.7      66.8      322.1      63.3        1
  20        322.1      63.3      442.1      65.6        1
  21        442.1      65.6      472.1      75.6        1
  22        472.1      75.6      762.7      69.9        1


--------------------------
ISOTROPIC Soil Parameters
--------------------------

 5 Soil unit(s) specified

Soil   Unit Weight   Cohesion  Friction    Pore Pressure      Water
Unit  Moist    Sat.  Intercept   Angle   Parameter  Constant  Surface
 No.  (pcf)   (pcf)    (psf)     (deg)      Ru        (psf)     No.

  1   108.7   130.0    644.0    12.60      .000         .0       0
  2   116.0   116.0    100.0    16.00      .000         .0       0
  3   120.0   120.0    100.0    16.00      .000         .0       0
  4   116.0   116.0    100.0    16.00      .000         .0       0
  5    66.0    66.0    288.0    23.00      .000         .0       0


 1 Water surface(s) have been specified

Unit weight of water =    62.40  (pcf)


Water Surface No.  1 specified by  2 coordinate points

**********************************
         PHREATIC SURFACE,
**********************************

  Point       x-water     y-water
   No.          (ft)        (ft)

    1            .00       75.00
    2         115.00       75.00

-- WARNING ----------------------------------------------
     Water surface number 1 has been defined but is not
     used by any soil unit.  The analysis will IGNORE
     water surface # 1.  Please make sure that this
     assumption is consistent with your subsurface model.
---------------------------------------------------------


A critical failure surface searching method, using a random
technique for generating CIRCULAR surfaces has been specified.


  100 trial surfaces will be generated and analyzed.


  10 Surfaces initiate from each of  10 points equally spaced
along the ground surface between  x =    50.0 ft
                             and  x =   130.0 ft
```

```
Each surface terminates between    x =     600.0 ft
                             and    x =     680.0 ft


Unless further limitations were imposed, the minimum elevation
at which a surface extends is  y =       .0 ft


* * * * *  DEFAULT SEGMENT LENGTH SELECTED BY XSTABL  * * * * *

   15.0 ft line segments define each trial failure surface.


---------------------
ANGULAR RESTRICTIONS
---------------------

  The first segment of each failure surface will be inclined
  within the angular range defined by :

          Lower angular limit :=   -45.0 degrees
          Upper angular limit :=  (slope angle - 5.0) degrees



Factors of safety have been calculated by the :

* * * * *   SIMPLIFIED BISHOP METHOD   * * * * *


  The most critical circular failure surface
  is specified by 47 coordinate points

       Point       x-surf       y-surf
        No.         (ft)         (ft)

         1          76.67       102.45
         2          89.83        95.26
         3         103.23        88.51
         4         116.84        82.21
         5         130.65        76.35
         6         144.64        70.95
         7         158.81        66.02
         8         173.13        61.55
         9         187.59        57.56
        10         202.17        54.05
        11         216.86        51.02
        12         231.64        48.47
        13         246.50        46.41
        14         261.42        44.84
        15         276.38        43.76
        16         291.37        43.18
        17         306.37        43.09
        18         321.36        43.49
        19         336.33        44.39
        20         351.27        45.77
        21         366.15        47.65
```

```
          22          380.96         50.02
          23          395.69         52.87
          24          410.31         56.21
          25          424.82         60.02
          26          439.19         64.31
          27          453.42         69.08
          28          467.48         74.30
          29          481.36         79.99
          30          495.04         86.13
          31          508.52         92.72
          32          521.77         99.74
          33          534.79        107.20
          34          547.55        115.09
          35          560.04        123.39
          36          572.26        132.09
          37          584.18        141.20
          38          595.80        150.69
          39          607.09        160.55
          40          618.06        170.79
          41          628.68        181.38
          42          638.95        192.31
          43          648.86        203.58
          44          658.39        215.16
          45          667.53        227.05
          46          676.27        239.24
          47          679.22        243.64

****  Simplified BISHOP FOS =   2.021  ****



The following is a summary of the TEN most critical surfaces

Problem Description : Hardin Final Cover Bishop Effective

       FOS      Circle Center    Radius  Initial Terminal  Resisting
    (BISHOP)  x-coord  y-coord           x-coord  x-coord   Moment
                (ft)     (ft)     (ft)     (ft)     (ft)    (ft-lb)

 1.   2.021   301.64   498.86   455.80    76.67   679.22  6.753E+08
 2.   2.048   294.12   451.90   418.21    67.78   656.33  6.292E+08
 3.   2.052   275.79   487.76   452.35    50.00   656.10  6.804E+08
 4.   2.059   269.05   492.33   446.82    58.89   639.35  6.006E+08
 5.   2.070   318.14   468.85   419.00   103.33   671.30  5.775E+08
 6.   2.073   321.54   452.32   413.39    94.44   678.32  6.293E+08
 7.   2.081   288.64   428.53   395.68    67.78   637.74  5.733E+08
 8.   2.087   298.83   524.78   471.14    85.56   676.84  6.615E+08
 9.   2.087   300.68   449.82   426.88    58.89   674.31  7.322E+08
10.   2.094   289.65   446.49   404.63    76.67   639.00  5.573E+08

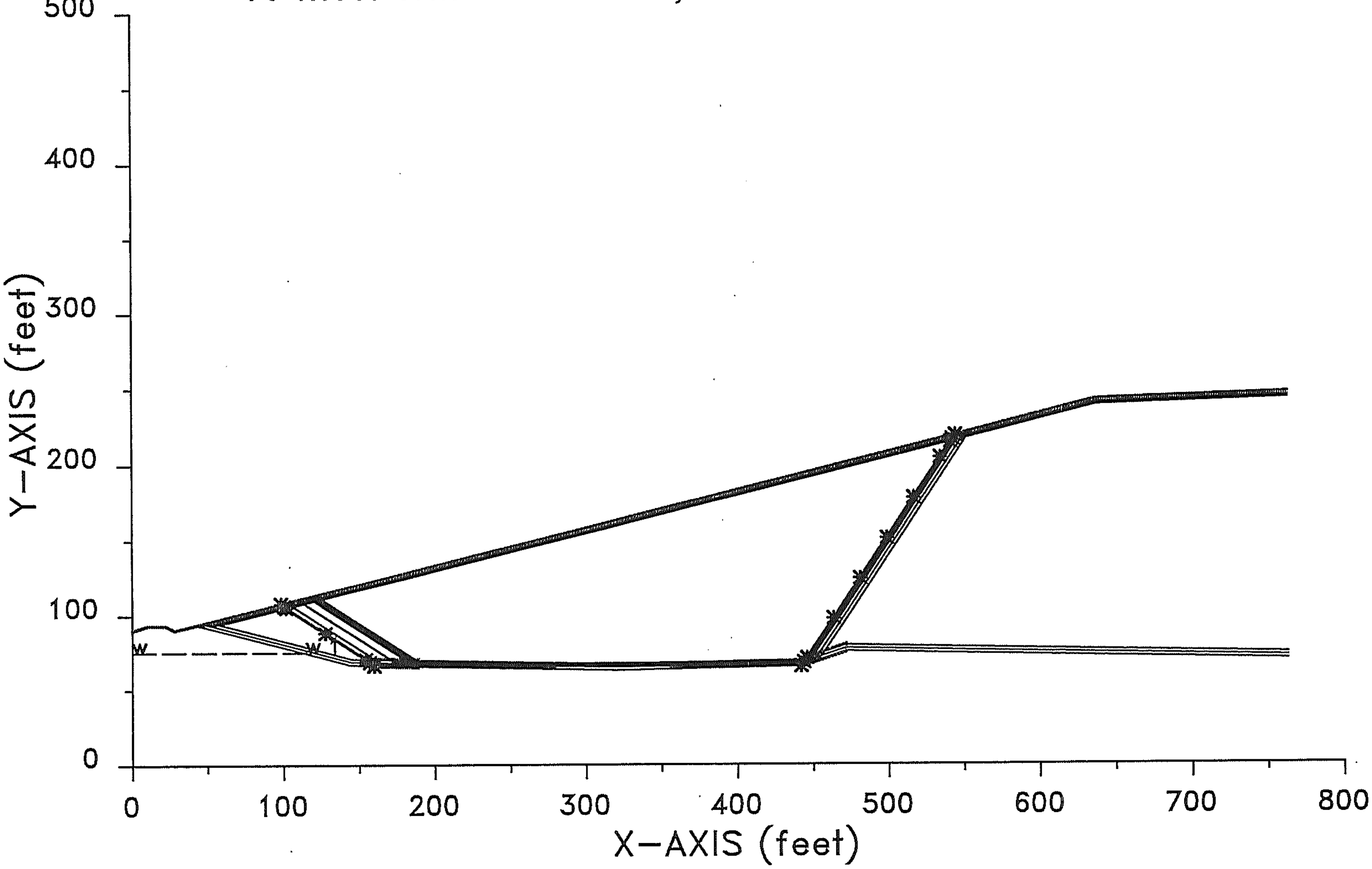
                   * * *  END OF FILE  * * *
```

SECTION E3-9 (FIGURE IIIE-A-4)

FINAL COVER SLOPE STABILITY ANALYSIS
RANKINE BLOCK METHOD
TOTAL AND EFFECTIVE STRESS MODELS

HFR-1T 2-03-17 14:52

Hardin Final Cover Rankine Total
10 most critical surfaces, MINIMUM JANBU FOS = 2.210



```
XSTABL File: HFR-1T     2-03-17   14:52

          ******************************************
          *              X S T A B L               *
          *                                        *
          *        Slope Stability Analysis        *
          *               using the                *
          *           Method of Slices             *
          *                                        *
          *        Copyright (C) 1992 - 2008       *
          *     Interactive Software Designs, Inc. *
          *        Moscow, ID 83843, U.S.A.        *
          *                                        *
          *          All Rights Reserved           *
          *                                        *
          *  Ver. 5.208                96 - 2046   *
          ******************************************


  Problem Description : Hardin Final Cover Rankine Total



  -----------------------------
  SEGMENT BOUNDARY COORDINATES
  -----------------------------

     7 SURFACE boundary segments

  Segment    x-left    y-left    x-right    y-right     Soil Unit
    No.       (ft)      (ft)       (ft)       (ft)    Below Segment

     1           .0      90.0        9.9       93.3          1
     2          9.9      93.3       21.9       93.3          1
     3         21.9      93.3       27.9       90.3          1
     4         27.9      90.3       42.8       94.0          1
     5         42.8      94.0       44.0       94.3          3
     6         44.0      94.3      637.0      242.3          4
     7        637.0     242.3      762.8      246.3          4


     22 SUBSURFACE boundary segments

  Segment    x-left    y-left    x-right    y-right     Soil Unit
    No.       (ft)      (ft)       (ft)       (ft)    Below Segment

     1         43.5      94.3       49.4       94.3          3
     2         49.4      94.3       52.3       94.3          4
     3         52.3      94.3      637.3      240.3          3
     4        637.3     240.3      762.9      244.3          3
     5         52.3      94.3       57.8       94.3          4
     6         57.8      94.3      637.5      238.8          5
     7        637.5     238.8      762.9      242.8          5
     8         57.8      94.3      145.2       70.7          4
     9        145.2      70.7      322.1       67.3          4
    10        322.1      67.3      441.4       69.6          4
    11        441.4      69.6      471.4       79.6          4
    12        471.4      79.6      762.8       73.9          4
    13         49.4      94.3      144.9       68.8          3
    14        144.9      68.8      322.1       65.3          3
    15        322.1      65.3      441.7       67.6          3
    16        441.7      67.6      471.8       77.6          3
```

```
  17        471.8       77.6       762.8       71.9        3
  18         42.8       94.0       144.7       66.8        1
  19        144.7       66.8       322.1       63.3        1
  20        322.1       63.3       442.1       65.6        1
  21        442.1       65.6       472.1       75.6        1
  22        472.1       75.6       762.7       69.9        1


--------------------------
ISOTROPIC Soil Parameters
--------------------------

 5 Soil unit(s) specified

Soil   Unit Weight   Cohesion  Friction   Pore Pressure      Water
Unit  Moist   Sat.   Intercept   Angle  Parameter Constant  Surface
 No.  (pcf)  (pcf)     (psf)     (deg)     Ru       (psf)     No.

  1   108.7  130.0     718.0     11.40     .000        .0       1
  2   116.0  116.0     100.0     16.00     .000        .0       0
  3   120.0  120.0     100.0     16.00     .000        .0       0
  4   116.0  116.0     100.0     16.00     .000        .0       0
  5    66.0   66.0     288.0     23.00     .000        .0       0


 1 Water surface(s) have been specified

Unit weight of water =    62.40  (pcf)


Water Surface No.. 1 specified by  2 coordinate points

**********************************
         PHREATIC SURFACE,
**********************************

  Point        x-water      y-water
   No.          (ft)         (ft)

    1            .00        75.00
    2         115.00        75.00


A critical failure surface searching method, using a random
technique for generating sliding BLOCK surfaces, has been
specified.


The active and passive portions of the sliding surfaces
are generated according to the Rankine theory.


  100 trial surfaces will be generated and analyzed.


  2 boxes specified for generation of central block base


* * * * *  DEFAULT SEGMENT LENGTH SELECTED BY XSTABL  * * * * *
```

```
Length of line segments for active and passive portions of
sliding block is     32.0 ft

Box          x-left      y-left     x-right     y-right        Width
no.           (ft)        (ft)        (ft)        (ft)          (ft)

 1           159.0        68.5       189.0        67.9          6.0
 2           441.7        67.6       471.8        77.6          6.0


Factors of safety have been calculated by the :

* * * * *   SIMPLIFIED JANBU METHOD   * * * * *


The 10 most critical of all the failure surfaces examined
are displayed below - the most critical first

Failure surface No. 1 specified by 19 coordinate points

       Point      x-surf      y-surf
        No.        (ft)        (ft)

         1        98.50      107.90
         2       100.56      106.34
         3       101.94      105.30
         4       127.81       88.18
         5       154.49       70.52
         6       157.10       68.56
         7       159.83       66.50
         8       160.01       66.35
         9       441.98       65.86
        10       443.82       68.31
        11       445.94       71.11
        12       463.60       97.80
        13       481.26      124.48
        14       498.93      151.17
        15       516.59      177.85
        16       534.25      204.53
        17       541.01      214.75
        18       542.42      216.62
        19       544.35      219.18

 **  Corrected JANBU FOS =   2.210  **    (Fo factor = 1.084)

Failure surface No. 2 specified by 21 coordinate points

       Point      x-surf      y-surf
        No.        (ft)        (ft)

         1       116.97      112.51
         2       119.03      110.95
         3       120.42      109.91
         4       127.32      105.34
         5       154.00       87.68
         6       180.69       70.02
         7       183.31       68.04
         8       186.04       65.98
         9       186.10       65.93
```

```
          10          451.94        68.03
          11          452.89        69.20
          12          455.02        72.03
          13          457.15        74.85
          14          474.81       101.53
          15          492.47       128.22
          16          510.14       154.90
          17          527.80       181.59
          18          545.46       208.27
          19          551.47       217.36
          20          552.89       219.23
          21          554.81       221.79

  **  Corrected JANBU FOS =    2.222   **     (Fo factor = 1.085)

 Failure surface No. 3 specified by 20 coordinate points

        Point        x-surf        y-surf
         No.          (ft)          (ft)

           1          110.20       110.82
           2          112.26       109.26
           3          113.65       108.22
           4          117.72       105.53
           5          144.40        87.86
           6          171.09        70.20
           7          173.70        68.23
           8          175.83        66.62
           9          445.37        66.42
          10          445.68        66.79
          11          447.82        69.63
          12          449.94        72.45
          13          467.60        99.13
          14          485.26       125.82
          15          502.93       152.50
          16          520.59       179.18
          17          538.25       205.87
          18          544.74       215.68
          19          546.16       217.55
          20          548.08       220.11

  **  Corrected JANBU FOS =    2.227   **     (Fo factor = 1.085)

 Failure surface No. 4 specified by 20 coordinate points

        Point        x-surf        y-surf
         No.          (ft)          (ft)

           1          116.95       112.51
           2          119.02       110.95
           3          120.41       109.91
           4          127.30       105.34
           5          153.99        87.68
           6          180.67        70.02
           7          183.29        68.04
           8          186.02        65.98
           9          186.34        65.72
          10          446.27        67.43
          11          447.97        69.68
          12          450.09        72.50
          13          467.75        99.18
```

```
        14          485.41        125.86
        15          503.07        152.55
        16          520.74        179.23
        17          538.40        205.92
        18          544.88        215.71
        19          546.29        217.59
        20          548.22        220.14

  **   Corrected JANBU FOS =    2.252   **     (Fo factor = 1.085)


 Failure surface No. 5 specified by 20 coordinate points

       Point        x-surf        y-surf
        No.          (ft)          (ft)

         1          117.21        112.57
         2          119.27        111.01
         3          120.66        109.97
         4          127.66        105.34
         5          154.34         87.67
         6          181.03         70.01
         7          183.65         68.03
         8          183.91         67.84
         9          442.72         65.43
        10          443.14         65.95
        11          445.28         68.79
        12          447.40         71.60
        13          465.06         98.28
        14          482.72        124.97
        15          500.38        151.65
        16          518.04        178.34
        17          535.71        205.02
        18          542.37        215.09
        19          543.78        216.96
        20          545.71        219.51

  **   Corrected JANBU FOS =    2.262   **     (Fo factor = 1.085)


 Failure surface No. 6 specified by 18 coordinate points

       Point        x-surf        y-surf
        No.          (ft)          (ft)

         1          119.79        113.22
         2          121.86        111.66
         3          123.25        110.61
         4          131.33        105.26
         5          158.01         87.60
         6          184.70         69.94
         7          187.04         68.17
         8          441.90         66.10
         9          443.47         68.19
        10          445.59         71.00
        11          463.25         97.68
        12          480.91        124.36
        13          498.57        151.05
        14          516.23        177.73
        15          533.90        204.42
        16          540.68        214.67
        17          542.09        216.54
        18          544.01        219.09
```

```
**  Corrected JANBU FOS =   2.274  **     (Fo factor = 1.085)

Failure surface No. 7 specified by 18 coordinate points

      Point       x-surf       y-surf
       No.         (ft)         (ft)

        1         110.36       110.86
        2         112.42       109.31
        3         113.81       108.26
        4         117.95       105.52
        5         144.63        87.86
        6         171.32        70.20
        7         172.99        68.94
        8         442.46        66.36
        9         443.96        68.35
       10         446.07        71.16
       11         463.74        97.84
       12         481.40       124.53
       13         499.06       151.21
       14         516.72       177.90
       15         534.38       204.58
       16         541.13       214.78
       17         542.55       216.65
       18         544.47       219.21

**  Corrected JANBU FOS =   2.301  **     (Fo factor = 1.085)

Failure surface No. 8 specified by 19 coordinate points

      Point       x-surf       y-surf
       No.         (ft)         (ft)

        1         102.85       108.99
        2         104.92       107.43
        3         106.30       106.39
        4         107.31       105.73
        5         133.99        88.06
        6         160.67        70.40
        7         163.28        68.44
        8         165.62        66.67
        9         447.39        68.71
       10         448.17        69.75
       11         450.29        72.56
       12         467.95        99.25
       13         485.62       125.93
       14         503.28       152.62
       15         520.94       179.30
       16         538.60       205.99
       17         545.07       215.76
       18         546.48       217.63
       19         548.41       220.19

**  Corrected JANBU FOS =   2.327  **     (Fo factor = 1.084)

Failure surface No. 9 specified by 19 coordinate points

      Point       x-surf       y-surf
       No.         (ft)         (ft)
```

```
          1         117.82        112.72
          2         119.89        111.17
          3         121.27        110.12
          4         128.53        105.32
          5         155.21         87.66
          6         181.90         69.99
          7         184.52         68.02
          8         184.88         67.75
          9         449.43         68.07
         10         451.53         70.87
         11         453.66         73.69
         12         471.32        100.37
         13         488.98        127.05
         14         506.64        153.74
         15         524.31        180.42
         16         541.97        207.11
         17         548.21        216.54
         18         549.63        218.42
         19         551.55        220.97

  **  Corrected JANBU FOS =   2.377  **    (Fo factor = 1.085)


 Failure surface No.10 specified by 19 coordinate points

       Point        x-surf        y-surf
        No.          (ft)          (ft)

          1         122.01        113.77
          2         124.07        112.21
          3         125.46        111.17
          4         134.47        105.20
          5         161.15         87.54
          6         187.83         69.88
          7         188.22         69.59
          8         443.47         65.48
          9         444.12         66.27
         10         446.26         69.12
         11         448.38         71.93
         12         466.04         98.61
         13         483.71        125.30
         14         501.37        151.98
         15         519.03        178.66
         16         536.69        205.35
         17         543.29        215.32
         18         544.70        217.19
         19         546.63        219.74

  **  Corrected JANBU FOS =   2.377  **    (Fo factor = 1.086)



 The following is a summary of the TEN most critical surfaces

 Problem Description : Hardin Final Cover Rankine Total

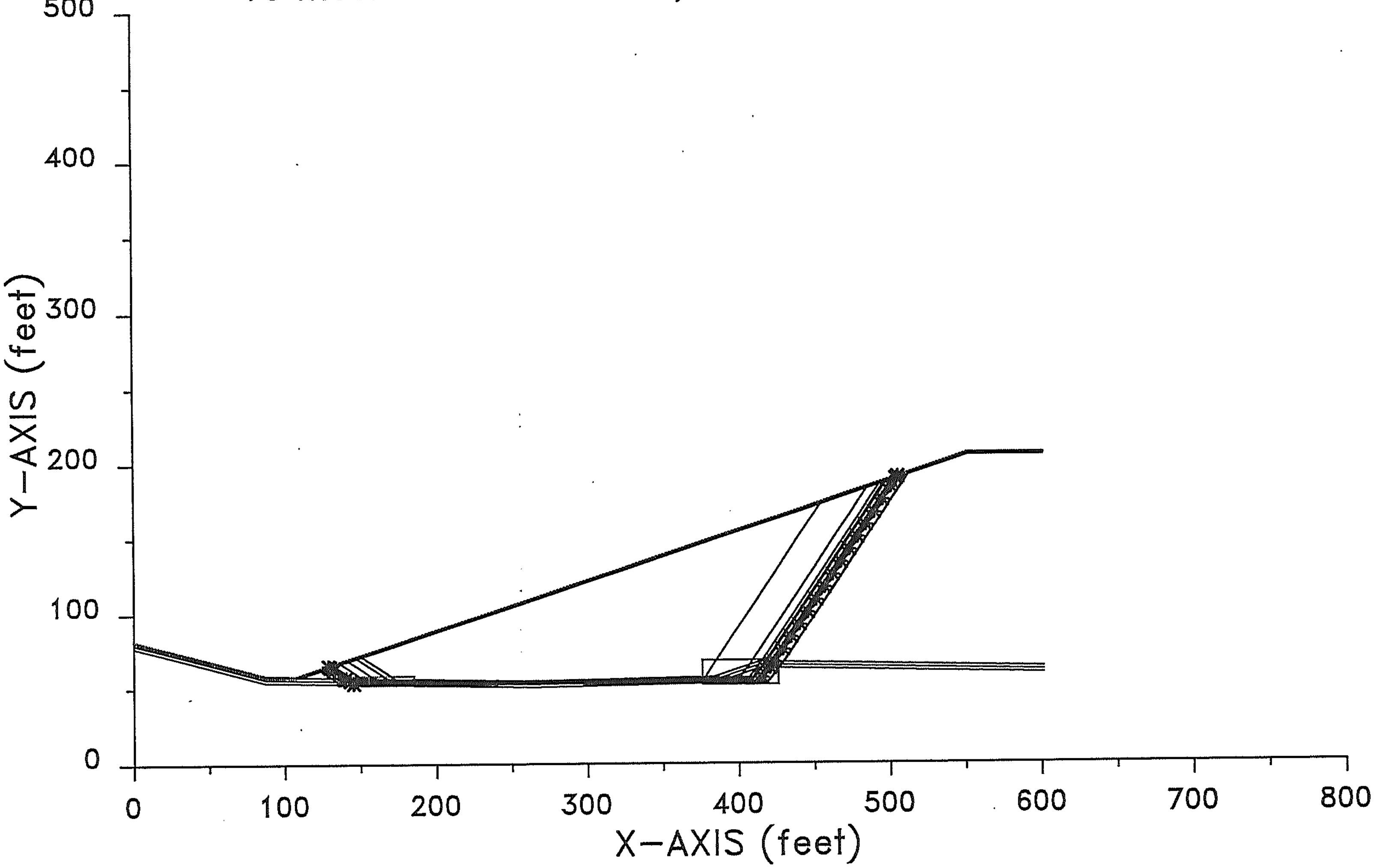
        Modified      Correction   Initial    Terminal    Available
       JANBU FOS        Factor     x-coord     x-coord     Strength
                                     (ft)        (ft)        (lb)
```

```
  1.      2.210         1.084       98.50       544.35     9.376E+05
  2.      2.222         1.085      116.97       554.81     9.402E+05
  3.      2.227         1.085      110.20       548.08     9.324E+05
  4.      2.252         1.085      116.95       548.22     9.293E+05
  5.      2.262         1.085      117.21       545.71     9.201E+05
  6.      2.274         1.085      119.79       544.01     9.094E+05
  7.      2.301         1.085      110.36       544.47     9.450E+05
  8.      2.327         1.084      102.85       548.41     9.956E+05
  9.      2.377         1.085      117.82       551.55     9.951E+05
 10.      2.377         1.086      122.01       546.63     9.623E+05

                    * * *  END OF FILE  * * *
```

HIR-1E 2-03-17 11:34

Hardin Interim Slope Effective
10 most critical surfaces, MINIMUM JANBU FOS = 1.613



```
XSTABL File: HIR-1E      2-03-17    11:34

          ******************************************
          *              X S T A B L               *
          *                                        *
          *        Slope Stability Analysis        *
          *               using the                *
          *            Method of Slices            *
          *                                        *
          *        Copyright (C) 1992 - 2008       *
          *   Interactive Software Designs, Inc.   *
          *        Moscow, ID 83843, U.S.A.        *
          *                                        *
          *          All Rights Reserved           *
          *                                        *
          *  Ver. 5.208                 96 - 2046  *
          ******************************************


    Problem Description : Hardin Interim Slope Effective



    -----------------------------
    SEGMENT BOUNDARY COORDINATES
    -----------------------------

        4 SURFACE boundary segments

    Segment    x-left    y-left    x-right   y-right    Soil Unit
      No.       (ft)      (ft)      (ft)      (ft)    Below Segment

       1          .0      81.8      87.4      58.4          4
       2        87.4      58.4     107.3      58.0          4
       3       107.3      58.0     551.3     206.0          2
       4       551.3     206.0     601.3     206.0          2

       17 SUBSURFACE boundary segments

    Segment    x-left    y-left    x-right   y-right    Soil Unit
      No.       (ft)      (ft)      (ft)      (ft)    Below Segment

       1       107.3      58.0     110.3      58.0          4
       2       110.3      58.0     551.4     205.0          5
       3       551.4     205.0     601.3     205.0          5
       4       110.3      58.0     264.4      55.0          4
       5       264.4      55.0     383.6      57.3          4
       6       383.6      57.3     413.7      67.3          4
       7       413.7      67.3     601.3      63.6          4
       8          .0      79.8      87.1      56.4          3
       9        87.1      56.4     264.4      53.0          3
      10       264.4      53.0     383.9      55.3          3
      11       383.9      55.3     414.0      65.3          3
      12       414.0      65.3     601.3      61.6          3
      13          .0      77.7      86.9      54.4          1
      14        86.9      54.4     264.4      51.0          1
      15       264.4      51.0     384.3      53.3          1
      16       384.3      53.3     414.3      63.3          1
      17       414.3      63.3     601.3      59.6          1
```

```
         --------------------------
         ISOTROPIC Soil Parameters
         --------------------------

          5 Soil unit(s) specified

         Soil   Unit Weight   Cohesion  Friction    Pore Pressure      Water
         Unit  Moist    Sat.  Intercept   Angle  Parameter Constant  Surface
          No.  (pcf)   (pcf)    (psf)     (deg)      Ru      (psf)     No.

           1   108.7   130.0     644.0    12.60     .000        .0      0
           2   116.0   116.0     100.0    16.00     .000        .0      0
           3   120.0   120.0     100.0    16.00     .000        .0      0
           4   116.0   116.0     100.0    16.00     .000        .0      0
           5    66.0    66.0     288.0    23.00     .000        .0      0


         A critical failure surface searching method, using a random
         technique for generating sliding BLOCK surfaces, has been
         specified.


         The active and passive portions of the sliding surfaces
         are generated according to the Rankine theory.


           100 trial surfaces will be generated and analyzed.


           2 boxes specified for generation of central block base


         Length of line segments for active and passive portions of
         sliding block is     10.0 ft


         Box         x-left     y-left    x-right    y-right      Width
         no.          (ft)       (ft)       (ft)       (ft)        (ft)

          1          135.1       55.5      185.1       55.5         6.0
          2          375.7       60.3      425.7       60.3        16.0

**********************************************************************
    --  WARNING  --  WARNING  --  WARNING  --  WARNING  --  (# 48)
**********************************************************************
Negative effective stresses were calculated at the base of a slice.
This warning is usually reported for cases where slices have low self
weight and a relatively high "c" shear strength parameter.  In such
cases, this effect can only be eliminated by reducing the "c" value.
**********************************************************************

          -----------------------------------------------------------
          USER SELECTED option to maintain strength greater than zero
          -----------------------------------------------------------


          Factors of safety have been calculated by the :

          * * * * *   SIMPLIFIED JANBU METHOD   * * * * *
```

```
The 10 most critical of all the failure surfaces examined
are displayed below - the most critical first

Failure surface No. 1 specified by 26 coordinate points

      Point        x-surf        y-surf
       No.          (ft)          (ft)

        1         128.88         65.19
        2         129.80         64.50
        3         132.17         62.93
        4         140.51         57.41
        5         143.29         55.32
        6         145.60         53.58
        7         415.17         56.72
        8         420.35         63.18
        9         421.83         65.15
       10         423.31         67.11
       11         428.83         75.45
       12         434.35         83.79
       13         439.87         92.13
       14         445.39        100.47
       15         450.91        108.80
       16         456.43        117.14
       17         461.95        125.48
       18         467.47        133.82
       19         472.99        142.16
       20         478.50        150.50
       21         484.02        158.84
       22         489.54        167.18
       23         495.06        175.52
       24         500.58        183.85
       25         504.16        189.26
       26         505.19        190.63

  **  Corrected JANBU FOS =   1.613  **     (Fo factor = 1.081)

Failure surface No. 2 specified by 26 coordinate points

      Point        x-surf        y-surf
       No.          (ft)          (ft)

        1         125.35         64.02
        2         126.27         63.32
        3         126.70         63.04
        4         135.04         57.52
        5         137.18         55.91
        6         406.93         54.11
        7         413.18         61.91
        8         414.29         63.30
        9         415.77         65.26
       10         417.25         67.23
       11         422.77         75.57
       12         428.29         83.91
       13         433.81         92.25
       14         439.33        100.59
       15         444.85        108.92
       16         450.37        117.26
       17         455.89        125.60
       18         461.41        133.94
```

```
        19         466.93      142.28
        20         472.45      150.62
        21         477.97      158.96
        22         483.49      167.30
        23         489.01      175.64
        24         494.53      183.97
        25         496.28      186.63
        26         497.32      188.01

  **  Corrected JANBU FOS =   1.635  **    (Fo factor = 1.081)


 Failure surface No. 3 specified by 26 coordinate points

       Point       x-surf       y-surf
        No.         (ft)         (ft)

         1         129.70        65.47
         2         130.62        64.77
         3         133.44        62.91
         4         141.78        57.39
         5         144.09        55.65
         6         406.80        52.94
         7         413.05        60.75
         8         415.09        63.28
         9         416.57        65.25
        10         418.05        67.21
        11         423.57        75.55
        12         429.09        83.89
        13         434.61        92.23
        14         440.13       100.57
        15         445.65       108.91
        16         451.17       117.25
        17         456.68       125.59
        18         462.20       133.93
        19         467.72       142.26
        20         473.24       150.60
        21         478.76       158.94
        22         484.28       167.28
        23         489.80       175.62
        24         495.32       183.96
        25         497.32       186.98
        26         498.35       188.35

  **  Corrected JANBU FOS =   1.636  **    (Fo factor = 1.082)


 Failure surface No. 4 specified by 27 coordinate points

       Point       x-surf       y-surf
        No.         (ft)         (ft)

         1         130.67        65.79
         2         131.59        65.09
         3         134.94        62.88
         4         143.27        57.36
         5         146.05        55.27
         6         148.77        53.21
         7         149.40        52.72
         8         405.35        55.40
         9         410.73        62.11
        10         412.85        64.92
        11         414.63        67.28
```

```
          12         420.15        75.62
          13         425.67        83.96
          14         431.19        92.30
          15         436.71       100.64
          16         442.23       108.98
          17         447.75       117.31
          18         453.26       125.65
          19         458.78       133.99
          20         464.30       142.33
          21         469.82       150.67
          22         475.34       159.01
          23         480.86       167.35
          24         486.38       175.69
          25         491.90       184.03
          26         492.87       185.50
          27         493.91       186.87

 **  Corrected JANBU FOS =   1.648  **    (Fo factor = 1.082)


Failure surface No. 5 specified by 28 coordinate points

        Point        x-surf       y-surf
         No.          (ft)         (ft)

          1          131.81        66.17
          2          132.73        65.48
          3          136.71        62.84
          4          145.05        57.32
          5          147.82        55.24
          6          150.55        53.18
          7          150.87        52.92
          8          410.07        53.42
          9          416.32        61.23
         10          417.93        63.23
         11          419.41        65.19
         12          420.89        67.16
         13          426.41        75.50
         14          431.93        83.84
         15          437.45        92.17
         16          442.96       100.51
         17          448.48       108.85
         18          454.00       117.19
         19          459.52       125.53
         20          465.04       133.87
         21          470.56       142.21
         22          476.08       150.55
         23          481.60       158.89
         24          487.12       167.22
         25          492.64       175.56
         26          498.16       183.90
         27          501.01       188.21
         28          502.04       189.58

 **  Corrected JANBU FOS =   1.651  **    (Fo factor = 1.082)


Failure surface No. 6 specified by 28 coordinate points

        Point        x-surf       y-surf
         No.          (ft)         (ft)

          1          146.52        71.07
```

```
          2          147.44         70.38
          3          151.15         67.92
          4          159.49         62.40
          5          167.83         56.88
          6          170.59         54.80
          7          173.20         52.83
          8          419.62         54.44
          9          425.87         62.24
         10          426.53         63.06
         11          428.01         65.02
         12          429.49         66.99
         13          435.01         75.33
         14          440.53         83.67
         15          446.05         92.01
         16          451.57        100.34
         17          457.09        108.68
         18          462.61        117.02
         19          468.13        125.36
         20          473.65        133.70
         21          479.17        142.04
         22          484.69        150.38
         23          490.20        158.72
         24          495.72        167.05
         25          501.24        175.39
         26          506.76        183.73
         27          512.19        191.93
         28          513.23        193.31

  **  Corrected JANBU FOS =   1.657  **    (Fo factor = 1.083)


 Failure surface No. 7 specified by 26 coordinate points

        Point        x-surf        y-surf
         No.          (ft)          (ft)

          1          135.89         67.53
          2          136.81         66.83
          3          143.03         62.72
          4          151.37         57.20
          5          154.13         55.11
          6          154.88         54.55
          7          408.24         55.91
          8          414.12         63.24
          9          415.65         65.27
         10          417.13         67.23
         11          422.65         75.57
         12          428.17         83.91
         13          433.69         92.25
         14          439.21        100.59
         15          444.73        108.93
         16          450.24        117.27
         17          455.76        125.60
         18          461.28        133.94
         19          466.80        142.28
         20          472.32        150.62
         21          477.84        158.96
         22          483.36        167.30
         23          488.88        175.64
         24          494.40        183.98
         25          496.12        186.58
         26          497.16        187.95
```

```
 **  Corrected JANBU FOS =   1.660  **    (Fo factor = 1.082)


Failure surface No. 8 specified by 28 coordinate points

      Point       x-surf      y-surf
       No.         (ft)        (ft)

        1        149.77       72.16
        2        150.69       71.46
        3        156.19       67.82
        4        164.53       62.30
        5        172.87       56.78
        6        175.63       54.70
        7        176.20       54.27
        8        418.36       53.35
        9        424.61       61.16
       10        426.14       63.07
       11        427.62       65.03
       12        429.11       67.00
       13        434.63       75.34
       14        440.14       83.67
       15        445.66       92.01
       16        451.18      100.35
       17        456.70      108.69
       18        462.22      117.03
       19        467.74      125.37
       20        473.26      133.71
       21        478.78      142.05
       22        484.30      150.38
       23        489.82      158.72
       24        495.34      167.06
       25        500.86      175.40
       26        506.38      183.74
       27        511.69      191.77
       28        512.73      193.14

 **  Corrected JANBU FOS =   1.668  **    (Fo factor = 1.084)


Failure surface No. 9 specified by 27 coordinate points

      Point       x-surf      y-surf
       No.         (ft)        (ft)

        1        141.07       69.26
        2        142.00       68.56
        3        142.72       68.08
        4        151.06       62.56
        5        159.40       57.04
        6        162.17       54.96
        7        162.70       54.56
        8        397.82       53.65
        9        402.36       59.32
       10        404.48       62.14
       11        406.59       64.94
       12        412.11       73.28
       13        417.63       81.62
       14        423.15       89.96
       15        428.67       98.29
       16        434.19      106.63
       17        439.71      114.97
       18        445.23      123.31
```

```
              19         450.75       131.65
              20         456.27       139.99
              21         461.79       148.33
              22         467.31       156.67
              23         472.83       165.01
              24         478.35       173.34
              25         483.86       181.68
              26         484.55       182.72
              27         485.59       184.10

       **   Corrected JANBU FOS =    1.671   **     (Fo factor = 1.083)


     Failure surface No.10 specified by 23 coordinate points

             Point        x-surf       y-surf
              No.          (ft)         (ft)

               1         122.86        63.19
               2         123.78        62.49
               3         131.19        57.59
               4         133.96        55.50
               5         136.69        53.45
               6         136.78        53.37
               7         376.16        55.40
               8         377.50        57.18
               9         383.02        65.52
              10         388.54        73.86
              11         394.06        82.20
              12         399.58        90.54
              13         405.10        98.88
              14         410.62       107.22
              15         416.14       115.55
              16         421.66       123.89
              17         427.18       132.23
              18         432.70       140.57
              19         438.22       148.91
              20         443.74       157.25
              21         449.25       165.59
              22         453.82       172.48
              23         454.85       173.85

       **   Corrected JANBU FOS =    1.688   **     (Fo factor = 1.081)



     The following is a summary of the TEN most critical surfaces

     Problem Description : Hardin Interim Slope Effective

            Modified      Correction    Initial     Terminal    Available
           JANBU FOS        Factor      x-coord      x-coord     Strength
                                         (ft)         (ft)         (lb)

      1.      1.613          1.081      128.88       505.19    5.537E+05
      2.      1.635          1.081      125.35       497.32    5.421E+05
      3.      1.636          1.082      129.70       498.35    5.476E+05
      4.      1.648          1.082      130.67       493.91    5.423E+05
      5.      1.651          1.082      131.81       502.04    5.722E+05
      6.      1.657          1.083      146.52       513.23    5.898E+05
      7.      1.660          1.082      135.89       497.16    5.433E+05
```

```
   8.     1.668         1.084      149.77      512.73     5.873E+05
   9.     1.671         1.083      141.07      485.59     5.138E+05
  10.     1.688         1.081      122.86      454.85     4.439E+05

                    * * *  END OF FILE  * * *
```

APPENDIX IIIE-A-3

INFINITE SLOPE STABILITY ANALYSIS

Includes pages IIIE-A-3-1 through IIIE-A-3-5

STATE OF TEXAS
DAVID E. POE
81734
LICENSED PROFESSIONAL ENGINEER

12-5-17

Prep By: DEP
Date: 12/5/2017

Chkd By:_______
Date:_______

HARDIN COUNTY LANDFILL
0120-758-11-02
COMPOSITE FINAL COVER SYSTEM
STABILITY ANALYSIS

Required:

1. Use infinite slope stability analysis to verify final cover system stability.

Note:

A. The Subtitle D final cover system for the site consists of an 18-inch-thick compacted clay infiltration layer with a maximum hydraulic conductivity of 1×10^{-5} cm/s, a 40-mil LLDPE flexible membrane liner, a 250-mil-thick drainage geocomposite, and a 24-inch-thick erosion layer.

B. The Type IV C&D final cover system for the site consists of an 18-inch-thick clay infiltration layer with a maximum hydraulic conductivity of 1×10^{-5} cm/sec, and a 12-inch-thick erosion layer.

Method:

1. Use Duncan and Buchignani's method for infinite stability analysis.

References:

1. Koerner, Robert M., *Designing with Geosynthetics*, 3rd Edition, Prentice-Hall Inc., 1994.
2. Duncan, J.M. and Buchignani, A. L., *An Engineering Manual for Slope Stability Studies*, Department of Civil Engineering - University of California-Berkeley, 1975.
3. TRI, Interface Friction/Direct Shear Testing & Slope Stability Issues Short Course, November 12-13, 1998, Austin, Texas.
4. US Army Corps of Engineers, *Slope Stability*, Engineering and Design Manual, EM 1110-2-1902, October 31, 2003.
5. Koerner, Robert M., *Analysis and Design of Veneer Cover Soils*, 1998 Sixth International Conference of Geosynthetics.
6. Koerner, George R. and Narejo, Dhani, *Direct Shear Database of Geosynthetic-to-Geosynthetic and Geosynthetic-to-Soil Interfaces,* GRI Report #30, June 14, 2005.

A. Subtitle D Final Cover System

1. Infinite Slope Stability Analysis of Cover System
25 Percent Sideslope (Top Layer of Final Cover Saturated)

Calculate factor of safety:

$$F.S. = A\frac{\tan\Delta}{\tan\beta} + B\frac{C_a}{\gamma H}$$

See Stability Charts for Infinite Slopes on page IIIE-A-3-6 for procedure.

where: H = thickness of material above interface (ft)

Note: Interface friction strength values are selected conservatively from laboratory testing of similar material/interfaces. Prior to construction, laboratory tests will be performed to verify the assumed values for interface adhesion and friction angle using project-specific soil and synthetic materials. The interface friction testing will be performed for large displacement (i.e., residual) shear strength. If test results differ from the assumed values, this analysis will be updated for acceptable factor of safety values using the procedures presented in the following pages.

Erosion Layer/Geocomposite Interface:

Δ =	16	deg
β =	14.04	deg
A =	0.48	
C_a =	100	psf
B =	4.3	
γ =	125	pcf
H =	2	ft

Calculate pore pressure ratio for saturated soil:

$$r_u = \frac{X}{T} \times \frac{\gamma_w}{\gamma} Cos^2\beta$$

See Stability Charts for Infinite Slopes on page IIIE-A-3-6 for procedure.

To be conservative the soil above the geocomposite is assumed to be saturated (X=T). This assumption provides maximum pore pressure ratio (r_u) as shown on page IIIE-A-4-20. Therefore the analysis is conservative.

r_u= 0.47
slope ratio, b= 4.00

Use r_u and slope ratio, b to determine parameters A and B on page IIIE-A-3-6.

F.S. = 2.3

Geocomposite/Geomembrane Interface:

Δ =	15	deg
β =	14.03	deg
A =	1	
C_a =	100	psf
B =	4.3	
γ =	125	pcf
H =	2	ft

The seepage will be entirely contained within the geocomposite. Therefore pore pressure and r_u are equal to zero and parameter A is equal to 1.

slope ratio, b= 4.00 Therefore, parameter B is equal to 4.3 according to the chart on page IIIE-A-3-6.

F.S. = 2.8

Geomembrane/Clay Liner Interface:

Δ = 16 deg
β = 14.03 deg
A = 1
C_a = 100 psf
B = 4.3
γ = 125 pcf
H = 2 ft

The seepage will be entirely contained within the geocomposite. Therefore, pore pressure and r_u are equal to zero, and parameter A is equal to 1.

slope ratio, b= 4.00 Therefore, parameter B is equal to 4.3 according to the chart on page IIIE-A-3-6.

F.S. = 2.9

2. **Infinite Slope Stability Analysis of Cover System**
5 Percent Topslope (Top Layer of Final Cover Saturated)

Erosion Layer/Geocomposite Interface:

Δ = 16 deg
β = 2.86 deg
A = 0.48
C_a = 100 psf
B = 6
γ = 125 pcf
H = 2 ft

Calculate pore pressure ratio for saturated soil:

$$r_u = \frac{X}{T} \times \frac{\gamma_w}{\gamma} Cos^2\beta$$

See Stability Charts for Infinite Slopes on page IIIE-A-3-6.

To be conservative the soil above the geocomposite is assumed to be saturated (X=T). This assumption provides maximum pore pressure ratio (r_u) as shown on page IIIE-A-3-6. Therefore the analysis is conservative.

r_u= 0.50
slope ratio, b= 20.0

Use r_u and slope ratio, b, to determine parameters A and B on page IIIE-A-3-6.

F.S. = 5.2

Geocomposite/Geomembrane Interface:

$\Delta =$	11	deg
$\beta =$	2.86	deg
A =	1	
$C_a =$	100	psf
B =	6	
$\gamma =$	125	pcf
H =	2	ft

No seepage will occur between this interface. Therefore pore pressure and r_u are equal to zero and parameter A is equal to 1.

slope ratio, b= 20.0 Therefore, parameter B is equal to 6 according to the chart on page IIIE-A-3-6.

F.S. =	6.3

Geomembrane/Compacted Clay Infiltration Layer Interface:

$\Delta =$	16	deg
$\beta =$	2.86	deg
A =	1	
$C_a =$	100	psf
B =	6	
$\gamma =$	125	pcf
H =	2	ft

No seepage will occur between this interface. Therefore pore pressure and r_u are equal to zero and parameter A is equal to 1.

slope ratio, b= 20.0 Therefore, parameter B is equal to 6 according to the chart on page IIIE-A-3-6.

F.S. =	8.1

B. Type IV C&D Area Final Cover System

1. Infinite Slope Stability Analysis of Cover System
25 Percent Sideslope (Top Layer of Final Cover Saturated)

Erosion Layer/Clay Infiltration Layer:

$\Delta =$ 16 deg
$\beta =$ 14.04 deg
$A =$ 0.48
$C_a =$ 100 psf
$B =$ 4.3
$\gamma =$ 125 pcf
$H =$ 1 ft

Calculate pore pressure ratio for saturated soil:

$$r_u = \frac{X}{T} \times \frac{\gamma_w}{\gamma} Cos\ ^2\beta$$

See Stability Charts for Infinite Slopes on page IIIE-A-4-20 for procedure.

To be conservative the soil above the geocomposite is assumed to be saturated (X=T). This assumption provides maximum pore pressure ratio (r_u) as shown on page IIIE-A-4-20. Therefore the analysis is conservative.

r_u= 0.47
slope ratio, b= 4.00

Use r_u and slope ratio, b to determine parameters A and B on page IIIE-A-3-6.

F.S. = 4.0

2. Infinite Slope Stability Analysis of Cover System
5 Percent Topslope (Top Layer of Final Cover Saturated)

Erosion Layer/Clay Infiltration Layer:

$\Delta =$ 16 deg
$\beta =$ 2.86 deg
$A =$ 0.49
$C_a =$ 100 psf
$B =$ 6
$\gamma =$ 125 pcf
$H =$ 1 ft

Calculate pore pressure ratio for saturated soil:

$$r_u = \frac{X}{T} \times \frac{\gamma_w}{\gamma} Cos\ ^2\beta$$

See Stability Charts for Infinite Slopes on page IIIE-A-3-6.

To be conservative the soil above the geocomposite is assumed to be saturated (X=T). This assumption provides maximum pore pressure ratio (r_u) as shown on page IIIE-A-3-6. Therefore the analysis is conservative

r_u= 0.50
slope ratio, b= 20.0

Use r_u and slope ratio, b, to determine parameters A and B on page IIIE-A-4-20.

F.S. = 7.6

A factor of safety of 1.5 is acceptable for long-term stability (Ref. 5, page 22). Therefore, the final cover systems are stable as designed. All the assumptions and values in the above demonstrations must be verified prior to construction using large diplacement interfacing shear testing.

APPENDIX IIIE-B

SETTLEMENT, STRAIN, AND HEAVE ANALYSES

Includes pages IIIE-B-1

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12-5-17

SETTLEMENT, STRAIN, AND HEAVE ANALYSES

This appendix includes the settlement analysis for foundation soils, and final cover system. Liner system settlement analysis evaluation points are shown on Sheet IIIE-B-1-2. Pages IIIE-B-1-3 through IIIE-B-1-9 contain the procedure and calculations for foundation settlement analysis. Pages IIIE-B-1-10 and IIIE-B-1-11 contain the calculations for the heave analysis. A strain analysis demonstrating that the liner and leachate collection system will maintain their integrity and perform as designed is included on pages IIIE-B-1-12 through IIIE-B-1-16. Based on this settlement analysis, a demonstration has been provided in Appendix IIIC to show that the LCS will continue to maintain less than 12 inches of head on the liner system after settlement.

The final cover and solid waste settlement and strain analyses have been performed to demonstrate that the final cover will maintain positive drainage and its components will be stable after settlement. The location of the evaluation points used for the final cover and solid waste settlement and strain calculations are shown on Sheet IIIE-B-2-2. The solid waste primary and secondary settlement analyses are provided on pages III-B-2-3 through IIIE-B-2-10. Strain calculations are summarized on page IIIE-B-2-11.

APPENDIX IIIE-B-1

LINER SYSTEM SETTLEMENT, STRAIN, AND HEAVE ANALYSES

Includes pages IIIE-B-1-1 through IIIE-B-1-18

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HARDIN COUNTY LANDFILL
0120-758-11-02
APPENDIX IIIE-B
FOUNDATION SETTLEMENT, HEAVE, AND STRAIN

Required:

1. Estimate the settlement of the landfill subgrade.

Method:

1. Waste filling and liner and final cover installation will result in loading of the foundation soils, causing consolidation. The magnitude of consolidation will be a function of the net stress increase and properties of the foundation soils. Net stress increase is assumed to be the addition of the loads to the excavation grades.

A. Select critical locations for settlement.

Evaluation point locations were chosen in the Subtitle D area based on the top of top of excavation grades to maximize the thickness of clay material beneath the excavation. Additionally, the evaluation points were chosen at specific locations to analyze the post-settlement slopes of the leachate collection system. Evaluation point locations are shown on Sheet IIIE-B-1-2.

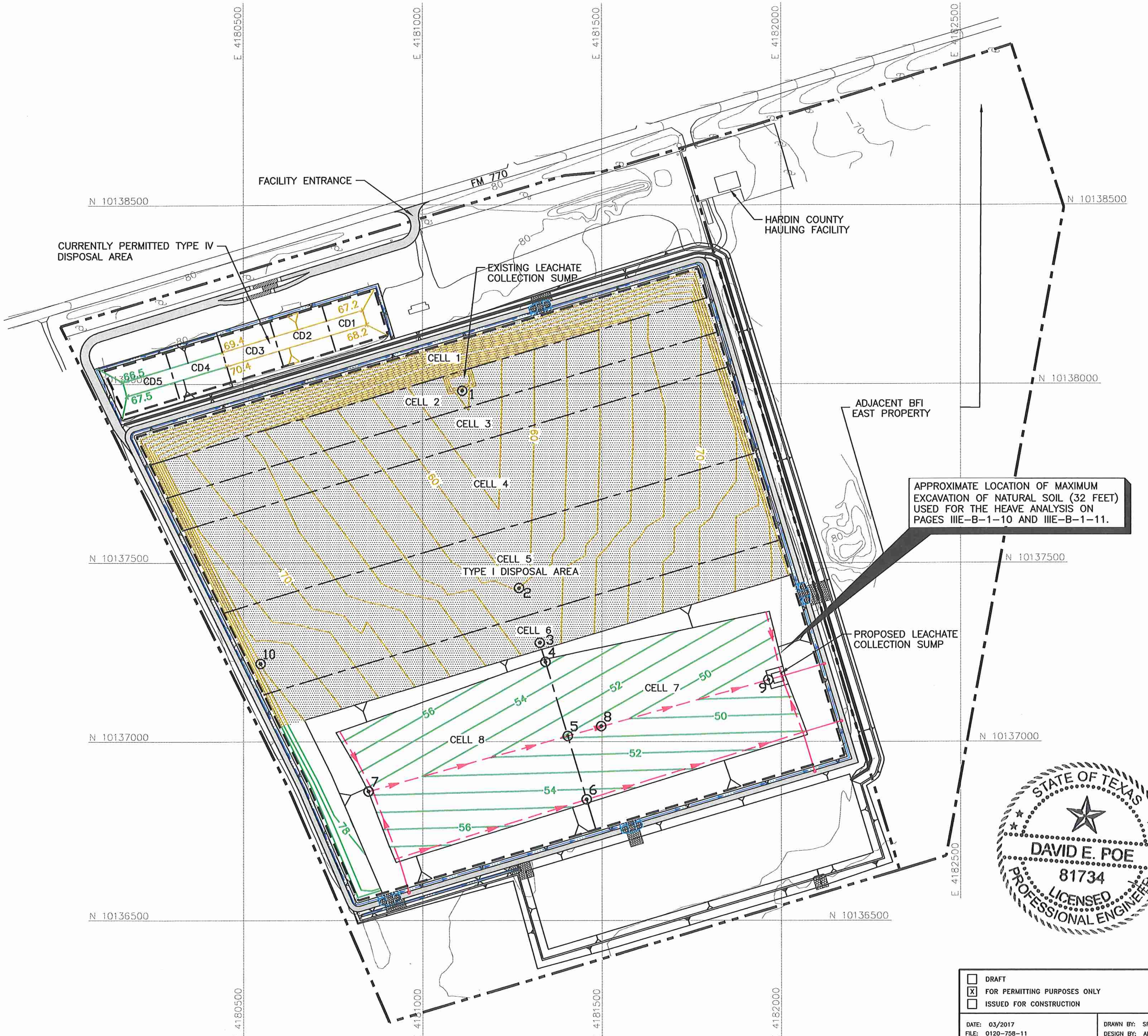
B. Use soil moisture, unit weight, and consolidation test strength values from available laboratory results.

Description of Contents:

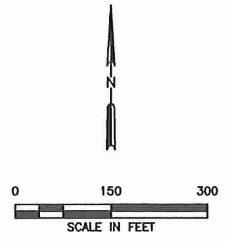
- Sheet IIIE-B-1-2 shows the top of protective cover (Cells 1-6) and excavation plan (Cells 7-8) contours.
- Sheets IIIE-B-1-3 through IIIE-B-1-7 detail the foundation settlement calculations.
- Sheets IIIE-B-1-8 and IIIE-B-1-9 provide a summary of the post-settlement slopes.
- Sheets IIIE-B-1-10 and IIIE-B-1-11 provide the heave analysis.
- Sheets IIIE-B-1-12 through IIIE-B-1-16 detail the liner strain calculations.

References:

1. Day, Robert W., Geotechnical Engineer's Portable Handbook, 2000.
2. Das, Braja M., Principles of Geotechnical Engineering, 4th edition, 1998.
3. Dunn, I.S., Anderson, L.R., and Kiefer, F.W., Fundamentals of Geotechnical Analysis, 1st Edition, 1980.
4. Coduto, Donald P., Geotechnical Engineering Principles and Practices, 1999.
5. Acar, Yalcin B.& Daniel, David E., *Geoenvironment 2000 Characterization, Containment, Remediation, and Performance in Environmental Geotechnics,* Volume 2, American Society of Civil Engineers, 1995.



APPROXIMATE LOCATION OF MAXIMUM EXCAVATION OF NATURAL SOIL (32 FEET) USED FOR THE HEAVE ANALYSIS ON PAGES IIIE-B-1-10 AND IIIE-B-1-11.



LEGEND

— · — · —	BFI EAST PROPERTY BOUNDARY
—— · ——	PERMIT BOUNDARY
— — — —	CURRENTLY PERMITTED LIMITS OF WASTE
——70——	EXISTING CONTOUR (SEE NOTE 1)
N 10137000	STATE PLANE COORDINATE SYSTEM (SEE NOTE 1)
—— —— ——	CELL BOUNDARY
——50——	PROPOSED EXCAVATION CONTOUR
——70——	CONSTRUCTED TOP OF PROTECTIVE COVER CONTOUR
— — ◄ — —	PROPOSED LEACHATE LINE
——●	PROPOSED LEACHATE RISER
▨	EXISTING SUBTITLE D COMPOSITE LINER AREA
⊙4	SETTLEMENT EVALUATION POINT

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS DEVELOPED BY WEAVER CONSULTANTS GROUP FROM AERIAL PHOTOGRAPHY FLOWN 05-17-2016. GRID SYSTEM IS TIED TO THE TEXAS STATE PLANE COORDINATE SYSTEM NAD 83, CENTRAL ZONE.
2. CURRENTLY PERMITTED TYPE IV CELLS INCLUDES CD1 THROUGH CD5. CD1, CD2, AND CD3 ARE CURRENTLY DEVELOPED AND RECEIVED TYPE IV WASTE.

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81734

12-5-17

☐ DRAFT
☒ FOR PERMITTING PURPOSES ONLY
☐ ISSUED FOR CONSTRUCTION

DATE: 03/2017
FILE: 0120-758-11
CAD: IIIE-B-1-2 SETTLEMENT EVAL.DWG

DRAWN BY: SRF
DESIGN BY: AE
REVIEWED BY: NT

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

PREPARED FOR
BFI WASTE SYSTEMS OF NORTH AMERICA, LLC

REVISIONS

NO.	DATE	DESCRIPTION
1	08/2017	FIRST NOD RESPONSE
2	11/2017	OWNERSHIP CHANGE

MAJOR PERMIT AMENDMENT
SETTLEMENT EVALUATION POINTS

HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS

WWW.WCGRP.COM

IIIE-B-1-2

O:\0120\758\2214B EXPANSION\IIIE\IIIE-B-1-2 SETTLEMENT ANALYSIS.dwg, 11/15/2017 8:36:58 AM, rsellers, 1:2

Solution: Note that the values for unit weights and natural moisture content match those used in the slope stability analysis.

The following equation was used to calculate the effective overburden stress. The Upper Clay Stratum was assumed to be moist, and the moist unit weight was used

$$p_{o(n)} = 0.5 H_{o(n)} \gamma_n + \sum_{i=1}^{i=n-1} (H_{o(i)} \gamma_i)$$

The following equation was used to calculate the increase in overburden stress due to the development of the landfill using moist unit weights.

$$\Delta p = H_{liner} \gamma_{liner} + H_{pc} \gamma_{pc} + H_{waste} \gamma_{waste} + H_{fc} \gamma_{fc}$$

The following equations were used to calculate the settlement.

If $p_o + \Delta p < p_c$:

$$S = \frac{C_r H_o}{1+e_0} \mathrm{Log}\left(\frac{p_o + \Delta p}{p_o} \right)$$

If $p_o + \Delta p > p_c$:

$$S = \frac{C_c H_o}{1+e_0} \mathrm{Log}\left(\frac{p_o + \Delta p}{p_c} \right) + \frac{C_r H_o}{1+e_0} \mathrm{Log}\left(\frac{p_c}{p_o} \right)$$

The following equation was used to calculate the final height of each layer after settlement.

$$H_f = H_o - S$$

Description of Variables:

H_o = Initial Thickness of Sublayer (ft)
γ_{dry} = Dry Unit Weight (pcf)
γ_m = Moist Unit Weight (pcf)
γ_{sat} = Saturated Unit Weight (pcf)
γ_w = Unit Weight of Water (pcf)
γ_{waste} = Unit Weight of Waste (pcf)
w = Moisture Content
G_s = Specific Gravity
e_o = In Situ Void Ratio
p_o = Initial Average Effective Overburden Pressure (psf)
Δp = Increase in Vertical Pressure (psf)
p_c = Preconsolidation Pressure (psf)
S = Settlement (ft)
C_c = Compression Index
C_r = Recompression Index
H_f = Final Thickness of Sublayer (ft)

Symbols for Indices:

$_{pc}$ = Protective Cover
$_{liner}$ = Liner
$_{waste}$ = Waste
$_n$ = Number of Layers Including Landfill System Components (i.e., liner system, waste, etc.)
$_{fc}$ = Final Cover

INCREASE IN OVERBURDEN STRESS DUE TO DEVELOPMENT OF LANDFILL:

Note: Weights of materials above the evaluated strata are assumed to have moist units weights for the purpose of calculating the overburden pressure generated by these layers to estimate Δp. The unit weight of waste was based on the average waste thickness using the Unit Weight Profile for Waste/Daily Cover within an MSW Landfill chart from Ref. 5.

Evaluation Point 1

Final Cover Elevation (ft-msl)= 116.50
Top of Waste Elevation (ft-msl)= 113
Top of Protective Cover Elevation (ft-msl)= 54.20
Top of Liner Elevation (ft-msl)= 52.20

Waste Thickness (ft)= 58.8
γ_{waste}(pcf)= 50
$p_{(waste)}$ (psf)= 2940.0
Protective Cover Thickness (ft)= 2.0
γ_{pc} (pcf)= 116
$p_{(pc)}$ (psf)= 232
Liner Thickness (ft)= 2.0
γ_{liner} (pcf)= 120
$p_{(liner)}$ (psf)= 240
Final Cover Thickness (ft)= 3.5
γ_{fc} (pcf)= 116
$p_{(fc)}$ (psf)= 406
Δp (psf)= 3818.0

Evaluation Point 2

Final Cover Elevation (ft-msl)= 234.10
Top of Waste Elevation (ft-msl)= 230.6
Top of Protective Cover Elevation (ft-msl)= 62.00
Top of Liner Elevation (ft-msl)= 60.00

Waste Thickness (ft)= 168.6
γ_{waste}(pcf)= 68
$p_{(waste)}$ (psf)= 11464.8
Protective Cover Thickness (ft)= 2.0
γ_{pc} (pcf)= 116
$p_{(pc)}$ (psf)= 232
Liner Thickness (ft)= 2.0
γ_{liner} (pcf)= 120
$p_{(liner)}$ (psf)= 240
Final Cover Thickness (ft)= 3.5
γ_{fc} (pcf)= 116
$p_{(fc)}$ (psf)= 406
Δp (psf)= 12342.8

Evaluation Point 3

Final Cover Elevation (ft-msl)= 216.00
Top of Waste Elevation (ft-msl)= 212.5
Top of Protective Cover Elevation (ft-msl)= 64.80
Top of Liner Elevation (ft-msl)= 62.80

Waste Thickness (ft)= 147.7
γ_{waste}(pcf)= 62
$p_{(waste)}$ (psf)= 9157.4
Protective Cover Thickness (ft)= 2.0
γ_{pc} (pcf)= 116
$p_{(pc)}$ (psf)= 232
Liner Thickness (ft)= 2.0
γ_{liner} (pcf)= 120
$p_{(liner)}$ (psf)= 240
Final Cover Thickness (ft)= 3.5
γ_{fc} (pcf)= 116
$p_{(fc)}$ (psf)= 406
Δp (psf)= 10035.4

Evaluation Point 4

Final Cover Elevation (ft-msl)= 202.00
Top of Waste Elevation (ft-msl)= 198.5
Top of Protective Cover Elevation (ft-msl)= 57.20
Top of Liner Elevation (ft-msl)= 55.20

Waste Thickness (ft)= 141.3
γ_{waste}(pcf)= 62
$p_{(waste)}$ (psf)= 8760.6
Protective Cover Thickness (ft)= 2.0
γ_{pc} (pcf)= 116
$p_{(pc)}$ (psf)= 232
Liner Thickness (ft)= 2.0
γ_{liner} (pcf)= 120
$p_{(liner)}$ (psf)= 240
Final Cover Thickness (ft)= 3.5
γ_{fc} (pcf)= 116
$p_{(fc)}$ (psf)= 406
Δp (psf)= 9638.6

Evaluation Point 5

Final Cover Elevation (ft-msl)= 148.00
Top of Waste Elevation (ft-msl)= 144.5
Top of Protective Cover Elevation (ft-msl)= 53.00
Top of Liner Elevation (ft-msl)= 51.00

Waste Thickness (ft)= 91.5
γ_{waste}(pcf)= 57
$p_{(waste)}$ (psf)= 5215.5
Protective Cover Thickness (ft)= 2.0
γ_{pc} (pcf)= 116
$p_{(pc)}$ (psf)= 232
Liner Thickness (ft)= 2.0
γ_{liner} (pcf)= 120
$p_{(liner)}$ (psf)= 240
Final Cover Thickness (ft)= 3.5
γ_{fc} (pcf)= 116
$p_{(fc)}$ (psf)= 406
Δp (psf)= 6093.5

Evaluation Point 6

Final Cover Elevation (ft-msl)= 101.50
Top of Waste Elevation (ft-msl)= 98
Top of Protective Cover Elevation (ft-msl)= 66.60
Top of Liner Elevation (ft-msl)= 64.60

Waste Thickness (ft)= 31.4
γ_{waste}(pcf)= 45
$p_{(waste)}$ (psf)= 1413.0
Protective Cover Thickness (ft)= 2.0
γ_{pc} (pcf)= 116
$p_{(pc)}$ (psf)= 232
Liner Thickness (ft)= 2.0
γ_{liner} (pcf)= 120
$p_{(liner)}$ (psf)= 240
Final Cover Thickness (ft)= 3.5
γ_{fc} (pcf)= 116
$p_{(fc)}$ (psf)= 406
Δp (psf)= 2291.0

Evaluation Point 7

Final Cover Elevation (ft-msl)= 116.70 Top of Waste Elevation (ft-msl)= 113.2
Top of Protective Cover Elevation (ft-msl)= 55.80 Top of Liner Elevation (ft-msl)= 53.80

Waste Thickness (ft)= 57.4 γ_{waste}(pcf)= 50 $p_{(waste)}$ (psf)= 2870.0
Protective Cover Thickness (ft)= 2.0 γ_{pc} (pcf)= 116 $p_{(pc)}$ (psf)= 232
Liner Thickness (ft)= 2.0 γ_{liner} (pcf)= 120 $p_{(liner)}$ (psf)= 240
Final Cover Thickness (ft)= 3.5 γ_{fc} (pcf)= 116 $p_{(fc)}$ (psf)= 406
Δp (psf)= 3748.0

Evaluation Point 8

Final Cover Elevation (ft-msl)= 148.00 Top of Waste Elevation (ft-msl)= 144.5
Top of Protective Cover Elevation (ft-msl)= 52.90 Top of Liner Elevation (ft-msl)= 50.90

Waste Thickness (ft)= 91.6 γ_{waste}(pcf)= 58 $p_{(waste)}$ (psf)= 5312.8
Protective Cover Thickness (ft)= 2.0 γ_{pc} (pcf)= 116 $p_{(pc)}$ (psf)= 232
Liner Thickness (ft)= 2.0 γ_{liner} (pcf)= 120 $p_{(liner)}$ (psf)= 240
Final Cover Thickness (ft)= 3.5 γ_{fc} (pcf)= 116 $p_{(fc)}$ (psf)= 406
Δp (psf)= 6190.8

Evaluation Point 9

Final Cover Elevation (ft-msl)= 114.60 Top of Waste Elevation (ft-msl)= 111.1
Top of Protective Cover Elevation (ft-msl)= 50.00 Top of Liner Elevation (ft-msl)= 48.00

Waste Thickness (ft)= 61.1 γ_{waste}(pcf)= 51 $p_{(waste)}$ (psf)= 3116.1
Protective Cover Thickness (ft)= 2.0 γ_{pc} (pcf)= 116 $p_{(pc)}$ (psf)= 232
Liner Thickness (ft)= 2.0 γ_{liner} (pcf)= 120 $p_{(liner)}$ (psf)= 240
Final Cover Thickness (ft)= 3.5 γ_{fc} (pcf)= 116 $p_{(fc)}$ (psf)= 406
Δp (psf)= 3994.1

Evaluation Point 10

Final Cover Elevation (ft-msl)= 90.00 Top of Waste Elevation (ft-msl)= 86.5
Top of Protective Cover Elevation (ft-msl)= 76.00 Top of Liner Elevation (ft-msl)= 74.00

Waste Thickness (ft)= 10.5 γ_{waste}(pcf)= 45 $p_{(waste)}$ (psf)= 472.5
Protective Cover Thickness (ft)= 2.0 γ_{pc} (pcf)= 116 $p_{(pc)}$ (psf)= 232
Liner Thickness (ft)= 2.0 γ_{liner} (pcf)= 120 $p_{(liner)}$ (psf)= 240
Final Cover Thickness (ft)= 3.5 γ_{fc} (pcf)= 116 $p_{(fc)}$ (psf)= 406
Δp (psf)= 1350.5

Prep By: DEP
Date: 12/5/2017

Chkd By: NT
Date: 12/5/2017

HARDIN COUNTY LANDFILL
0120-758-11-02
APPENDIX IIIE-B
FOUNDATION SETTLEMENT

Evaluation Point	Unit	Top Elevation (ft-msl)	Bottom Elevation (ft-msl)	H_o (ft)	G_s	$\gamma_{dry}{}^3$ (pcf)	γ_w (pcf)	γ_m (pcf)	γ_{sat} (pcf)	w^3 (%)	e_o	$C_c{}^1$	$C_r{}^1$	p_c (psf)	p_o (psf)	Δp (psf)	S^2 (ft)	H_f (ft)
1	Upper Clay Stratum	50.20	-150	200.20	2.7	92.9	62.4	119.4	121.9	28.5	0.7810	0.060	0.050	1980	11949.6	3818.0	1.690	198.510
2	Upper Clay Stratum	58.00	-150	208.00	2.7	92.9	62.4	119.4	121.9	28.5	0.7810	0.060	0.050	1980	12415.2	12342.8	3.032	204.968
3	Upper Clay Stratum	60.80	-150	210.80	2.7	92.9	62.4	119.4	121.9	28.5	0.7810	0.060	0.050	1980	12582.3	10035.4	2.759	208.041
4	Upper Clay Stratum	53.20	-150	203.20	2.7	92.9	62.4	119.4	121.9	28.5	0.7810	0.060	0.050	1980	12128.7	9638.6	2.637	200.563
5	Upper Clay Stratum	49.00	-150	199.00	2.7	92.9	62.4	119.4	121.9	28.5	0.7810	0.060	0.050	1980	11878.0	6093.5	2.075	196.925
6	Upper Clay Stratum	52.60	-150	202.60	2.7	92.9	62.4	119.4	121.9	28.5	0.7810	0.060	0.050	1980	12092.8	2291.0	1.408	201.192
7	Upper Clay Stratum	51.80	-150	201.80	2.7	92.9	62.4	119.4	121.9	28.5	0.7810	0.060	0.050	1980	12045.1	3748.0	1.688	200.112
8	Upper Clay Stratum	48.90	-150	198.90	2.7	92.9	62.4	119.4	121.9	28.5	0.7810	0.060	0.050	1980	11872.0	6190.8	2.090	196.810
9	Upper Clay Stratum	46.00	-150	196.00	2.7	92.9	62.4	119.4	121.9	28.5	0.7810	0.060	0.050	1980	11698.9	3994.1	1.691	194.309
10	Upper Clay Stratum	72.00	-150	222.00	2.7	92.9	62.4	119.4	121.9	28.5	0.7810	0.060	0.050	1980	13250.8	1350.5	1.344	220.656

[1] C_c and C_r are the slopes of the *e* -log *p* plot determined using standard laboratory consolidation testing methods. The void ratio and preconsolidation pressure are obtained from the consolidation tests. For surface clay, these parameters are obtained from the consolidation test from a boring nearest to the evaluation point.

[2] Settlement has been estimated from excavation grades.

[3] Values selected for γ_{dry}, γ_m, γ_{sat} and w are values consistent with Appendix IIIE-A values.

HARDIN COUNTY LANDFILL
0120-758-11-02
APPENDIX IIIE-B
SUMMARY OF LINER SLOPES AFTER SETTLEMENT

Slope from Evaluation Point 2 to Evaluation Point 1

Prior to Settlement				Estimated Settlement		After Settlement			
Length	Elevation Point 2 (ft-msl)	Elevation Point 1 (ft-msl)	Slope (Percent)	Elevation Point 2 (ft-msl)	Elevation Point 1 (ft-msl)	Length (ft)	Elevation Point 2 (ft-msl)	Elevation Point 1 (ft-msl)	Slope (Percent)
575.00	60.00	52.20	**1.36**	3.032	1.690	575.00	56.97	50.51	**1.12**

Slope from Evaluation Point 3 to Evaluation Point 2

Prior to Settlement				Estimated Settlement		After Settlement			
Length	Elevation Point 3 (ft-msl)	Elevation Point 2 (ft-msl)	Slope (Percent)	Elevation Point 3 (ft-msl)	Elevation Point 2 (ft-msl)	Length	Elevation Point 3 (ft-msl)	Elevation Point 2 (ft-msl)	Slope (Percent)
165.00	62.80	60.00	**1.70**	2.759	3.032	165.00	60.04	56.97	**1.86**

Slope from Evaluation Point 4 to Evaluation Point 5

Prior to Settlement				Estimated Settlement		After Settlement			
Length	Elevation Point 4 (ft-msl)	Elevation Point 5 (ft-msl)	Slope (Percent)	Elevation Point 4 (ft-msl)	Elevation Point 5 (ft-msl)	Length	Elevation Point 4 (ft-msl)	Elevation Point 5 (ft-msl)	Slope (Percent)
215.00	55.30	51.00	**2.00**	2.637	2.075	215.00	52.66	48.93	**1.74**

Slope from Evaluation Point 6 to Evaluation Point 5

Prior to Settlement				Estimated Settlement		After Settlement			
Length	Elevation Point 6 (ft-msl)	Elevation Point 5 (ft-msl)	Slope (Percent)	Elevation Point 6 (ft-msl)	Elevation Point 5 (ft-msl)	Length	Elevation Point 6 (ft-msl)	Elevation Point 5 (ft-msl)	Slope (Percent)
182.00	55.60	51.00	**2.53**	1.408	2.075	182.00	54.19	48.93	**2.89**

Slope from Evaluation Point 7 to Evaluation Point 5

Prior to Settlement				Estimated Settlement		After Settlement			
Length	Elevation Point 7 (ft-msl)	Elevation Point 5 (ft-msl)	Slope (Percent)	Elevation Point 7 (ft-msl)	Elevation Point 5 (ft-msl)	Length	Elevation Point 7 (ft-msl)	Elevation Point 5 (ft-msl)	Slope (Percent)
579.00	53.80	51.00	**0.48**	1.688	2.075	579.00	52.11	48.93	**0.55**

Slope from Evaluation Point 5 to Evaluation Point 9

Prior to Settlement				Estimated Settlement		After Settlement			
Length	Elevation Point 5 (ft-msl)	Elevation Point 9 (ft-msl)	Slope (Percent)	Elevation Point 5 (ft-msl)	Elevation Point 9 (ft-msl)	Length	Elevation Point 5 (ft-msl)	Elevation Point 9 (ft-msl)	Slope (Percent)
580.00	51.00	48.00	**0.52**	2.075	1.691	580.00	48.93	46.31	**0.45**

Slope from Evaluation Point 4 to Evaluation Point 8

Prior to Settlement				Estimated Settlement		After Settlement			
Length	Elevation Point 4 (ft-msl)	Elevation Point 8 (ft-msl)	Slope (Percent)	Elevation Point 4 (ft-msl)	Elevation Point 8 (ft-msl)	Length	Elevation Point 4 (ft-msl)	Elevation Point 8 (ft-msl)	Slope (Percent)
226.00	55.20	50.90	**1.90**	2.637	2.090	226.00	52.56	48.81	**1.66**

Slope from Evaluation Point 10 to Evaluation Point 2

Prior to Settlement				**Estimated Settlement**		**After Settlement**			
Length	Elevation Point 10 (ft-msl)	Elevation Point 2 (ft-msl)	Slope (Percent)	Elevation Point 10 (ft-msl)	Elevation Point 2 (ft-msl)	Length	Elevation Point 10 (ft-msl)	Elevation Point 2 (ft-msl)	Slope (Percent)
755.00	74.00	60.00	**1.85**	1.344	3.032	755.00	72.66	56.97	**2.08**

Conclusion:

The above calculations verify that the slopes and lengths used to design the leachate collection system in Appendix IIIC are valid.

Required:

1. Estimate the potential heave of the excavation bottom that may occur due to the excavation of overburden soils.

Method:

Heave will be calculated using standard consolidation theory.

References:

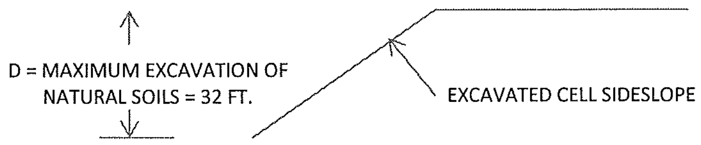
1. Terzaghi, Karl and Peck, Ralph Soil Mechanics in Engineering Principle, Third Edition, John Wiley and Sons, Inc, New York, 1996.
2. Das, Braja M., Principles of Geotechnical Engineering, Fourth Edition, PWS, Boston, 1998.
3. Day, Robert W., *Geotechnical Engineer's Portable Handbook*, McGraw-Hill, New York, 2000.

Note:

1. Approximate evaluation location for the heave analysis is shown on Sheet IIIE-B-1-2.

Solution:

Diagram for Heave Analysis:



Definition of Terms/Variables:

C_{ss} = swell index
e_o = initial void ratio
γ = unit weight of soil
ΔP = change in overburden pressure

P_o = present overburden pressure
D = depth of excavation
H_i = depth of Upper Clay Stratum (for analysi

Based on the laboratory test results included in Appendix IIIE-C, the material properties of the soil materials to be excavated at the site are:

Upper Clay Stratum

$\gamma_{CS(DRY)}$ = 95.9 pcf
Natural Moisture Content= 28.5 %
$\gamma_{CS(IN\text{-}SITU)}$ = 119.4 pcf

e_o = 0.781
C_{ss} = 0.05
H_i = 68 ft (100 ft - 32 ft excavation)

1) Estimate Potential Heave of the Excavation Bottom

The change in loading is due to the excavation of overburden soils.

The maximum depth of excavation is approximately: 32 ft
through 1 soil layers, therefore:

$$\Delta P = D_{CS} \times \gamma_{CS(IN\text{-}SITU)}$$

D_{CS} = 32 ft

ΔP = 3,821 psf

Using the standard consolidation theory:

$$S = C_{ss} H_i \log((P_o - \Delta P) / P_o)$$ (Reference 1)

$$P_o = (H_i/2) \times \gamma_{S(IN\text{-}SITU)} + \Delta P$$

P_o = 7,880 psf
S = -0.979 ft

Projected Heave = -0.979 ft

Strain Percentage in Liner System

References:

1. Quian, Xuede, R.M. Koerner, D. H. Gray, Geotechnical Aspects of Landfill Design and Construction, Prentice-Hall, Inc., New Jersey, 2002.
2. Koerner, Robert M., Designing with Geosynthetics, Third Edition. Prentice-Hall, New Jersey, 1994.

Required:

Determine the strain percentage in the Subtitle D liner system based on the total settlement between the evaluation points.

Solution:

Strain Equation:

$$Strain = \frac{L_f - L_o}{L_o} x\ 100$$ (Reference 1, Page 472)

L_f= Final distance between evaluation points after total settlement (ft)
L_o= Initial distance between evaluation points before total settlement (ft)

Note: A negative strain value indicates the component is in compression. A positive strain value indicates the component is in tension.

Strain between Points 2 and 1:

Initial Distance:

Elev. Point 2=	60	ft-msl
Elev. Point 1=	52.2	ft-msl
Plan View Distance=	575	ft
L_o=	575.0529	ft

Total Settlement:

Total Settlement Point 2=	3.032	ft
Total Settlement Point 1=	1.690	ft

Final Distance:

Elev. Point 2=	56.97	ft-msl
Elev. Point 1=	50.51	ft-msl
Plan View Distance=	575	ft
L_f=	575.0363	ft

Strain = -0.0029%

Strain between Points 3 and 2:

Initial Distance:

Elev. Point 3= 62.8 ft-msl
Elev. Point 2= 60 ft-msl
Plan View Distance= 165 ft
L_o= 165.0238 ft

Total Settlement:

Total Settlement Point 3= 2.759 ft
Total Settlement Point 2= 3.032 ft

Final Distance:

Elev. Point 3= 60.04 ft-msl
Elev. Point 2= 56.97 ft-msl
Plan View Distance= 165 ft
L_f= 165.0286 ft

Strain = 0.0029%

Strain between Points 4 and 5:

Initial Distance:

Elev. Point 4= 55.2 ft-msl
Elev. Point 5= 51 ft-msl
Plan View Distance= 215 ft
L_o= 215.0410 ft

Total Settlement:

Total Settlement Point 4= 2.637 ft
Total Settlement Point 5= 2.075 ft

Final Distance:

Elev. Point 4= 52.56 ft-msl
Elev. Point 5= 48.93 ft-msl
Plan View Distance= 215 ft
L_f= 215.0308 ft

Strain = -0.0048%

Strain between Points 6 and 5:

Initial Distance:

Elev. Point 6= 54.5 ft-msl
Elev. Point 5= 51.00 ft-msl
Plan View Distance= 182 ft
L_o= 182.0337 ft

Total Settlement:

Total Settlement Point 6= 1.408 ft
Total Settlement Point 5= 2.075 ft

Final Distance:

Elev. Point 6= 53.09 ft-msl
Elev. Point 5= 48.93 ft-msl
Plan View Distance= 182 ft
L_f= 182.0477 ft

Strain = 0.0077%

Strain between Points 7 and 5:

Initial Distance:

Elev. Point 7= 53.80 ft-msl
Elev. Point 5= 51.00 ft-msl
Plan View Distance= 579 ft
L_o= 579.0068 ft

Total Settlement:

Total Settlement Point 7= 1.688 ft
Total Settlement Point 5= 2.075 ft

Final Distance:

Elev. Point 7= 52.11 ft-msl
Elev. Point 5= 48.93 ft-msl
Plan View Distance= 579 ft
L_f= 579.0088 ft

Strain = 0.0003%

Strain between Points 5 and 9:

Initial Distance:

Elev. Point 5=	51.00 ft-msl
Elev. Point 9=	48.00 ft-msl
Plan View Distance=	580.00 ft
L_o=	580.0078 ft

Total Settlement:

Total Settlement Point 5=	2.075 ft
Total Settlement Point 9=	1.691 ft

Final Distance:

Elev. Point 5=	48.93 ft-msl
Elev. Point 9=	46.31 ft-msl
Plan View Distance=	580 ft
L_f=	580.0059 ft

Strain = -0.0003%

Strain between Points 4 and 8:

Initial Distance:

Elev. Point 4=	55.20 ft-msl
Elev. Point 8=	50.90 ft-msl
Plan View Distance=	226.00 ft
L_o=	226.0409 ft

Total Settlement:

Total Settlement Point 4=	2.637 ft
Total Settlement Point 8=	2.090 ft

Final Distance:

Elev. Point 4=	52.56 ft-msl
Elev. Point 8=	48.81 ft-msl
Plan View Distance=	226.00 ft
L_f=	226.0312 ft

Strain = -0.0043%

Strain between Points 10 and 2:

Initial Distance:

Elev. Point 10= 74.00 ft-msl
Elev. Point 2= 60.00 ft-msl
Plan View Distance= 755.00 ft
L_o= 755.1298 ft

Total Settlement:

Total Settlement Point 10= 1.344 ft
Total Settlement Point 2= 3.032 ft

Final Distance:

Elev. Point 10= 72.66 ft-msl
Elev. Point 2= 56.97 ft-msl
Plan View Distance= 755.00 ft
L_f= 755.1630 ft

Strain = 0.0044%

Conclusion:

Strain is acceptable.

- The allowable tensile strain for an HDPE geomembrane is 25 percent (Reference 1, page 94).
- The allowable tensile strain for a drainage geocomposite is more than 20 percent for the geotextile (reference 2, page 112) and 200 percent for the geonet (reference 2, page 400).
  The allowable tensile strain for compacted clay liner is 0.5 percent (Reference 1, page 469).
- The maximum calculated tensile strain (0.0077%) is below the allowable tensile strain for the components of the liner system; therefore, the system will be stable.

APPENDIX IIIE-B-2

FINAL COVER SYSTEM SETTLEMENT AND STRAIN ANALYSES

Includes pages IIIE-B-2-1 through IIIE-B-2-11

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HARDIN COUNTY LANDFILL
0120-758-11-02
APPENDIX IIIE-B
SOLID WASTE AND FINAL COVER
SETTLEMENT AND STRAIN

Required: Determine the post-settlement slope of the final cover system and verify that the strain induced on the final cover system due to settlement is within acceptable limits.

Method:

A. Estimate primary settlement of waste below the final cover system.
B. Estimate secondary settlement of waste below the final cover system.
C. Estimate total settlement of waste below the final cover system.
D. Verify that strain induced on the geocomposite due to settlement is within acceptable limits.

Description of Contents:

- Sheet IIIE-B-2-2 shows the final contour plan and evaluation points.
- Sheets IIIE-B-2-3 through IIIE-B-2-9 detail the procedure for the MSW and final cover settlement calculations.
- Sheet IIIE-B-2-10 provides a summary of the settlement analysis for the final cover.
- Sheet IIIE-B-2-11 provides a summary of the strain evaluation for the final cover.

References:

1. Sowers, George F., Settlement of Solid Waste, *Proceedings of the Eighth International Conference on Soil Mechanics and Foundations Engineering, 1973*.

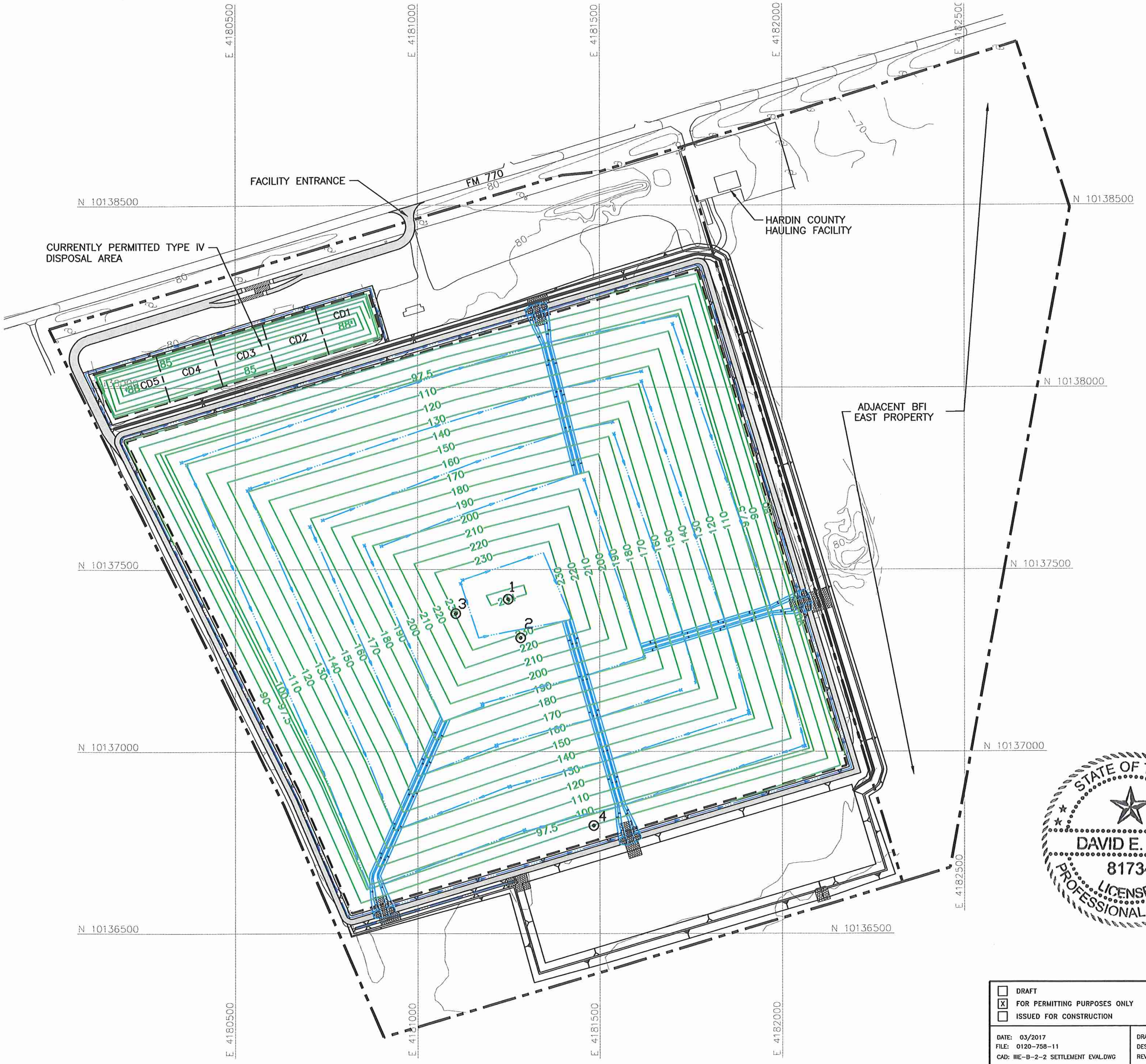
2. Quian, Xuede, R.M. Koerner, D. H. Gray, Geotechnical Aspects of Landfill Design and Construction, Prentice-Hall, Inc., New Jersey, 2002.

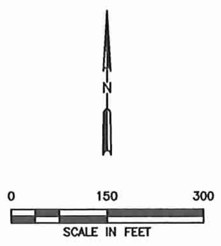
3. Koerner, Robert M., Designing with Geosynthetics, Third Edition. Prentice-Hall, New Jersey, 1994.

4. Acar, Yalcin B. & Daniel, David E., *Geoenvironment 2000 Characterization, Containment, Remediation, and Performance in Environmental Geotechnics,* Volume 2, American Society of Civil Engineers, 1995.

5. Zornberg, Jorge G., et al., *Retention of Free Liquids in Landfills Undergoing* Vertical Expansion, Journal of Geotechnical and Geoenvironmental Engineering, July 1999.

6. Fassett, Jeffrey B., et al., Geotechnical Properties of Municipal Solid Wastes and Their Use in Landfill Design, Waste Tech, 1994.





LEGEND

— - — -	BFI EAST PROPERTY BOUNDARY
— - - —	PERMIT BOUNDARY
— — — —	CURRENTLY PERMITTED LIMITS OF WASTE
— — - —	CELL BOUNDARY
N 10137500	STATE PLANE COORDINATE GRID
—70—	EXISTING CONTOUR
—234—	PROPOSED FINAL CONTOUR (SEE NOTE 3)
—→—	PROPOSED DRAINAGE SWALE
▭	PROPOSED DRAINAGE CHUTE
⊙4	SETTLEMENT EVALUATION POINT

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS DEVELOPED BY WEAVER CONSULTANTS GROUP FROM AERIAL PHOTOGRAPHY FLOWN 05-17-2016. GRID SYSTEM IS TIED TO THE TEXAS STATE PLANE COORDINATE SYSTEM NAD 83, CENTRAL ZONE.
2. MAXIMUM FINAL COVER ELEVATION IS 234 FT-MSL. MAXIMUM TOP OF WASTE ELEVATION IS 230.5 FT-MSL.

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12·5·17

☐ DRAFT
☒ FOR PERMITTING PURPOSES ONLY
☐ ISSUED FOR CONSTRUCTION

DATE: 03/2017
FILE: 0120-758-11
CAD: IIIE-B-2-2 SETTLEMENT EVAL.DWG

DRAWN BY: SRF
DESIGN BY: AE
REVIEWED BY: NT

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

PREPARED FOR
BFI WASTE SYSTEMS OF NORTH AMERICA, LLC

REVISIONS

NO.	DATE	DESCRIPTION
1	11/2017	OWNERSHIP CHANGE

MAJOR PERMIT AMENDMENT
FINAL COVER
SETTELEMENT EVALUATION POINTS

HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS

WWW.WCGRP.COM

DRAWING IIIE-B-2-2

Solution:

A) Estimate primary settlement of waste below the final cover system.

MSW will undergo primary consolidation due to its own weight, final cover, equipment, etc. Primary consolidation occurs quickly, generally within the first month after loading. Therefore, the weight of the final cover system is the only remaining factor that contributes to primary consolidation. In addition, by the time the construction of the final cover is complete, settlement of the waste due to the weight of the final cover will be complete.

Primary settlement is calculated using the following equation:

$$S_p = \frac{H_o C_c}{1+e_o} \log\left(\frac{\sigma'_o + \Delta\sigma}{\sigma'_o}\right)$$

S_p = primary settlement, ft
H_o = waste thickness below the final cover system, ft
C_c = compression index
e_o = void ratio of the waste layer below final cover before settlement (i.e., before final cover placement)
$\Delta\sigma$ = change in loading/increase in overburden pressure, psf
σ'_o = overburden pressure acting at mid-height of refuse below the final cover, psf

For this site assume: $C_c = 0.35 \times e_o$ (Ref. 1, p. 210)

The compression index is a function of the void ratio. The compression index can range from C_c=0.15e_o to C_c=0.55e_o for fills that are low and high in organic content, respectively. An average compression index value was chosen because it is consistent with the types of waste accepted in the past. It is also representative of the minimal amount of settlement the site has experienced.

The average void ratio of waste below the final cover is estimated by determining the void ratio at the midpoint of the waste column below the final cover system. The void ratio is calculated for each settlement evaluation point using the following equation.

$$e_o = 1.86 - 0.00102\, \sigma'_o \qquad \text{(Ref. 5, p. 590)}$$

where: σ'_o = overburden pressure in kPa

$\sigma'_o = 0.5\, \gamma_{msw} H_o$
$\Delta\sigma = \gamma_{cov} T_c$
γ_{msw} = unit weight of waste below the final cover system, pcf
γ_{cov} = unit weight of cover, pcf
T_c = thickness of final cover system, ft

Parameters:

γ_{cov} = 116 pcf
T_c = 3.5 feet
γ_{msw} = varies (see note below)

Note: γ_{msw} is selected based on the waste thicknesses below the final cover system using the Unit Weight Profile for Waste/Daily Cover within an MSW Landfill chart from Ref. 4.

The settlement points analyzed are shown on Sheet IIIE-B-2-2. An example calculation of the estimated primary settlement is shown below for Evaluation Points 1 and 2. The estimated primary settlement for all evaluation points is shown on Sheets IIIE-B-2-10.

At Evaluation Point 1

Top of Waste Elevation (ft-msl)= 230.5
Bottom of Waste Elevation (ft-msl)= 64.0
H_o = 166.5 ft
γ_{msw} = 68 pcf

$\sigma'_o = 0.5\ \gamma_{msw}\ H_o$
σ'_o = 5661.0 psf
σ'_o = 271.1 kPa

$e_o = 1.86 - 0.00102\ \sigma'_o$
e_o = 1.6

$C_c = 0.35\ e_o$
C_c = 0.55

$\Delta\sigma$ = 406.0 psf

$$S_p = \frac{297.1\text{x}0.45}{1\text{+}1.3}\log\left(\frac{11735.5\text{+}406.0}{11735.5}\right)$$

S_p = 1.1 ft

At Evaluation Point 2

Top of Waste Elevation (ft-msl)= 226.5
Bottom of Waste Elevation (ft-msl)= 66.3
$H_o =$ 160.2 ft
$\gamma_{msw} =$ 68 pcf

$\sigma'_o = 0.5\ \gamma_{msw}\ H_o$
$\sigma'_o =$ 5446.8 psf
$\sigma'_o =$ 260.8 kPa

$e_o = 1.86 - 0.00102\ \sigma'_o$
$e_o =$ 1.6

$C_c = 0.35\ e_o$
$C_c =$ 0.56

$\Delta\sigma =$ 406.0 psf

$$S_p = \frac{272.8x0.47}{1+1.3} \log\left(\frac{10502.8+406.0}{10502.8}\right)$$

$S_p =$ 1.1 ft

B) Estimate secondary settlement of waste below the final cover system.

Secondary consolidation continues at substantial rates for periods of time well beyond primary settlement. It is a combination of mechanical secondary compression, physico-chemical reaction, and bio-chemical decay. The settlement-log time relationship is similar to secondary compression of soils and can be expressed by:

$$S_c = \frac{H'_o \alpha}{1 + e'_o} \log (t_2/t_1)$$ (Ref. 2, p. 451)

Parameters:

S_c = secondary settlement, ft
α = secondary compression index
e'_o = void ratio of the waste layer below the final cover after primary settlement has occurred due to the final cover
H'_o = waste thickness below the final cover system after settlement, ft
t_1 = starting time of secondary settlement in years
t_2 = time at which settlement is determined in years

For this site assume: $\alpha =$ 0.06 x e'_o (Ref. 1, p. 210)

As reported by Sowers (Ref. 1), the secondary compression index is used to estimate waste decomposition. The secondary compression index ranges from $\alpha = 0.03e'_o$ to $\alpha = 0.09e'_o$ for conditions that are unfavorable and favorable to decay, respectively. An average secondary compression index value was chosen because it is consistent with the types of waste accepted in the past. It is also representative of the minimal amount of settlement the site has experienced.

The void ratio of the waste below the final cover at closure is a function of the overburden pressure caused by placement of the final cover system. The void ratio is calculated for each settlement evaluation point using the following equation.

$$e'_o = 1.86 - 0.00102\, \sigma''_o$$ (Ref. 5, p. 590)

where: σ''_o = overburden pressure in kPa

$\sigma''_o = 0.5\, \gamma'_{msw} H'_o$
γ'_{msw} = unit weight of waste below the final cover after primary settlement has occurred, pcf

For this site, the void ratio after primary settlement for the waste/cover soils below the final cover system varies between 1.3 to 1.9. Therefore, the secondary compression index will range between 0.08 to 0.11. Most literature sources report the secondary compression index in terms of the "modified secondary compression index" (Refs. 2, 6). The modified secondary compression index is defined by the following.

$$C'_\alpha = \frac{\alpha}{1 + e'_o}$$

The secondary compression index calculated for this site translates to a modified secondary compression index of 0.03 to 0.04 (for a void ratio of 1.3 to 1.9). These values are consistent with reported values for the modified secondary compression index which vary from 0.03 to 0.1 (Refs. 2, 6).

Time frame used for this analysis:

$t_1 =$ 0.083 years
$t_2 =$ 30.0 years (postclosure period)

An example calculation of the estimated secondary settlement using the above secondary settlement period is shown below for Evaluation Points 1 and 2. The estimated secondary settlement for all evaluation points is shown on Sheet IIIE-B-2-10.

At Evaluation Point 1:

$H'_o = H_o - S_p$
$H_o' =$ 165.4 ft

$\sigma''_o = 0.5\, \gamma'_{msw}\, H'_o$
$\gamma'_{msw} =$ 71.0 pcf
$\sigma''_o =$ 5872.6 psf
$\sigma''_o =$ 281.2 kPa

$e'_o = 1.86 - 0.00102\, \sigma''_o$
$e'_o =$ 1.6

$\alpha = 0.06\, e'_o$
$\alpha =$ 0.09

$$S_c = \frac{H'_o\, \alpha}{1 + e'_o} \log (t_2/t_1)$$

$$S_c = \frac{296.2 \text{ x } 0.08}{1 + 1.3} \log (30/0.083)$$

$S_c =$ 15.5 ft

At Evaluation Point 2:

$H'_o = H_o - S_p$

$H_o' =$ 159.1 ft

$\sigma''_o = 0.5\,\gamma'_{msw}\,H'_o$

$\gamma'_{msw} =$ 71.0 pcf

$\sigma''_o =$ 5648.9 psf

$\sigma''_o =$ 270.5 kPa

$e'_o = 1.86 - 0.00102\,\sigma''_o$

$e'_o =$ 1.6

$\alpha = 0.06\,e'_o$

$\alpha =$ 0.10

$$S_c = \frac{H'_o\,\alpha}{1 + e'_o}\log(t_2/t_1)$$

$$S_c = \frac{271.9 \times 0.08}{1{+}1.3}\log(30.0/0.083)$$

$S_c =$ 15.0 ft

C) Estimate total settlement of waste below the final cover system.

Total settlement is the combination of primary and secondary settlement. An example calculation of the estimated total settlement is shown below for Evaluation Points 1 and 2. The estimated total settlement for all evaluation points is shown on page IIIE-B-2-10.

At Evaluation Point 1:				
Thickness of waste column, ft =	166.5	Primary Settlement =	1.1	ft
		Secondary Settlement =	15.5	ft
		Total Settlement =	16.6	ft
At Evaluation Point 2:				
Thickness of waste column, ft=	160.2	Primary Settlement =	1.1	ft
		Secondary Settlement =	15.0	ft
		Total Settlement =	16.0	ft

D) Verify that strain induced on the geocomposite due to settlement is within acceptable limits.

Determine the post-settlement slope of the final cover system and verify the strain induced on the geocomposite due to settlement is within acceptable limits.

$$\text{Strain} = \frac{L_f - L_o}{L_o} x100$$

(Reference 2, Page 472)

L_f = Final distance between evaluation points after total settlement (ft)
L_o = Initial distance between evaluation points before total settlement (ft)

An example calculation of the estimated strain is shown below for Evaluation Points 1 and 2. The estimated strain for all evaluation points is shown on page IIIE-B-2-11.

Evaluation Point 1 to Evaluation Point 3:

Initial Distance:
Evaluation Point 1 Elev. = 234.0 ft-msl
Evaluation Point 2 Elev. = 230.0 ft-msl
Plan View Distance= 50 ft
L_o= 50.16 ft

Total Settlement:
Total Settlement Point 1= 16.60 ft
Total Settlement Point 2= 16.05 ft

Final Distance (after settlement):
Evaluation Point 1 Elev. = 217.40 ft-msl
Evaluation Point 2 Elev. = 213.95 ft-msl
Plan View Distance= 50 ft
L_f= 50.12 ft

Strain= -0.081%

Conclusion:

Strain is acceptable.

- Compacted clay component of final cover has the smallest average allowable tensile strain value which is 0.5 percent (Reference 2, Page 469).
- The allowable tensile strain for an HDPE geomembrane is 25 percent (Reference 2, page 94).
- The allowable tensile strain for a drainage geocomposite is more than 20 percent for the geotextile (reference 3, page 112) and 200 percent for the geonet (reference 3, page 400).
- No actual strain was calculated. Both analyses indicated a minor shortening of the analyz slopes. Therefore the system is stable.

Prep By: DEP
Date: 12/5/2017

Chkd By: NT
Date: 12/5/2017

HARDIN COUNTY LANDFILL
0120-758-11-02
APPENDIX IIIE-B
FINAL COVER SETTLEMENT SUMMARY

FINAL COVER EVALUATION

Evaluation Point[1]	Initial Top of Final Cover Elevation (ft-msl)	Initial Top of Waste Elevation (ft-msl)	Bottom of Waste Elevation (ft-msl)	H_o (ft)	γ_{msw} (pcf)	σ'_o (psf)	$\Delta\sigma$ (psf)	e_o	C_c	S_p (ft)	H'_o (ft)	γ'_{msw} (pcf)	σ''_o (psf)	e'_o	α	S_c (ft)	Total Settlement (ft)	Post-Settlement Top of Final Cover Elevation (ft-msl)
1	234.0	230.5	64.0	166.5	68	5661.0	406.0	1.6	0.55	1.1	165.4	71	5872.6	1.6	0.09	15.5	16.6	217.4
2	230.0	226.5	66.3	160.2	68	5446.8	406.0	1.6	0.56	1.1	159.1	71	5648.9	1.6	0.10	15.0	16.0	214.0
3	230.0	226.5	64.2	162.3	68	5518.2	406.0	1.6	0.56	1.1	161.2	71	5723.5	1.6	0.09	15.2	16.2	213.8
4	100.0	96.5	58.9	37.6	45	846.0	406.0	1.8	0.64	1.4	36.2	47	849.6	1.8	0.11	3.6	5.0	95.0

[1] Refer to Sheet IIIE-B-2-2 for Evaluation Point locations.

Prep By: DEP
Date: 12/5/2017

Chkd By: NT
Date: 12/5/2017

HARDIN COUNTY LANDFILL
0120-758-11-02
APPENDIX IIIE-B
FINAL COVER SYSTEM PERCENT STRAIN SUMMARY

FINAL COVER EVALUATION

Evaluation Point[1]		Initial Top of Final Cover Elevation (ft-msl)		Post-Settlement Top of Final Cover Elevation (ft-msl)		Plan View Distance (ft)	L_o (ft)	L_f (ft)	Initial Slope (ft/ft)	Post-Settlement Slope (ft/ft)	Strain (%)
A	B	A	B	A	B						
1	2	234.0	230.0	217.4	214.0	50.0	50.2	50.1	0.080	0.069	-0.081
3	4	230.0	100.0	213.8	95.0	528.0	543.8	541.2	0.246	0.225	-0.473

[1] Refer to Sheet IIIE-B-2-2 for Evaluation Point locations.

APPENDIX IIIE-C

LABORATORY SUMMARY TABLES AND TEST RESULTS

Includes pages IIIE-C-1 through IIIE-C-52

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SUMMARY

The attached includes summary tables and results of geotechnical testing performed during piezometer installations for the 1995 permit application. Also attached are the results of geotechnical testing performed by WCG in 2017. Locations of borings are provided in Figure IIIG-C.1 in Appendix IIIG-C. Compiled logs of the borings are contained in Appendix IIIG – Geology Report, Appendix IIIG-B.

GEOTECHNICAL TESTING
SUMMARY TABLES

HARDIN COUNTY LANDFILL
0120-758-11-02
APPENDIX IIIE-C
GEOTECHNICAL TESTING SUMMARY

1995 Permit Application Piezometer Installations

Piezo No.	Surface El. (ft-msl)	Sample Depth (ft)	Sample El. (ft-msl)	Description	Unit Description	Geotechnical Properties			
						Moisture Content	Liquid Limit	Plasticity Index	Passing #200 Sieve
1A	82.68	6	76.68	Silty clay	1-Upper Clay Stratum	20	37	23	85
1A	82.68	11	71.68	Silty clay	1-Upper Clay Stratum	20	49	34	--
1A	82.68	19	63.68	Silty Clay with sand partings	1-Upper Clay Stratum	26	63	42	--
1A	82.68	24	58.68	Silty Clay with sand partings	1-Upper Clay Stratum	12	47	33	85
2A	81.22	3	78.22	Very silty clay to clayey silt, slightly sandy	1-Upper Clay Stratum	18	--	--	80
2A	81.22	7	74.22	Clay, slightly silty	1-Upper Clay Stratum	18	33	16	83
2A	81.22	11	70.22	Clay, slightly silty	1-Upper Clay Stratum	18	49	35	--
2A	81.22	19	62.22	Clay, slightly silty	1-Upper Clay Stratum	18	70	53	93
2A	81.22	25	56.22	Clay, slightly silty	1-Upper Clay Stratum	17	33	17	76
3A	80.24	3	77.24	Silt, slightly sandy, to very silty clay	1-Upper Clay Stratum	19	--	--	77
3A	80.24	9	71.24	Clay, slightly silty	1-Upper Clay Stratum	17	35	20	83
3A	80.24	15	65.24	Clay, slightly silty	1-Upper Clay Stratum	23	70	53	--
3A	80.24	19	61.24	Clay, slightly silty	1-Upper Clay Stratum	14	64	48	90
3A	80.24	24	56.24	Clay, silty, slightly sandy	1-Upper Clay Stratum	16	29	14	--
4A	77.94	3	74.94	Silty clay to clayey silt	1-Upper Clay Stratum	17	21	5	--
4A	77.94	7	70.94	Silty clay to clayey silt	1-Upper Clay Stratum	19	23	7	--
4A	77.94	11	66.94	Clay with sand partings	1-Upper Clay Stratum	29	65	41	--
4A	77.94	13	64.94	Clay with sand partings	1-Upper Clay Stratum	24	67	52	93
4A	77.94	21	56.94	Clay with sand partings	1-Upper Clay Stratum	23	44	27	--
4A	77.94	27	50.94	Clay with sand partings	1-Upper Clay Stratum	18	25	10	59
4A	77.94	33	44.94	Clay with sand partings	1-Upper Clay Stratum	25	65	49	--
4A	77.94	39	38.94	Clay with silt partings and sand seams	1-Upper Clay Stratum	25	34	15	88

HARDIN COUNTY LANDFILL
0120-758-11-02
APPENDIX IIIE-C
GEOTECHNICAL TESTING SUMMARY

1995 Permit Application Piezometer Installations

Piezo No.	Surface El. (ft-msl)	Sample Depth (ft)	Sample El. (ft-msl)	Description	Unit Description	Geotechnical Properties			
						Moisture Content	Liquid Limit	Plasticity Index	Passing #200 Sieve
5A	81.9	5	76.9	Silty clay	1-Upper Clay Stratum	18	29	11	--
5A	81.9	9	72.9	Silty clay	1-Upper Clay Stratum	18	47	31	85
5A	81.9	15	66.9	Silty clay	1-Upper Clay Stratum	30	74	53	--
5A	81.9	24	57.9	Clay with sand partings	1-Upper Clay Stratum	24	59	42	90
6A	81.4	7	74.4	Silty clay, slightly sandy	1-Upper Clay Stratum	18	33	16	82
6A	81.4	13	68.4	Clay with sand partings	1-Upper Clay Stratum	22	55	39	--
6A	81.4	23	58.4	Clay with sand partings	1-Upper Clay Stratum	20	36	23	--
8A	76.3	1	75.3	Sandy silt, slightly clayey	1-Upper Clay Stratum	22	--	--	75
8A	76.3	7	69.3	Very silty clay slightly sandy	1-Upper Clay Stratum	17	27	17	--
8A	76.3	13	63.3	Very silty clay slightly sandy	1-Upper Clay Stratum	18	45	14	82
8A	76.3	21	55.3	Silty clay, slightly sandy	1-Upper Clay Stratum	16	30	15	57
8A	76.3	25	51.3	Clayey fine sand	1-Upper Clay Stratum	19	24	18	50
9	79.2	1	78.2	Silt, slightly sandy	1-Upper Clay Stratum	22	--	--	75
9	79.2	5	74.2	Very silty clay, slightly sandy	1-Upper Clay Stratum	17	27	9	80
9	79.2	9	70.2	Very silty clay, slightly sandy	1-Upper Clay Stratum	17	33	17	81
9	79.2	13	66.2	Very silty clay, slightly sandy	1-Upper Clay Stratum	20	39	26	85
9	79.2	19	60.2	Very silty clay, slightly sandy	1-Upper Clay Stratum	24	64	46	--
12A	78.5	3	75.5	Very silty clay, slightly sandy	1-Upper Clay Stratum	20	27	7	78
12A	78.5	7	71.5	Very silty clay, slightly sandy	1-Upper Clay Stratum	17	31	14	--
12A	78.5	11	67.5	Clay, slightly silty	1-Upper Clay Stratum	25	59	44	91
12A	78.5	21	57.5	Clay, slightly silty	1-Upper Clay Stratum	19	46	31	72
12A	78.5	29	49.5	Silty clay	1-Upper Clay Stratum	33	29	13	--

1995 Permit Application Piezometer Installations

Piezo No.	Surface El. (ft-msl)	Sample Depth (ft)	Sample El. (ft-msl)	Description	Unit Description	Geotechnical Properties			
						Moisture Content	Liquid Limit	Plasticity Index	Passing #200 Sieve
20	79.23	7	72.23	Silty clay, slightly sandy	1-Upper Clay Stratum	16	27	10	--
20	79.23	15	64.23	Clay with sand partings	1-Upper Clay Stratum	26	65	46	--
20	79.23	21	58.23	Clay with sand partings	1-Upper Clay Stratum	19	41	27	73
20	79.23	29	50.23	Clay, slightly sandy	1-Upper Clay Stratum	29	68	46	--
20	79.23	41	38.23	Clay, slightly sandy	1-Upper Clay Stratum	27	57	38	--
21	77.24	5	72.24	Silty clay	1-Upper Clay Stratum	15	34	20	--
21	77.24	19	58.24	Clay with sand partings	1-Upper Clay Stratum	20	47	29	92
22	79.38	5	74.38	Silty clay, slightly sandy	1-Upper Clay Stratum	15	31	13	--
22	79.38	9	70.38	Clay with sand partings	1-Upper Clay Stratum	24	64	46	86
22	79.38	21	58.38	Clay with sand partings	1-Upper Clay Stratum	23	56	41	93
23	79.29	11	68.29	Clay with sand partings	1-Upper Clay Stratum	28	70	52	--
23	79.29	17	62.29	Clay with sand partings	1-Upper Clay Stratum	29	64	44	86
23	79.29	21	58.29	Clay, slightly sandy, slightly silty	1-Upper Clay Stratum	17	36	21	70
24	73.51	9	64.51	Silt with clay seams	1-Upper Clay Stratum	13	34	18	72
24	73.51	15	58.51	Clay, slightly sandy	1-Upper Clay Stratum	23	63	47	79
25	75.81	7	68.81	Clay	1-Upper Clay Stratum	16	31	13	--
25	75.81	17	58.81	Clay with sand partings	1-Upper Clay Stratum	23	56	38	85
26	75.48	9	66.48	Silty clay	1-Upper Clay Stratum	16	49	32	--
26	75.48	19	56.48	Clay, slightly sandy	1-Upper Clay Stratum	17	30	15	65
26	75.48	29	46.48	Clay, slightly sandy	1-Upper Clay Stratum	20	58	40	--
27	82.2	9	73.2	Clay with sand partings	1-Upper Clay Stratum	22	52	35	86
27	82.2	19	63.2	Clay with sand partings	1-Upper Clay Stratum	23	61	44	89

HARDIN COUNTY LANDFILL
0120-758-11-02
APPENDIX IIIE-C
GEOTECHNICAL TESTING SUMMARY

1995 Permit Application Piezometer Installations

Piezo No.	Surface El. (ft-msl)	Sample Depth (ft)	Sample El. (ft-msl)	Description	Unit Description	Geotechnical Properties			
						Moisture Content	Liquid Limit	Plasticity Index	Passing #200 Sieve
1A	82.68	29	53.68	Silty Fine Sand	2-Upper Sand Stratum (Uppermost Aquifer)	25	--	--	26
3A	80.24	30	50.24	silt, slightly sandy with clay seams	2-Upper Sand Stratum (Uppermost Aquifer)	26	--	--	80
21	77.24	31	46.24	Clayey sand	2-Upper Sand Stratum (Uppermost Aquifer)	19	24	7	57
22	79.38	31	48.38	Very silty clay with sand	2-Upper Sand Stratum (Uppermost Aquifer)	25	27	9	94
23	79.29	25	54.29	Clayey sand, slightly silty	2-Upper Sand Stratum (Uppermost Aquifer)	16	18	0	43
24	73.51	23	50.51	Clayey Sand	2-Upper Sand Stratum (Uppermost Aquifer)	19	24	5	51
25	75.81	27	48.81	Clayey sand	2-Upper Sand Stratum (Uppermost Aquifer)	21	22	2	49
27	82.2	31	51.2	Silty fine sand	2-Upper Sand Stratum (Uppermost Aquifer)	26	18	0	45
1A	82.68	43	39.68	Clay with Silt Partings	3-Lower Clay Stratum (Aquiclude/tard)	28	60	40	94
3A	80.24	34	46.24	Clay with silt partings	3-Lower Clay Stratum (Aquiclude/tard)	33	74	49	--
3A	80.24	43	37.24	Silty clay with sand seams and silt partings	3-Lower Clay Stratum (Aquiclude/tard)	21	29	11	77
21	77.24	43	34.24	clay with silt partings	3-Lower Clay Stratum (Aquiclude/tard)	23	49	33	--
24	73.51	35	38.51	Silt with sand partings	3-Lower Clay Stratum (Aquiclude/tard)	24	23	3	--

Reference: Attachment 4, Geology Report. SWL Environmental Services. April 13, 1994.

GEOTECHNICAL TESTING SUMMARY

2017 Geotechnical Investigations

Boring No.	Surface El. (ft-msl)	Sample Depth (ft)	Sample El. (ft-msl)	Description	Unit Description	Geotechnical Properties					
						Moisture Content (%)	Dry Density (pcf)	Liquid Limit	Plasticity Index	Passing #200 Sieve	Hydraulic Conductivity (cm/sec)
WC-1	70.6	30	40.6	Clay (CH), silty	3-Lower Clay Stratum	30.4	91.6	66	37	98	1.7×10^{-7}
WC-2	70.8	20	50.8	Clay (CH), silty	1-Upper Clay Stratum	32.2	91.8	73	49	96	9.7×10^{-8}
WC-3	68.9	28	40.9	Clay (CH), silty	3-Lower Clay Stratum	31.4	87.1	65	41	99	1.1×10^{-8}
WC-4	70.9	16	54.9	Clay (CH), silty	1-Upper Clay Stratum	17.3	87.1	44	30	57	--
WC-5	80.0	21	59	Clay (CL), silty	1-Upper Clay Stratum	23.7	101.5	67	45	87	--
WC-5	80.0	31	49	Clay (CL), silty	2-Upper Sand Stratum	19.3		27	11	34	--
WC-5	80.0	39	41	Clay (CH), silty	3-Lower Clay Stratum	30.6	92.1	51	29	99	1.1×10^{-8}
WC-5	80.0	46	34	Clay (CL), silty	3-Lower Clay Stratum	18.7	110	35	18	63	--
WC-5	80.0	56	24	Sand (SC-SM), silty	4-Lower Sand Stratum	--		25	7	13	--
WC-5	80.0	80	0	Sand (SC-SM), silty	4-Lower Sand Stratum	--		--	--	2	--
WC-5	80.0	97	-17	Clay (CL), silty	5-Basal Clay Stratum	17.4	113.9	45	30	66	--

Geotechnical Testing Min/Max/Average Values

	Moisture Content	Liquid Limit	Plasticity Index	Passing #200 Sieve
1-Upper Clay Stratum				
Count	69.0	65.0	65.0	41.0
Min	12.0	21.0	5.0	50.0
Max	33.0	74.0	53.0	93.0
Average	20.7	46.4	29.5	80.3
2-Upper Sand Stratum				
Count	9.0	7.0	7.0	9.0
Min	16.0	18.0	0.0	26.0
Max	26.0	27.0	11.0	94.0
Average	21.8	22.6	4.9	53.2
3-Lower Clay Stratum				
Count	9.0	9.0	9.0	6.0
Min	18.7	23.0	3.0	63.0
Max	33.0	74.0	49.0	99.0
Average	26.7	50.2	29.0	88.3
4-Lower Sand Stratum				
Count	0.0	1.0	1.0	2.0
Min	--	--	--	2.0
Max	--	--	--	13.0
Average	--	25.0	7.0	7.5
5-Basal Clay Stratum				
Count	1.0	1.0	1.0	1.0
Min	--	--	--	--
Max	--	--	--	--
Average	17.4	45.0	30.0	66.0

GEOTECHNICAL LABORATORY TESTING RESULTS

Borehole	Depth	Liquid Limit	Plastic Limit	Plasticity Index	Maximum Size (mm)	%<#200 Sieve	Class-ification	Water Content (%)	Dry Density (pcf)	Percent Passing #40	Unconfined Compressive Strength (tsf)
WC-1	30.0	66	29	37	0.075	98	CH	30.4	91.6		
WC-2	20.0	73	24	49	0.075	96	CH	32.2	91.8		
WC-3	28.0	65	24	41	0.075	99	CH	31.4	87.1		
WC-4	16.0	44	14	30	0.075	57	CL	17.3	110.7		
WC-5	21.0	67	22	45	0.075	87	CH	23.7	101.5		
WC-5	31.0	27	16	11	0.075	34	SC	19.3			
WC-5	39.0	51	22	29	0.075	99	CH	30.6	92.1		
WC-5	46.0	35	17	18	0.075	63	CL	18.7	110.0		
WC-5	56.0	25	18	7	0.075	13	SC-SM				
WC-5	80.0				0.075	2					
WC-5	97.0	45	15	30	0.075	66	CL	17.4	113.9		

US LAB SUMMARY HARDIN COUNTY EXPANSION.GPJ - 1/31/17

-

Telephone:
Fax:

Summary of Laboratory Results

Project: Hardin County Expansion

Number: 0771-365-11-07-24

LAB DATA SHEET

JOB NAME: Hardin County LF
JOB NUMBER: 0771-365-11-07-24

Date: 1/29/18
Page ______ of ______
Tech: MLT

Perm

Boring No.: WC-5 Depth: 39' Location: ______

	WET WEIGHT	DRY WEIGHT	TARE WEIGHT	TARE No.	BLOWS
LL					
PL					
-200					
MC					
UDW					
Qu					

LL = ______
PL = ______
PI = ______
-200 = ______
Ring No.: ______

REACTION TO

HCl		Pheno.
☐	NONE	☐
☐	WEAK	☐
☐	STRONG	☐

Description: ______ Hand Penetrometer: ______ % Swell ______

Perm
Consol

Boring No.: WC-3 Depth: 28' Location: ______

	WET WEIGHT	DRY WEIGHT	TARE WEIGHT	TARE No.	BLOWS
LL	35.29	27.32	15.10	84	23
PL	20.86	19.11	15.13	88	
-200	149.6	94.6	93.9	B-19	
MC	167.1	149.6	93.9	B-19	
UDW	2.8	2.75	499.9		
Qu					

LL = ______
PL = ______
PI = ______
-200 = ______
Ring No.: ______

REACTION TO

HCl		Pheno.
☐	NONE	☐
☐	WEAK	☐
☐	STRONG	☐

Description: ______ Hand Penetrometer: ______ % Swell ______

Triax

Boring No.: WC-4 Depth: 16' Location: ______

	WET WEIGHT	DRY WEIGHT	TARE WEIGHT	TARE No.	BLOWS
LL	37.07	30.59	15.68	86	26
PL	22.77	21.85	15.48	77	
-200	156.9	121.8	95.6	501	
MC	167.9	156.9	95.6	501	
UDW	5.55	2.75	1123.4		
Qu					

LL = ______
PL = ______
PI = ______
-200 = ______
Ring No.: ______

REACTION TO

HCl		Pheno.
☐	NONE	☐
☐	WEAK	☐
☐	STRONG	☐

Description: ______ Hand Penetrometer: ______ % Swell ______

Perm

Boring No.: WC-2 Depth: 20' Location: ______

	WET WEIGHT	DRY WEIGHT	TARE WEIGHT	TARE No.	BLOWS
LL	38.53	28.64	15.23	87	23
PL	20.44	19.38	15.00	89	
-200	147.8	95.9	93.5	CS	
MC	165.3	147.8	93.5	CS	
UDW	5.55	2.75	1050.8		
Qu					

LL = ______
PL = ______
PI = ______
-200 = ______
Ring No.: ______

REACTION TO

HCl		Pheno.
☐	NONE	☐
☐	WEAK	☐
☐	STRONG	☐

Description: ______ Hand Penetrometer: ______ % Swell ______

Perm

Boring No.: WC-1 Depth: 30' Location: ______

	WET WEIGHT	DRY WEIGHT	TARE WEIGHT	TARE No.	BLOWS
LL	32.02	25.26	15.16	79	23
PL	21.78	20.27	15.04	92	
-200	143.5	93.1	92.1	mm	
MC	159.1	143.5	92.1	mm	
UDW	5.5	2.75	1024.1		
Qu					

LL = ______
PL = ______
PI = ______
-200 = ______
Ring No.: ______

REACTION TO

HCl		Pheno.
☐	NONE	☐
☐	WEAK	☐
☐	STRONG	☐

Description: ______ Hand Penetrometer: ______ % Swell ______

LAB DATA SHEET

JOB NAME: Hardin County Expansion

JOB NUMBER: 0771-365-11-07-24

Date: 1/12/17

Page ______ of ______

Tech: MLT

Boring No.: WC-5 Depth: 31' Location: ______

	WET WEIGHT	DRY WEIGHT	TARE WEIGHT	TARE No.	BLOWS
LL	44.99	38.63	15.08	51	25
PL	22.17	21.20	15.09	52	
-200	166.8	143.8	99.9	pep	
MC	179.7	166.8	99.9	pep	
UDW	Sample no good				
Qu					

LL = ______
PL = ______
PI = ______
-200 = ______
Ring No.: ______

REACTION TO

HCl		Pheno.
☐	NONE	☐
☐	WEAK	☐
☐	STRONG	☐

Description: ______ Hand Penetrometer: ______ % Swell ______

Boring No.: WC-5 Depth: 21' Location: ______

	WET WEIGHT	DRY WEIGHT	TARE WEIGHT	TARE No.	BLOWS
LL	37.52	28.55	15.13	53	25
PL	23.54	22.03	15.12	54	
-200	167.2	107.1	98.5	AbA	
MC	183.5	167.2	98.5	AbA	
UDW	5.5	2.75	1077.9		
Qu					

LL = ______
PL = ______
PI = ______
-200 = ______
Ring No.: ______

REACTION TO

HCl		Pheno.
☐	NONE	☐
☐	WEAK	☐
☐	STRONG	☐

Description: ______ Hand Penetrometer: ______ % Swell ______

no sample in box

Boring No.: WC-5 Depth: 37' Location: ______

	WET WEIGHT	DRY WEIGHT	TARE WEIGHT	TARE No.	BLOWS
LL					
PL					
-200					
MC					
UDW					
Qu					

LL = ______
PL = ______
PI = ______
-200 = ______
Ring No.: ______

REACTION TO

HCl		Pheno.
☐	NONE	☐
☐	WEAK	☐
☐	STRONG	☐

Description: ______ Hand Penetrometer: ______ % Swell ______

Boring No.: WC-5 Depth: 39' Location: ______

	WET WEIGHT	DRY WEIGHT	TARE WEIGHT	TARE No.	BLOWS
LL	36.27	29.02	14.82	55	24
PL	21.23	20.15	15.16	56	
-200	143.0	92.5	92.0	MM	
MC	158.6	143.0	92.0	MM	
UDW	4.5	2.75	843.8		
Qu					

LL = ______
PL = ______
PI = ______
-200 = ______
Ring No.: ______

REACTION TO

HCl		Pheno.
☐	NONE	☐
☐	WEAK	☐
☐	STRONG	☐

Description: ______ Hand Penetrometer: ______ % Swell ______

Boring No.: WC-5 Depth: 46' Location: ______

	WET WEIGHT	DRY WEIGHT	TARE WEIGHT	TARE No.	BLOWS
LL	46.15	38.23	15.71	57	24
PL	22.02	21.04	15.26	58	
-200	178.8	124.7	93.3	C5	
MC	194.8	178.8	93.3	C5	
UDW	5.55	2.75	1130.4		
Qu					

LL = ______
PL = ______
PI = ______
-200 = ______
Ring No.: ______

REACTION TO

HCl		Pheno.
☐	NONE	☐
☐	WEAK	☐
☐	STRONG	☐

Description: ______ Hand Penetrometer: ______ % Swell ______

LAB DATA SHEET

JOB NAME: Hardin County Expansion

JOB NUMBER: 0771-365-11-07-24

Date: 1/12/17

Page ______ of ______

Tech: MLT

Boring No.: WC-5 Depth: 56-58' Location: ______

	WET WEIGHT	DRY WEIGHT	TARE WEIGHT	TARE No.	BLOWS
LL	41.37	36.11	14.87	59	25
PL	23.77	22.45	15.29	60	
-200	147.1	140.2	94.4	ML	
MC					
UDW					
Qu					

LL = ______

PL = ______

PI = ______

-200 = ______

Ring No.: ______

REACTION TO

HCl		Pheno.
☐	NONE	☐
☐	WEAK	☐
☐	STRONG	☐

Description: ______ Hand Penetrometer: ______ % Swell ______

Boring No.: WC-5 Depth: 80-82' Location: ______

	WET WEIGHT	DRY WEIGHT	TARE WEIGHT	TARE No.	BLOWS
LL					
PL					
-200	169.5	168.0	94.7	1-2	
MC					
UDW					
Qu					

LL = ______

PL = ______

PI = ______

-200 = ______

Ring No.: ______

REACTION TO

HCl		Pheno.
☐	NONE	☐
☐	WEAK	☐
☐	STRONG	☐

Description: ______ Hand Penetrometer: ______ % Swell ______

Perm & Consol

Boring No.: WC-5 Depth: 97' Location: ______

	WET WEIGHT	DRY WEIGHT	TARE WEIGHT	TARE No.	BLOWS
LL	43.52	34.69	15.15	61	26
PL	18.95	18.15	12.73	62	
-200	147.9	111.1	92.2	KH	
MC	157.6	147.9	92.2	KH	
UDW	5.5	2.75	1146.3		
Qu					

LL = ______

PL = ______

PI = ______

-200 = ______

Ring No.: ______

REACTION TO

HCl		Pheno.
☐	NONE	☐
☐	WEAK	☐
☐	STRONG	☐

Description: ______ Hand Penetrometer: ______ % Swell ______

Boring No.: ______ Depth: ______ Location: ______

	WET WEIGHT	DRY WEIGHT	TARE WEIGHT	TARE No.	BLOWS
LL					
PL					
-200					
MC					
UDW					
Qu					

LL = ______

PL = ______

PI = ______

-200 = ______

Ring No.: ______

REACTION TO

HCl		Pheno.
☐	NONE	☐
☐	WEAK	☐
☐	STRONG	☐

Description: ______ Hand Penetrometer: ______ % Swell ______

Boring No.: ______ Depth: ______ Location: ______

	WET WEIGHT	DRY WEIGHT	TARE WEIGHT	TARE No.	BLOWS
LL					
PL					
-200					
MC					
UDW					
Qu					

LL = ______

PL = ______

PI = ______

-200 = ______

Ring No.: ______

REACTION TO

HCl		Pheno.
☐	NONE	☐
☐	WEAK	☐
☐	STRONG	☐

Description: ______ Hand Penetrometer: ______ % Swell ______

CLIENT:

REPORT DATE: 2/1/2017

PROJECT NO.: 0771-365-11-07-24

PROJECT: Hardin County Expansion

HYDRAULIC CONDUCTIVITY WORKSHEET
FALLING HEAD - FIXED WALL PERMEAMETER

LOCATION:
MATERIAL:
BORING/SAMPLE: WC-1
PROCTOR # :
SAMPLE ORIENTATION: H _____ V ✓
Remold _____

LAB START DATE: 1/29/2017
LAB RPT. DATE: 2/1/2017
TECHNICIAN: MLT
DEPTH/LIFT: 30.0'
PERM FLUID USED: De-aired Tap Water

a. Length of Specimen, L: 1.0 in
c. Sample Volume
(π b^2 / 4 * a): 4.909 cu in

b. Avg. Diameter of Specimen: 2.5 in
d. Wet Unit Weight:
[((f-h)*3.8095)/c)]: 117.9 pcf

INITIAL CONDITIONS

e. Ring + Wet Weight Soil: 692.7 gms
f. Wet Weight Soil + Tare: 245.5 gms
g. Dry Weight Soil + Tare: 209.1 gms
h. Tare Weight: 93.6 gms
i. Moisture Content
[(f-g)/(g-h)]*100: 31.5 %
j. Unit Dry Weight
[d/(1+(i/100))]: 89.6 pcf

FINAL CONDITIONS

k. Wet Weight Soil + Tare: 250.7 gms
l. Dry Weight Soil + Tare: 209.1 gms
m. Tare Weight: 93.6 gms
n. Moisture Content
[(k-l)/(l-m)]*100: 36.0 %
o. Unit Dry Weight
[d/(1+(n/100))]: 86.7 pcf
p. Ring Weight: 540.8 gms

Date	Time	t sec	Initial Height, ho	Corrected ho - C	Final Height, hf	Corrected hf - C	Temp C	Rt	k @ 20C cm/sec
29-Jan	14:55		46.8	39.8					
30-Jan	06:40	56700			38.5	31.5	22	0.953	2.7E-07
30-Jan	06:40		38.5	31.5					
30-Jan	10:40	14400			37.4	30.4	22	0.953	1.6E-07
30-Jan	10:40		37.4	30.4					
30-Jan	17:30	24600			35.7	28.7	22	0.953	1.5E-07
30-Jan	17:30		35.7	28.7					
31-Jan	05:58	44880			32.2	25.2	22	0.953	1.9E-07
31-Jan	05:58		32.2	25.2					
31-Jan	10:30	16320			31.5	24.5	22	0.953	1.1E-07
30-Jan	06:40		38.5	31.5					
31-Jan	10:30	100200			31.5	24.5	22	0.953	1.7E-07

Height of Top of Specimen
From Top of Table: 7.00 cm

Standpipe Diameter
1.05 cm

Standpipe Area
0.866 sq cm

Test Method: Corps of Engineers EM 1110-2-1906, Appendix VII

Hx-C = Hx-Ht

HYDRAULIC CONDUCTIVITY WORKSHEET
Falling Head - Fixed Wall Permeameter

PROJECT: Hardin County Expansion
LOCATION:
MATERIAL:
BORING/SAMPLE NO: WC-1
SAMPLE ORIENTATION H [✓ V] R (Circle One)
a. HEIGHT: 1.0 in
c. VOLUME: (0.7854 * b ^ 2) cu in

JOB NO: 0771-365-11-07-24
DATE: 1/29/17
TECHNICIAN: MLT
DEPTH/LIFT: 30'
PERM FLUID USED: Tap Water
b. AVERAGE DIAMETER: 2.5 in
d. WET UNIT WEIGHT: [(g-i * 3.8095)/c] pcf

INITIAL CONDITIONS

e. Ring Wt: 540.8 gms
f. Ring Wt + Wet Soil: 692.7 gms
g. Wet Wt Soil + tare [f-e+i]: gms
h. Dry Wt Soil + tare: gms
i. Tare Wt: gms
j. Moisture Content [(g-h)/(h-i)]*100 %
k. Unit Dry Wt [d/(1 + (j/100))] pcf

FINAL CONDITIONS

l. Wet Wt. Soil + tare: 250.7 gms
m. Dry Wt. Soil + tare: 209.1 gms
n. Tare Wt: C-5 93.6 gms
o. Moisture Content [(l-m)/(m-n)]*100 %
p. Unit Dry Wt. [d/(1 + (o/100))] pcf

DATE	CLOCK TIME	TIME SECONDS	INITIAL HEIGHT, ho	FINAL HEIGHT, hf	TEMP °C	PERMEABILITY k, cm/sec
1/29/17	14:56		46.8			
1/30/17	6:40			38.5		
1/30/17	6:40		38.5			
1/30/17	10:40			37.4		
1/30/17	10:40		37.4			
1/30/17	17:30			35.7		
1/30/17	17:30		35.7			
1/31/17	5:58			32.2		
1/31/17	5:58		32.2			
1/31/17	10:30			31.5		

PROCTOR NO:
MDD:
OMC:
PERCENT COMPACTION:

Height of Top of Specimen from Top of Table: 7.0 *cm*
Stand Pipe Diameter: 1.05 *cm*

SAMPLE CALCULATIONS

$$k = \{[(a * L)/(A * t)] * \ln[ho/hf]\}$$

Where k = *permeability in cm/sec*
a = *area of standpipe in sq cm*
L = *length of specimen in cm*
A = *area of specimen in sq cm*
t = *elapsed time in seconds*
ho = *initial height in cm* [1]
hf = *final height in cm* [1]
$\ln$ = *natural logarithm*

[1] *Corrected for height of specimen from top of table during computations*

CLIENT:

REPORT DATE: 2/1/2017

PROJECT NO.: 0771-365-11-07-24

PROJECT: Hardin County Expansion

HYDRAULIC CONDUCTIVITY WORKSHEET
FALLING HEAD - FIXED WALL PERMEAMETER

LOCATION:
MATERIAL:
BORING/SAMPLE: WC-2
PROCTOR #:
SAMPLE ORIENTATION: H _____ V ✓
Remold _____

LAB START DATE: 1/29/2017
LAB RPT. DATE: 2/1/2017
TECHNICIAN: MLT
DEPTH/LIFT: 20.0'
PERM FLUID USED: De-aired Tap Water

a. Length of Specimen, L: 1.0 in
c. Sample Volume
($\pi b^2 / 4 * a$): 4.909 cu in

b. Avg. Diameter of Specimen: 2.5 in
d. Wet Unit Weight:
[((f-h)*3.8095)/c)]: 119.1 pcf

INITIAL CONDITIONS		FINAL CONDITIONS	
e. Ring + Wet Weight Soil:	669.1 gms	k. Wet Weight Soil + Tare:	251.7 gms
f. Wet Weight Soil + Tare:	245.7 gms	l. Dry Weight Soil + Tare:	210.5 gms
g. Dry Weight Soil + Tare:	210.5 gms	m. Tare Weight:	92.2 gms
h. Tare Weight:	92.2 gms	n. Moisture Content [(k-l)/(l-m)]*100:	34.8 %
i. Moisture Content [(f-g)/(g-h)]*100:	29.8 %	o. Unit Dry Weight [d/(1+(n/100))]:	88.4 pcf
j. Unit Dry Weight [d/(1+(i/100))]:	91.8 pcf	p. Ring Weight:	515.6 gms

Date	Time	t sec	Initial Height, ho	Corrected ho - C	Final Height, hf	Corrected hf - C	Temp C	Rt	k @ 20C cm/sec
29-Jan	14:37		44.5	37.0					
30-Jan	06:40	57780			41.2	33.7	22	0.953	1.1E-07
30-Jan	06:40		41.2	33.7					
30-Jan	10:40	14400			40.5	33.0	22	0.953	9.6E-08
30-Jan	10:40		40.5	33.0					
30-Jan	17:30	24600			39.3	31.8	22	0.953	1.0E-07
30-Jan	17:30		39.3	31.8					
31-Jan	05:58	44880			37.3	29.8	22	0.953	9.6E-08
31-Jan	05:58		37.3	29.8					
31-Jan	10:30	16320			36.6	29.1	22	0.953	9.6E-08
30-Jan	06:40		41.2	33.7					
31-Jan	10:30	100200			36.6	29.1	22	0.953	9.7E-08

Height of Top of Specimen
From Top of Table: 7.50 cm

Standpipe Diameter
1.05 cm

Standpipe Area
0.866 sq cm

Test Method: Corps of Engineers EM 1110-2-1906, Appendix VII

Hx-C = Hx-Ht

HYDRAULIC CONDUCTIVITY WORKSHEET
Falling Head - Fixed Wall Permeameter

PROJECT: Hardin County Expansion

LOCATION:

MATERIAL:

BORING/SAMPLE NO: WC-2

SAMPLE ORIENTATION H [✓ V] R (Circle One)

a. HEIGHT: 1.0 in

c. VOLUME: (0.7854 * b ^ 2) cu in

JOB NO: 0771-365-11-07-24

DATE: 1/29/17

TECHNICIAN: MLT

DEPTH/LIFT: 20'

PERM FLUID USED: Tap Water

b. AVERAGE DIAMETER: 2.5 in

d. WET UNIT WEIGHT: [(g-i * 3.8095)/c] pcf

INITIAL CONDITIONS

e. Ring Wt: 515.6 gms

f. Ring Wt + Wet Soil: 669.1 gms

g. Wet Wt Soil + tare [f-e+i]: gms

h. Dry Wt Soil + tare: gms

i. Tare Wt: gms

j. Moisture Content [(g-h)/(h-i)]*100 %

k. Unit Dry Wt [d/(1 + (j/100))] pcf

FINAL CONDITIONS

l. Wet Wt. Soil + tare: 251.7 gms

m. Dry Wt. Soil + tare: 210.5 gms

n. Tare Wt: MM 92.2 gms

o. Moisture Content [(l-m)/(m-n)]*100 %

p. Unit Dry Wt. [d/(1 + (o/100))] pcf

DATE	CLOCK TIME	TIME SECONDS	INITIAL HEIGHT, ho	FINAL HEIGHT, hf	TEMP °C	PERMEABILITY k, cm/sec
1/29/17	14:37		44.5			
1/30/17	6:40			41.2		
1/30/17	6:40		41.2			
1/30/17	10:40			40.5		
1/30/17	10:40		40.5			
1/30/17	17:30			39.3		
1/30/17	17:30		39.3			
1/31/17	5:58			37.3		
1/31/17	5:58		37.3			
1/31/17	10:30			36.6		

PROCTOR NO:

MDD:

OMC:

PERCENT COMPACTION:

Height of Top of Specimen from Top of Table: 7.5 *cm*

Stand Pipe Diameter: 1.05 *cm*

SAMPLE CALCULATIONS

$k = \{[(a * L)/(A * t)] * \ln[ho/hf]\}$

Where k = *permeability in cm/sec*

a = *area of standpipe in sq cm*

L = *length of specimen in cm*

A = *area of specimen in sq cm*

t = *elapsed time in seconds*

ho = *initial height in cm* [1]

hf = *final height in cm* [1]

$\ln$ = *natural logarithm*

[1] *Corrected for height of specimen from top of table during computations*

CLIENT:

REPORT DATE: 2/1/2017

PROJECT NO.: 0771-365-11-07-24

PROJECT: Hardin County Expansion

HYDRAULIC CONDUCTIVITY WORKSHEET
FALLING HEAD - FIXED WALL PERMEAMETER

LOCATION:
MATERIAL:
BORING/SAMPLE: WC-3
PROCTOR # :
SAMPLE ORIENTATION: H ______ V ✓
Remold ______

LAB START DATE: 1/29/2017
LAB RPT. DATE: 2/1/2017
TECHNICIAN: MLT
DEPTH/LIFT: 28.0'
PERM FLUID USED: De-aired Tap Water

a. Length of Specimen, L: 1.0 in
c. Sample Volume (π b^2 / 4 * a): 4.909 cu in

b. Avg. Diameter of Specimen: 2.5 in
d. Wet Unit Weight: [((f-h)*3.8095)/c)]: 118.0 pcf

INITIAL CONDITIONS

e. Ring + Wet Weight Soil: 678.4 gms
f. Wet Weight Soil + Tare: 247.6 gms
g. Dry Weight Soil + Tare: 211.8 gms
h. Tare Weight: 95.6 gms
i. Moisture Content [(f-g)/(g-h)]*100: 30.8 %
j. Unit Dry Weight [d/(1+(i/100))]: 90.2 pcf

FINAL CONDITIONS

k. Wet Weight Soil + Tare: 251.4 gms
l. Dry Weight Soil + Tare: 211.8 gms
m. Tare Weight: 95.6 gms
n. Moisture Content [(k-l)/(l-m)]*100: 34.1 %
o. Unit Dry Weight [d/(1+(n/100))]: 88.0 pcf
p. Ring Weight: 526.4 gms

Date	Time	t sec	Initial Height, ho	Corrected ho - C	Final Height, hf	Corrected hf - C	Temp C	Rt	k @ 20C cm/sec
29-Jan	14:13		45.3	37.8					
30-Jan	06:40	59220			44.1	36.6	22	0.953	3.6E-08
30-Jan	06:40		44.1	36.6					
30-Jan	10:40	14400			44.0	36.5	22	0.953	1.3E-08
30-Jan	10:40		44.0	36.5					
30-Jan	17:30	24600			43.9	36.4	22	0.953	7.4E-09
30-Jan	17:30		43.9	36.4					
31-Jan	05:58	44880			43.6	36.1	22	0.953	1.2E-08
31-Jan	05:58		43.6	36.1					
31-Jan	10:30	16320			43.5	36.0	22	0.953	1.1E-08
30-Jan	06:40		44.1	36.6					
31-Jan	10:30	100200			43.5	36.0	22	0.953	1.1E-08

Height of Top of Specimen From Top of Table:	Standpipe Diameter	Standpipe Area
7.50 cm	1.05 cm	0.866 sq cm

Test Method: Corps of Engineers EM 1110-2-1906, Appendix VII

Hx-C = Hx-Ht

HYDRAULIC CONDUCTIVITY WORKSHEET
Falling Head - Fixed Wall Permeameter

PROJECT: Hardin County Expansion
LOCATION:
MATERIAL: Cly Silty Rd-Br
BORING/SAMPLE NO: WC-3
SAMPLE ORIENTATION H [✓ V] R (Circle One)
a. HEIGHT: 1.0 in
c. VOLUME: (0.7854 * b ^ 2) cu in

JOB NO: 0771-356-11-07-24
DATE: 1/29/17
TECHNICIAN: MLT
DEPTH/LIFT: 28'
PERM FLUID USED: Tap Water
b. AVERAGE DIAMETER: 2.5 in
d. WET UNIT WEIGHT: [(g-i * 3.8095)/c] pcf

INITIAL CONDITIONS

e. Ring Wt: 526.4 gms
f. Ring Wt + Wet Soil: 678.4 gms
g. Wet Wt Soil + tare [f-e+i]: gms
h. Dry Wt Soil + tare: gms
i. Tare Wt: gms
j. Moisture Content [(g-h)/(h-i)]*100 %
k. Unit Dry Wt [d/(1 + (j/100))] pcf

FINAL CONDITIONS

l. Wet Wt. Soil + tare: 251.4 gms
m. Dry Wt. Soil + tare: 211.8 gms
n. Tare Wt: 501 95.6 gms
o. Moisture Content [(l-m)/(m-n)]*100 %
p. Unit Dry Wt. [d/(1 + (o/100))] pcf

DATE	CLOCK TIME	TIME SECONDS	INITIAL HEIGHT, ho	FINAL HEIGHT, hf	TEMP °C	PERMEABILITY k, cm/sec
1/29/17	14:13		45.3			
1/30/17	6:40			44.1		
1/30/17	6:40		44.1			
1/30/17	10:40			44.0		
1/30/17	10:40		44.0			
1/30/17	17:30			43.9		
1/30/17	17:30		43.9			
1/31/17	5:58			43.6		
1/31/17	5:58		43.6			
1/31/17	10:30			43.5		

PROCTOR NO:
MDD:
OMC:
PERCENT COMPACTION:

Height of Top of Specimen from Top of Table: 7.5 *cm*
Stand Pipe Diameter: 1.05 *cm*

SAMPLE CALCULATIONS

*k = {[(a * L)/(A * t)] * ln[ho/hf]}*

Where k = permeability in cm/sec
a = area of standpipe in sq cm
L = length of specimen in cm
A = area of specimen in sq cm
t = elapsed time in seconds
ho = initial height in cm [1]
hf = final height in cm [1]
ln = natural logarithm

[1] *Corrected for height of specimen from top of table during computations*

CLIENT:

REPORT DATE: 2/1/2017

PROJECT NO.: 0771-365-11-07-24

PROJECT: Hardin County Expansion

HYDRAULIC CONDUCTIVITY WORKSHEET
FALLING HEAD - FIXED WALL PERMEAMETER

LOCATION:
MATERIAL:
BORING/SAMPLE: WC-5
PROCTOR # :
SAMPLE ORIENTATION: H _____ V ✓
Remold _____

LAB START DATE: 1/29/2017
LAB RPT. DATE: 2/1/2017
TECHNICIAN: MLT
DEPTH/LIFT: 39.0'
PERM FLUID USED: De-aired Tap Water

a. Length of Specimen, L: 1.0 in
c. Sample Volume
(π b^2 / 4 * a): 4.909 cu in

b. Avg. Diameter of Specimen: 2.5 in
d. Wet Unit Weight:
[((f-h)*3.8095)/c)]: 117.5 pcf

INITIAL CONDITIONS

e. Ring + Wet Weight Soil: 671.6 gms
f. Wet Weight Soil + Tare: 245.3 gms
g. Dry Weight Soil + Tare: 208.0 gms
h. Tare Weight: 93.9 gms
i. Moisture Content
[(f-g)/(g-h)]*100: 32.7 %
j. Unit Dry Weight
[d/(1+(i/100))]: 88.5 pcf

FINAL CONDITIONS

k. Wet Weight Soil + Tare: 246.9 gms
l. Dry Weight Soil + Tare: 208.0 gms
m. Tare Weight: 93.9 gms
n. Moisture Content
[(k-l)/(l-m)]*100: 34.1 %
o. Unit Dry Weight
[d/(1+(n/100))]: 87.6 pcf
p. Ring Weight: 520.2 gms

Date	Time	t sec	Initial Height, ho	Corrected ho - C	Final Height, hf	Corrected hf - C	Temp C	Rt	k @ 20C cm/sec
29-Jan	13:54		38.2	30.7					
30-Jan	06:40	60360			37.7	30.2	22	0.953	1.8E-08
30-Jan	06:40		37.7	30.2					
30-Jan	10:40	14400			37.7	30.2	22	0.953	<1.00E-09
30-Jan	10:40		37.7	30.2					
30-Jan	17:30	24600			37.6	30.1	22	0.953	8.7E-09
30-Jan	17:30		37.6	30.1					
31-Jan	05:58	44880			37.3	29.8	22	0.953	1.4E-08
31-Jan	05:58		37.3	29.8					
31-Jan	10:30	16320			37.2	29.7	22	0.953	1.3E-08
30-Jan	06:40		37.7	30.2					
31-Jan	10:30	100200			37.2	29.7	22	0.953	1.1E-08

Height of Top of Specimen
From Top of Table: 7.48 cm

Standpipe Diameter
1.04 cm

Standpipe Area
0.849 sq cm

Test Method: Corps of Engineers EM 1110-2-1906, Appendix VII

Hx-C = Hx-Ht

HYDRAULIC CONDUCTIVITY WORKSHEET
Falling Head - Fixed Wall Permeameter

PROJECT: Hardin County BExpansion

LOCATION:

MATERIAL: Si Cly br

BORING/SAMPLE NO: WSC-5

SAMPLE ORIENTATION H [✓ V] R (Circle One)

a. HEIGHT: 1.0 in

c. VOLUME: (0.7854 * b ^ 2) ______ cu in

JOB NO: 0771-356-11-07-24

DATE: 1/29/17

TECHNICIAN: MLT

DEPTH/LIFT: 39'

PERM FLUID USED: Tap Water

b. AVERAGE DIAMETER: 2.5 in

d. WET UNIT WEIGHT: [(g-i * 3.8095)/c] ______ pcf

INITIAL CONDITIONS

e. Ring Wt: 520.2 gms

f. Ring Wt + Wet Soil: 671.6 gms

g. Wet Wt Soil + tare [f-e+i]: ______ gms

h. Dry Wt Soil + tare: ______ gms

i. Tare Wt: ______ gms

j. Moisture Content [(g-h)/(h-i)]*100 ______ %

k. Unit Dry Wt [d/(1 + (j/100))] ______ pcf

FINAL CONDITIONS

l. Wet Wt. Soil + tare: 246.9 gms

m. Dry Wt. Soil + tare: 208.0 gms

n. Tare Wt: B-19 93.9 gms

o. Moisture Content [(l-m)/(m-n)]*100 ______ %

p. Unit Dry Wt. [d/(1 + (o/100))] ______ pcf

DATE	CLOCK TIME	TIME SECONDS	INITIAL HEIGHT, ho	FINAL HEIGHT, hf	TEMP °C	PERMEABILITY k, cm/sec
1/29/17	13:54		38.2			
1/30/17	6:40			37.7		
1/30/17	6:40		37.7			
1/30/17	10:40			37.7		
1/30/17	10:40		37.7			
1/30/17	17:30			37.6		
1/30/17	17:30		37.6			
1/31/17	5:58			37.3		
1/31/17	5:58		37.3			
1/31/17	10:30			37.2		

PROCTOR NO: ______

MDD: ______

OMC: ______

PERCENT COMPACTION: ______

Height of Top of Specimen from Top of Table: 7.48 cm

Stand Pipe Diameter: 1.04 cm

SAMPLE CALCULATIONS

*k = {[(a * L)/(A * t)] * ln[ho/hf]}*

Where k = permeability in cm/sec

a = area of standpipe in sq cm

L = length of specimen in cm

A = area of specimen in sq cm

t = elapsed time in seconds

ho = initial height in cm [1]

hf = final height in cm [1]

ln = natural logarithm

[1] *Corrected for height of specimen from top of table during computations*

CLIENT:

REPORT DATE: 1/14/2017

PROJECT NO.: 0771-365-11-07-24

PROJECT: Hardin County Expansion

HYDRAULIC CONDUCTIVITY WORKSHEET
FALLING HEAD - FIXED WALL PERMEAMETER

LOCATION:
MATERIAL: Sandy clay, gray
BORING/SAMPLE: WC-5
PROCTOR # :
SAMPLE ORIENTATION: H _____ [✓ V]
Remold _____

LAB START DATE: 1/12/2017
LAB RPT. DATE: 1/14/2017
TECHNICIAN: MLT
DEPTH/LIFT: 97.0'
PERM FLUID USED: De-aired Tap Water

a. Length of Specimen, L: 1.0 in
c. Sample Volume
(π b^2 / 4 * a): 4.909 cu in

b. Avg. Diameter of Specimen: 2.5 in
d. Wet Unit Weight:
[((f-h)*3.8095)/c)]: 131.2 pcf

INITIAL CONDITIONS

e. Ring + Wet Weight Soil: 689.3 gms
f. Wet Weight Soil + Tare: 259.4 gms
g. Dry Weight Soil + Tare: 235.0 gms
h. Tare Weight: 90.3 gms
i. Moisture Content
[(f-g)/(g-h)]*100: 16.9 %
j. Unit Dry Weight
[d/(1+(i/100))]: 112.3 pcf

FINAL CONDITIONS

k. Wet Weight Soil + Tare: 262.9 gms
l. Dry Weight Soil + Tare: 235.0 gms
m. Tare Weight: 90.3 gms
n. Moisture Content
[(k-l)/(l-m)]*100: 19.3 %
o. Unit Dry Weight
[d/(1+(n/100))]: 110.0 pcf
p. Ring Weight: 520.2 gms

Date	Time	t sec	Initial Height, ho	Corrected ho - C	Final Height, hf	Corrected hf - C	Temp C	Rt	k @ 20C cm/sec
12-Jan	08:04		43.7	36.2					
12-Jan	22:30	51960			42.2	34.7	22	0.953	5.3E-08
12-Jan	22:30		42.2	34.7					
13-Jan	06:02	27120			41.8	34.3	22	0.953	2.8E-08
13-Jan	06:02		41.8	34.3					
13-Jan	12:31	23340			41.4	33.9	22	0.953	3.3E-08
13-Jan	12:31		41.4	33.9					
13-Jan	17:00	16140			41.2	33.7	22	0.953	2.4E-08
13-Jan	17:00		41.2	33.7					
13-Jan	21:30	16200			41.0	33.5	22	0.953	2.4E-08
12-Jan	22:30		42.2	34.7					
13-Jan	21:30	82800			41.0	33.5	22	0.953	2.8E-08

Height of Top of Specimen
From Top of Table: 7.48 cm

Standpipe Diameter
1.04 cm

Standpipe Area
0.849 sq cm

Test Method: Corps of Engineers EM 1110-2-1906, Appendix VII

Hx-C = Hx-Ht

HYDRAULIC CONDUCTIVITY WORKSHEET
Falling Head - Fixed Wall Permeameter

PROJECT: Hardin County Expansion
LOCATION:
MATERIAL: Sa Cl Gr
BORING/SAMPLE NO: WC-5
SAMPLE ORIENTATION H [✓ V] R (Circle One)
a. HEIGHT: 1.0 in
c. VOLUME: (0.7854 * b ^ 2) cu in

JOB NO: 0771-365-11-07-24
DATE: 1/12/17
TECHNICIAN: MLT
DEPTH/LIFT: 97'
PERM FLUID USED: Tap Water
b. AVERAGE DIAMETER: 2.5 in
d. WET UNIT WEIGHT: [(g-i * 3.8095)/c] pcf

INITIAL CONDITIONS

e. Ring Wt: 520.2 gms
f. Ring Wt + Wet Soil: 689.3 gms
g. Wet Wt Soil + tare [f-e+i]: gms
h. Dry Wt Soil + tare: gms
i. Tare Wt: gms
j. Moisture Content [(g-h)/(h-i)]*100 %
k. Unit Dry Wt [d/(1 + (j/100))] pcf

FINAL CONDITIONS

l. Wet Wt. Soil + tare: 262.9 gms
m. Dry Wt. Soil + tare: 235.0 gms
n. Tare Wt: C1 90.3 gms
o. Moisture Content [(l-m)/(m-n)]*100 %
p. Unit Dry Wt. [d/(1 + (o/100))] pcf

DATE	CLOCK TIME	TIME SECONDS	INITIAL HEIGHT, ho	FINAL HEIGHT, hf	TEMP °C	PERMEABILITY k, cm/sec
1/12/17	8:04		43.7			
1/12/17	22:30			42.2		
1/12/17	22:30		42.2			
1/13/17	6:02			41.8		
1/13/17	6:02		41.8			
1/13/17	12:31			41.4		
1/13/17	12:31		41.4			
1/13/17	17:00			41.2		
1/13/17	17:00		41.2			
1/13/17	21:30			41.0		

PROCTOR NO:
MDD:
OMC:
PERCENT COMPACTION:

Height of Top of Specimen from Top of Table: 7.48 *cm*
Stand Pipe Diameter: 1.04 *cm*

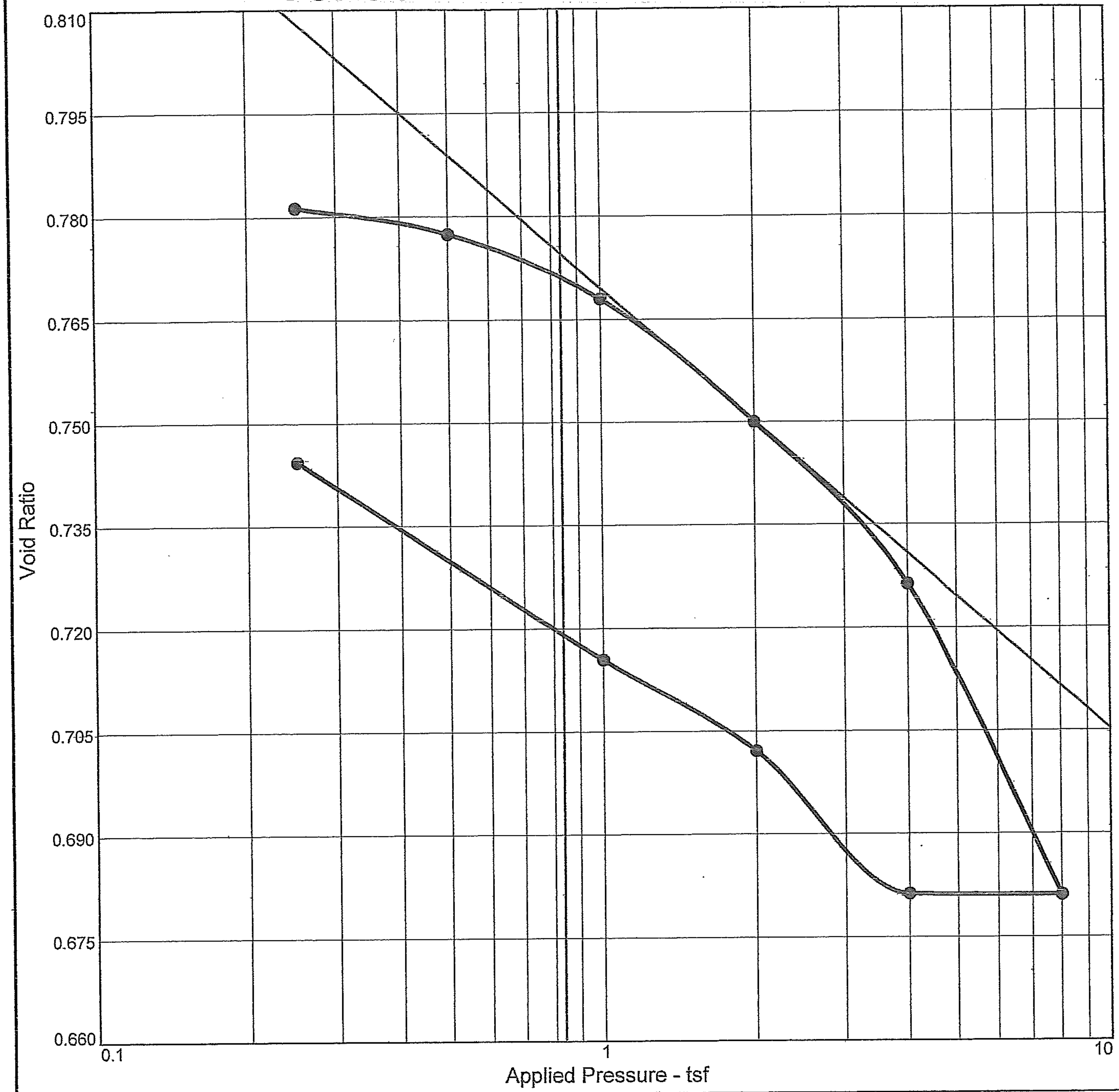
SAMPLE CALCULATIONS

*k = {[(a * L)/(A * t)] * ln[ho/hf]}*

Where k = permeability in cm/sec
a = area of standpipe in sq cm
L = length of specimen in cm
A = area of specimen in sq cm
t = elapsed time in seconds
ho = initial height in cm [1]
hf = final height in cm [1]
ln = natural logarithm

[1] Corrected for height of specimen from top of table during computations

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	P_c (tsf)	C_c	Initial Void Ratio
Saturation	Moisture							
96.7 %	28.5 %	92.9	65	41	2.65	0.99	0.06	0.781

MATERIAL DESCRIPTION	USCS	AASHTO
Silty clay, brown-red		

Project No. 0771-365- **Client:**

Project: Hardin County Expansion

Location: WC-3 **Depth:** 28.0'

M L Testing, LLC

Bluff Dale, TX

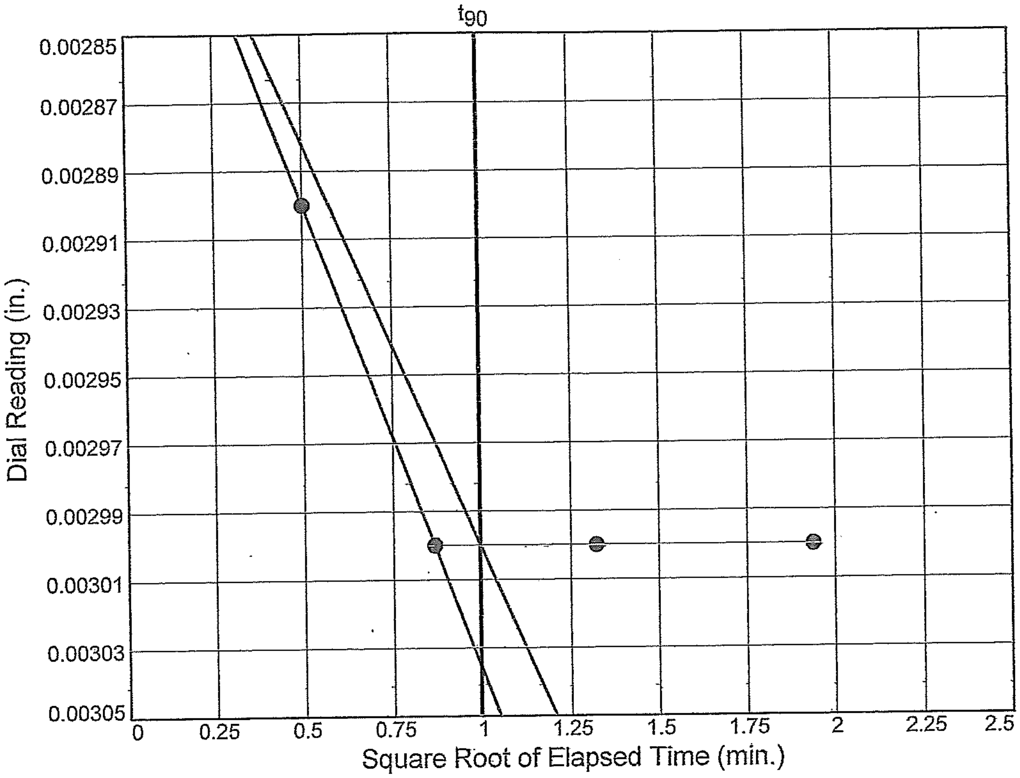
Remarks:

Figure

Dial Reading vs. Time

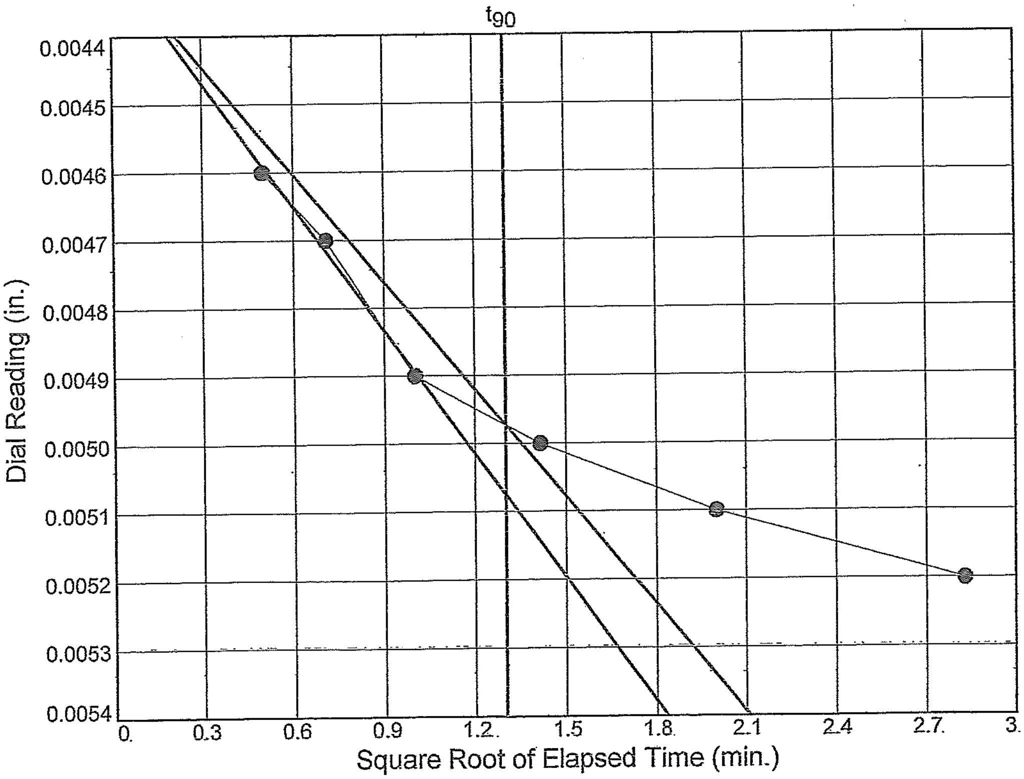
Project No.: 0771-365-11-07-24
Project: Hardin County Expansion

Location: WC-3 **Depth:** 28.0'



Load No.= 1
Load= 0.25 tsf
D_0 = 0.0028
D_{90} = 0.0030
D_{100} = 0.0030
T_{90} = 0.99 min.

C_v @ T_{90}
2.147 ft.^{2}/day



Load No.= 2
Load= 0.50 tsf
D_0 = 0.0043
D_{90} = 0.0050
D_{100} = 0.0050
T_{90} = 1.69 min.

C_v @ T_{90}
1.252 ft.^{2}/day

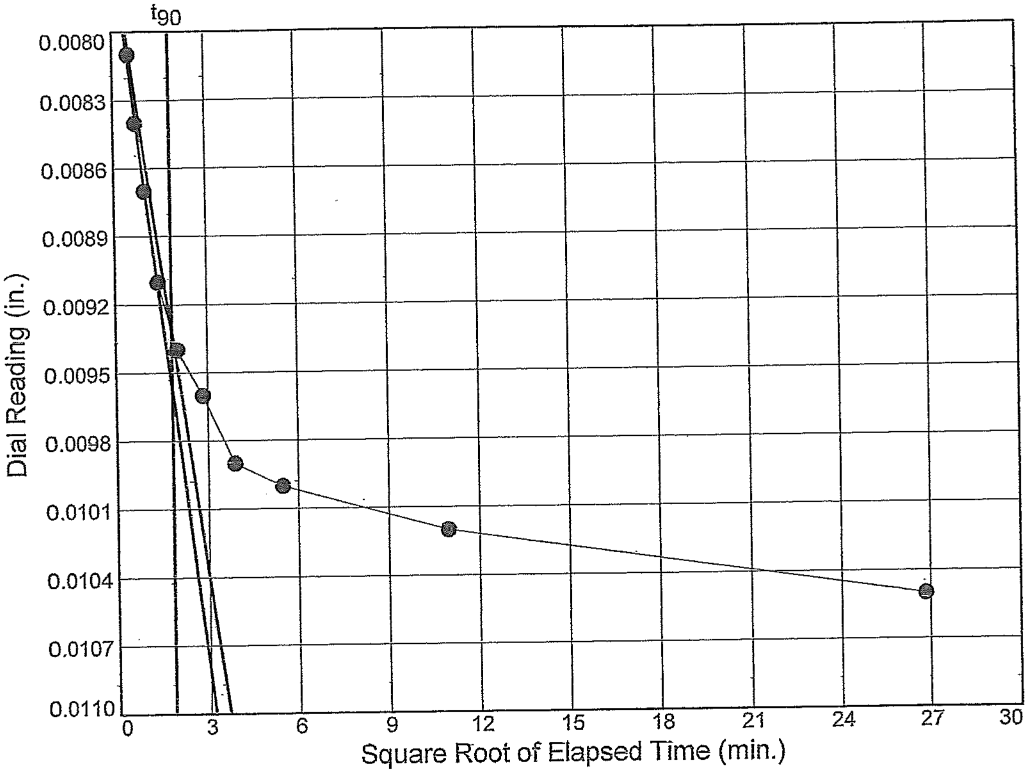
Figure

M L Testing, LLC

Dial Reading vs. Time

Project No.: 0771-365-11-07-24
Project: Hardin County Expansion

Location: WC-3 Depth: 28.0'



Load No.= 3

Load= 1.00 tsf

D_0 = 0.0076

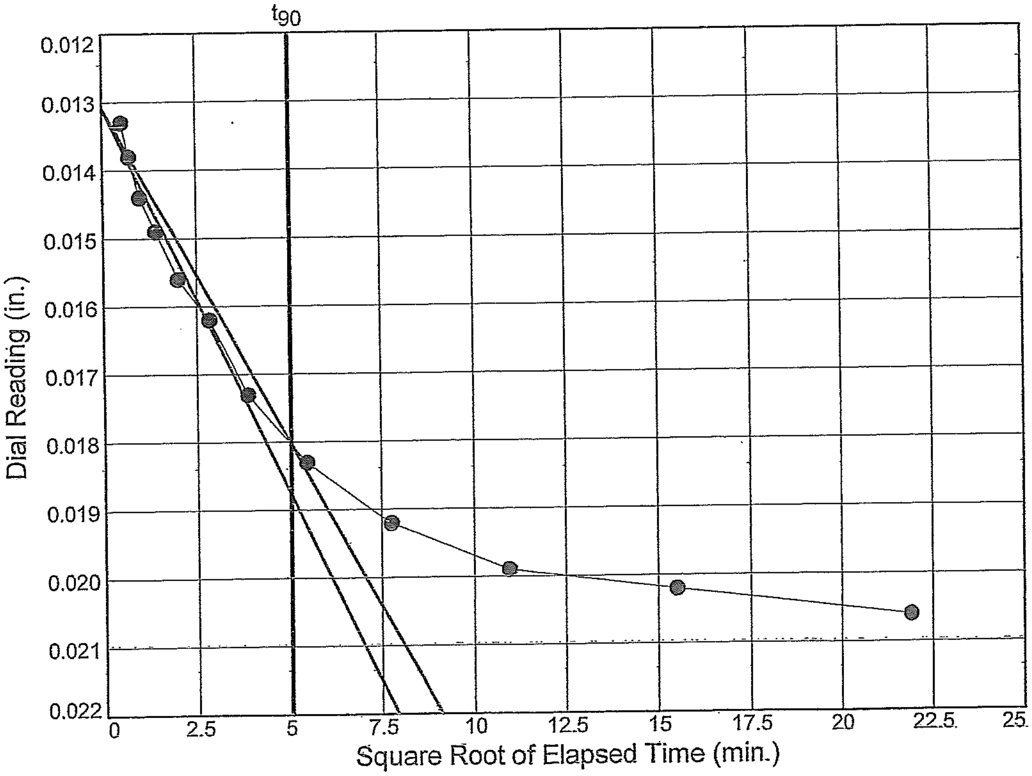
D_{90} = 0.0093

D_{100} = 0.0095

T_{90} = 3.36 min.

C_V @ T_{90}

0.625 ft.^{2}/day



Load No.= 4

Load= 2.00 tsf

D_0 = 0.0131

D_{90} = 0.0180

D_{100} = 0.0186

T_{90} = 25.66 min.

C_V @ T_{90}

0.081 ft.^{2}/day

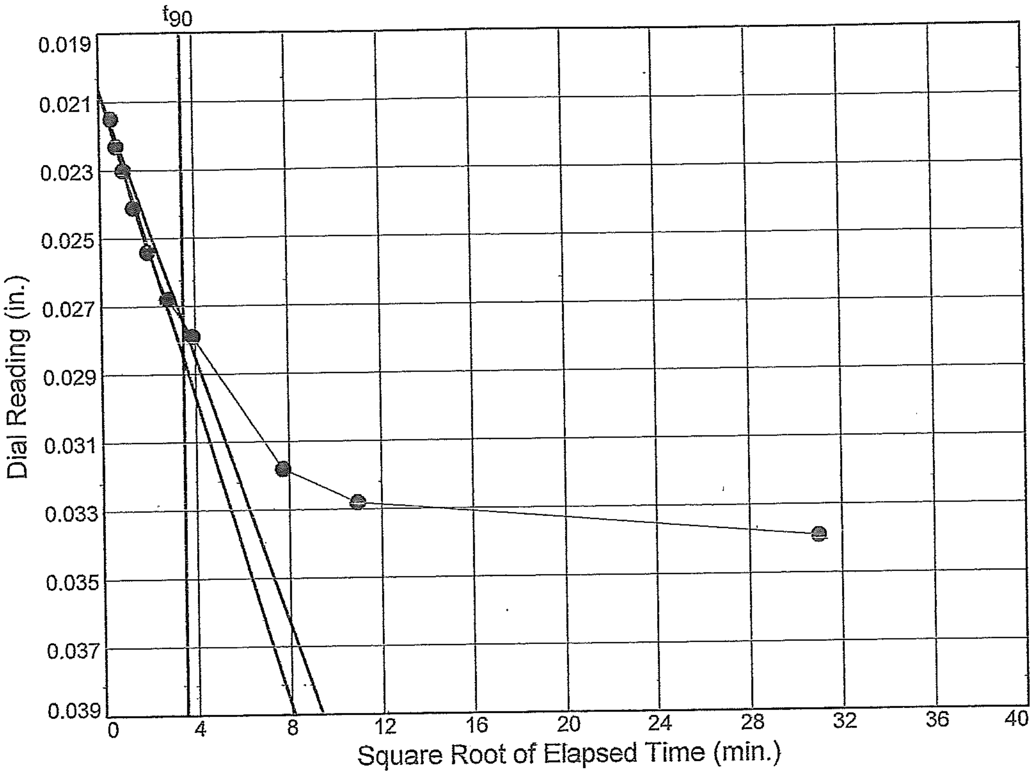
Figure

M L Testing, LLC

Dial Reading vs. Time

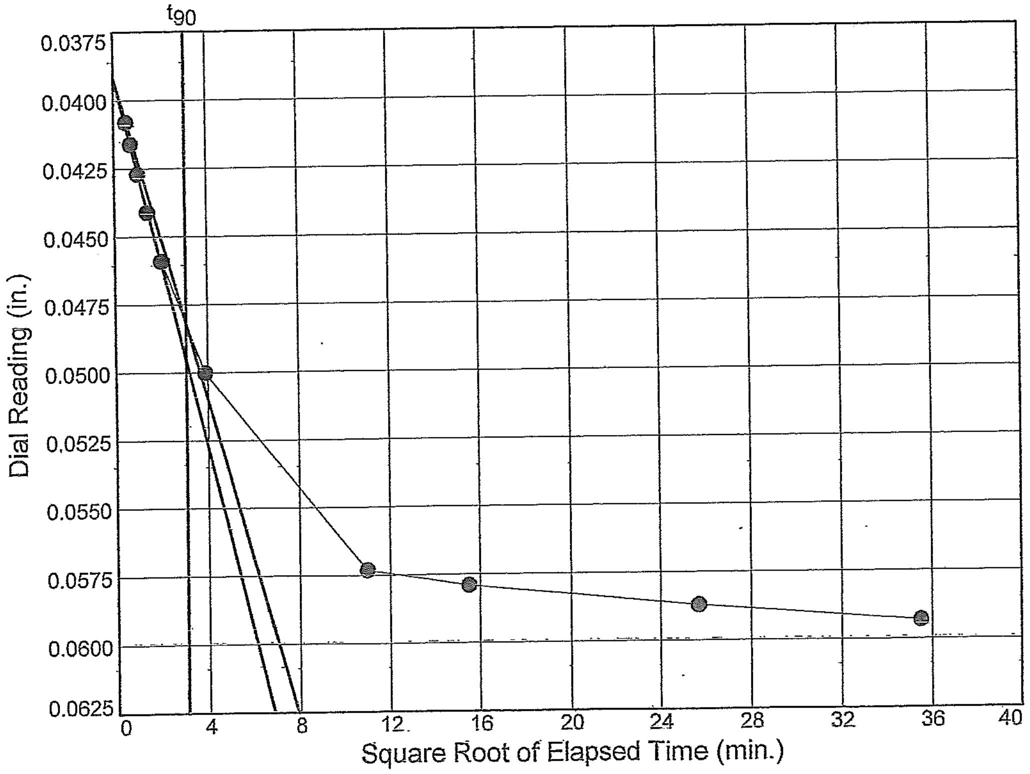
Project No.: 0771-365-11-07-24
Project: Hardin County Expansion

Location: WC-3 Depth: 28.0'



Load No.= 5
Load= 4.00 tsf
D_0 = 0.0207
D_{90} = 0.0275
D_{100} = 0.0282
T_{90} = 12.04 min.

C_V @ T_{90}
0.168 ft.^{2}/day



Load No.= 6
Load= 8.00 tsf
D_0 = 0.0392
D_{90} = 0.0482
D_{100} = 0.0492
T_{90} = 9.25 min.

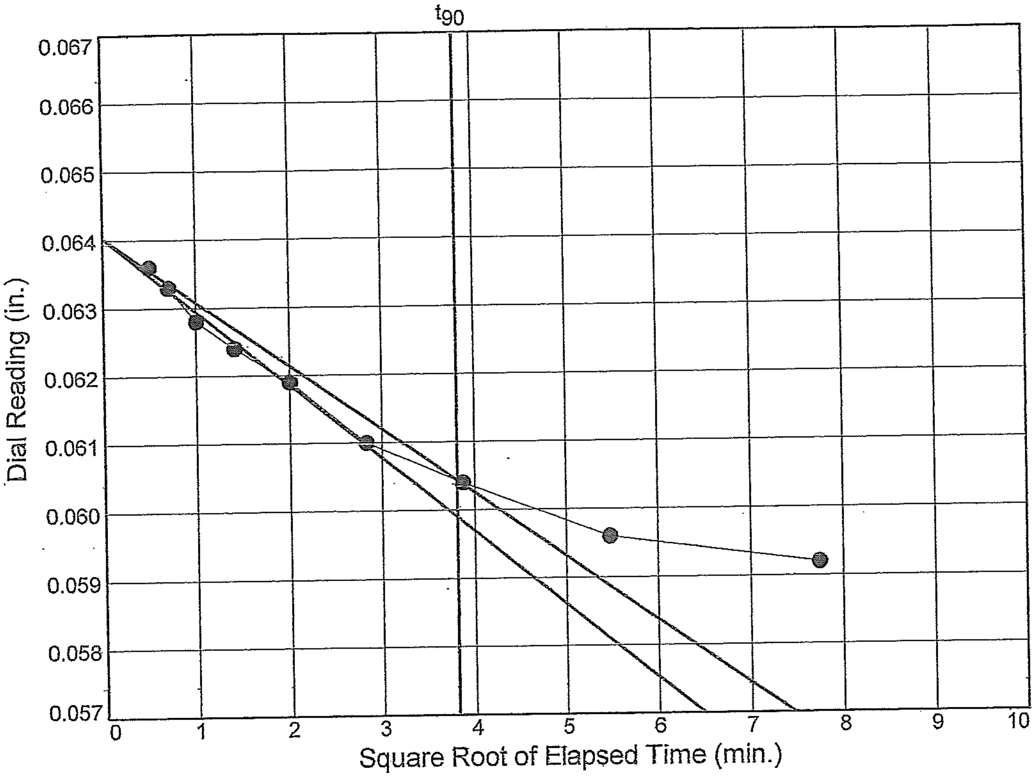
C_V @ T_{90}
0.210 ft.^{2}/day

Figure

Dial Reading vs. Time

Project No.: 0771-365-11-07-24
Project: Hardin County Expansion

Location: WC-3 Depth: 28.0'



Load No.= 7

Load= 4.00 tsf

D_0 = 0.0640

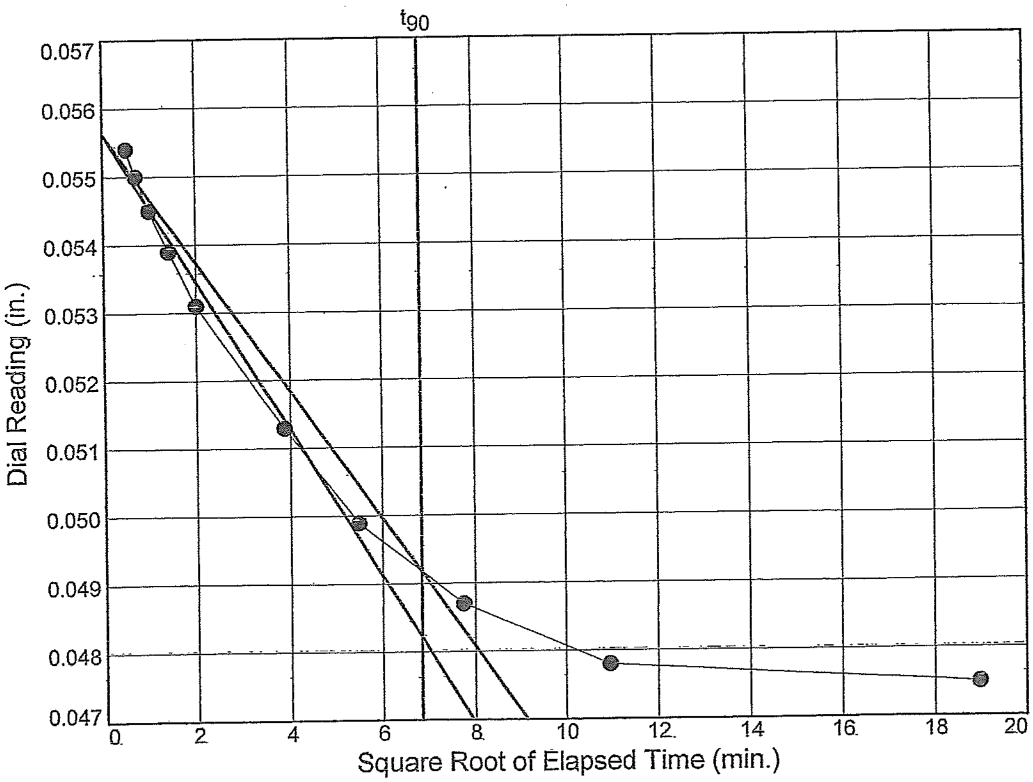
D_{90} = 0.0604

D_{100} = 0.0600

T_{90} = 14.55 min.

C_V @ T_{90}

0.130 ft.^{2}/day



Load No.= 8

Load= 2.00 tsf

D_0 = 0.0556

D_{90} = 0.0492

D_{100} = 0.0485

T_{90} = 46.55 min.

C_V @ T_{90}

0.041 ft.^{2}/day

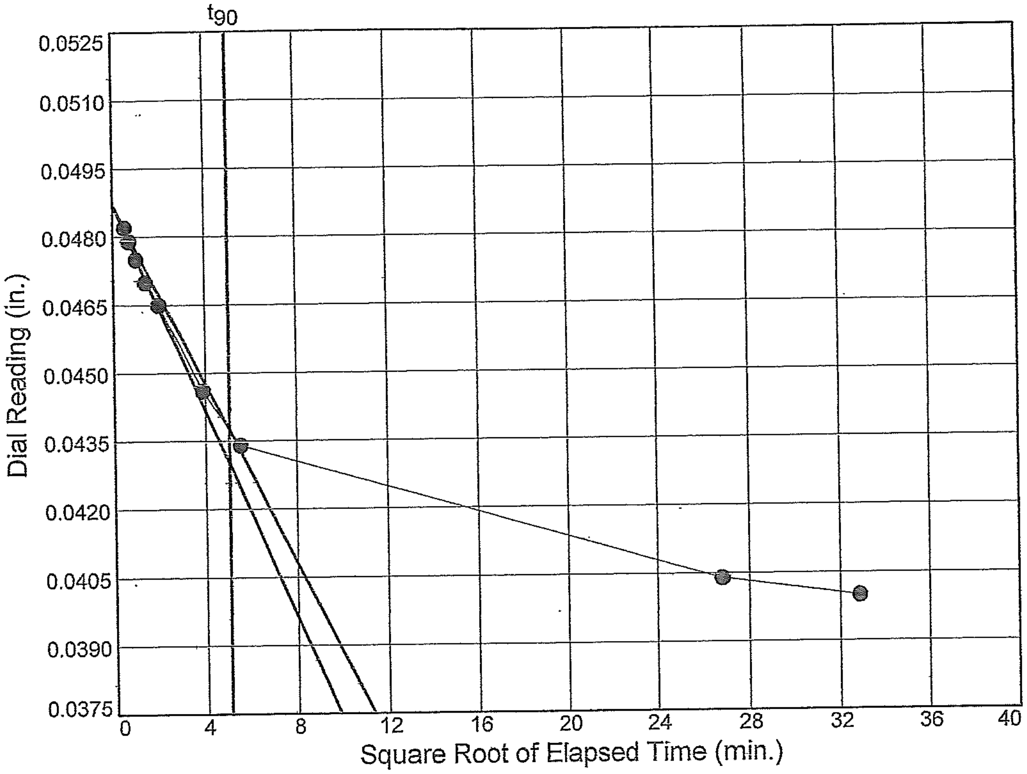
Figure

M L Testing, LLC

Dial Reading vs. Time

Project No.: 0771-365-11-07-24
Project: Hardin County Expansion

Location: WC-3 Depth: 28.0'



Load No.= 9

Load= 1.00 tsf

D_0 = 0.0487

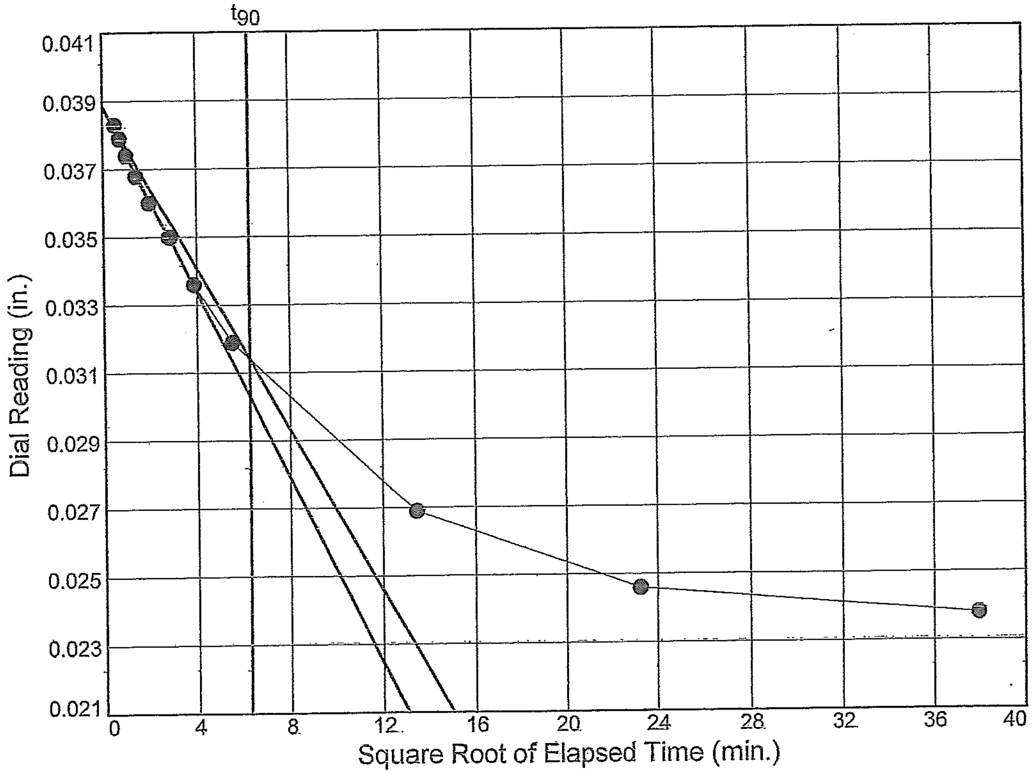
D_{90} = 0.0438

D_{100} = 0.0432

T_{90} = 25.04 min.

C_v @ T_{90}

0.078 ft.^{2}/day



Load No.= 10

Load= 0.25 tsf

D_0 = 0.0388

D_{90} = 0.0314

D_{100} = 0.0306

T_{90} = 38.90 min.

C_v @ T_{90}

0.051 ft.^{2}/day

Figure

CONSOLIDATION TEST DATA SHEET

Project : Hardin County LF
Location :
Material : Si Cly br-Rd
Boring No : WC-3

Project No : 0771-365-11-07-24
Date : 1/30/17
Technician : MLT
Sample Depth : 28'

SPECIFIC GRAVITY OF SOILS = SG = ____ ASSUMED ____ TEST RESULTS ____
HEIGHT OF SPECIMEN, cm 2.54
RING WEIGHT, gm 539.0

	BEFORE	AFTER
WET WEIGHT OF SAMPLE + RING + TARE, g	692.8	802.0
DRY WEIGHT OF SAMPLE + RING + TARE, g		765.5
TARE WEIGHT, g		106.8

INITIAL SOIL PARAMETERS

PERCENT SATURATION ____ VOID RATIO ____ POROSITY ____
WATER CONTENT ____ DRY DENSITY ____
CHANGE IN SAMPLE HEIGHT START OF TEST TO END TEST = ΔH = ____ cm

FINAL SOIL PARAMETERS

PERCENT SATURATION ____ VOID RATIO ____ POROSITY ____
WATER CONTENT ____ DRY DENSITY ____

LOAD		DIAL READING		TIME FOR 50% CONSOLIDATION	Δe	VOID RATIO	Cv
No.	tsf	U=0%	U=100%				
1	0.0625						
2	0.125						
3	0.25						
4	0.5						
5	1						
6	2						
7	4						
8	8						
9	12						
10	20						
REBOUND							
9	12						
8	8						
7	4						
6	2						
5	1						
4	0.5						
3	0.25						
2	0.125						
1	0.0625						

CONSOLIDATION DATA SHEET

LOADING TEST DATA

PROJECT: Hardin County Exp.

LOCATION:

MATERIAL:

BORING NO.: WC-3

JOB NO.: 0771-365-11-07-24

DATE: 1/29/17

TECHNICIAN:

DEPTH: 28'

LOAD 1/4 ton load

DATE APPLIED: 1/29/17

Clock Time and Date	Elapsed Time, min	Original
17:49	0	
	0.1	
	0.25	33
	0.5	33
	1	34
	2	34
	4	34
	8	
	15	
	30	
	60	
	120	
	240	
	480	
	960	
	1440	

LOAD 1/2 ton load

DATE APPLIED: 1/29/17

Clock Time and Date	Elapsed Time, min	Original
17:54	0	
	0.1	
	0.25	54
	0.5	55
	1	57
	2	58
	4	59
	8	60
	15	
	30	
	60	
	120	
	240	
	480	
	960	
	1440	

Dial Readings x ______

CONSOLIDATION DATA SHEET

LOADING TEST DATA

PROJECT: Hardin County Sxp

LOCATION:

MATERIAL:

BORING NO.: WC-3

JOB NO.: 0771-365-11-07-24

DATE:

TECHNICIAN: MLT

DEPTH: 28'

LOAD 1 ton load

DATE APPLIED: 1/29/17

Clock Time and Date	Elapsed Time, min	Original
18:02	0	
	0.1	
	0.25	97
	0.5	100
	1	103
	2	107
	4	110
	8	112
	15	115
	30	116
	60	
	120	118
	240	
	~~480~~ 720	121
	960	
	1440	

LOAD 2 ton load

DATE APPLIED: 1/30/17

Clock Time and Date	Elapsed Time, min	Original
6:49	0	
	0.1	
	0.25	157
	0.5	162
	1	168
	2	173
	4	180
	8	186
	15	197
	30	207
	60	216
	120	223
	240	226
	480	230
	960	
	1440	

Dial Readings x ______

CONSOLIDATION DATA SHEET

LOADING TEST DATA

PROJECT: Hardin County Exp.

JOB NO.: 0771-365-11-07-24

LOCATION:

DATE:

MATERIAL:

TECHNICIAN:

BORING NO.: WC-3

DEPTH: 28'

LOAD 4 ton load

DATE APPLIED: 1/30/17

Clock Time and Date	Elapsed Time, min	Original
17:28	0	
	0.1	
	0.25	285
	0.5	293
	1	300
	2	311
	4	324
	8	338
	15	349
	30	
	60	348
	120	398
	240	
	480	
	960	409
	1440	

LOAD 8 ton load

DATE APPLIED: 1/31/17

Clock Time and Date	Elapsed Time, min	Original
9:40	0	
	0.1	
	0.25	478
	0.5	486
	1	497
	2	511
	4	529
	8	
	15	570
	30	
	60	
	120	643
	~~240~~ 240	649
	~~480~~ 660	657
	960	
	~~1440~~ 1260	663

Dial Readings x ______

CONSOLIDATION DATA SHEET

LOADING TEST DATA

PROJECT: Hardin County Exp

JOB NO.: 0771-365-11-07-24

LOCATION:

DATE:

MATERIAL:

TECHNICIAN:

BORING NO.: WC-3

DEPTH: 28'

LOAD 1 ton rebound

DATE APPLIED: 2/1/17

Clock Time and Date	Elapsed Time, min	Original
13:58	0	
	0.1	
	0.25	492
	0.5	489
	1	485
	2	480
	4	475
	8	
	15	456
	30	444
	60	
	120	
	240	
	~~480~~ 720	414
	960	
	~~1440~~ 1680	410

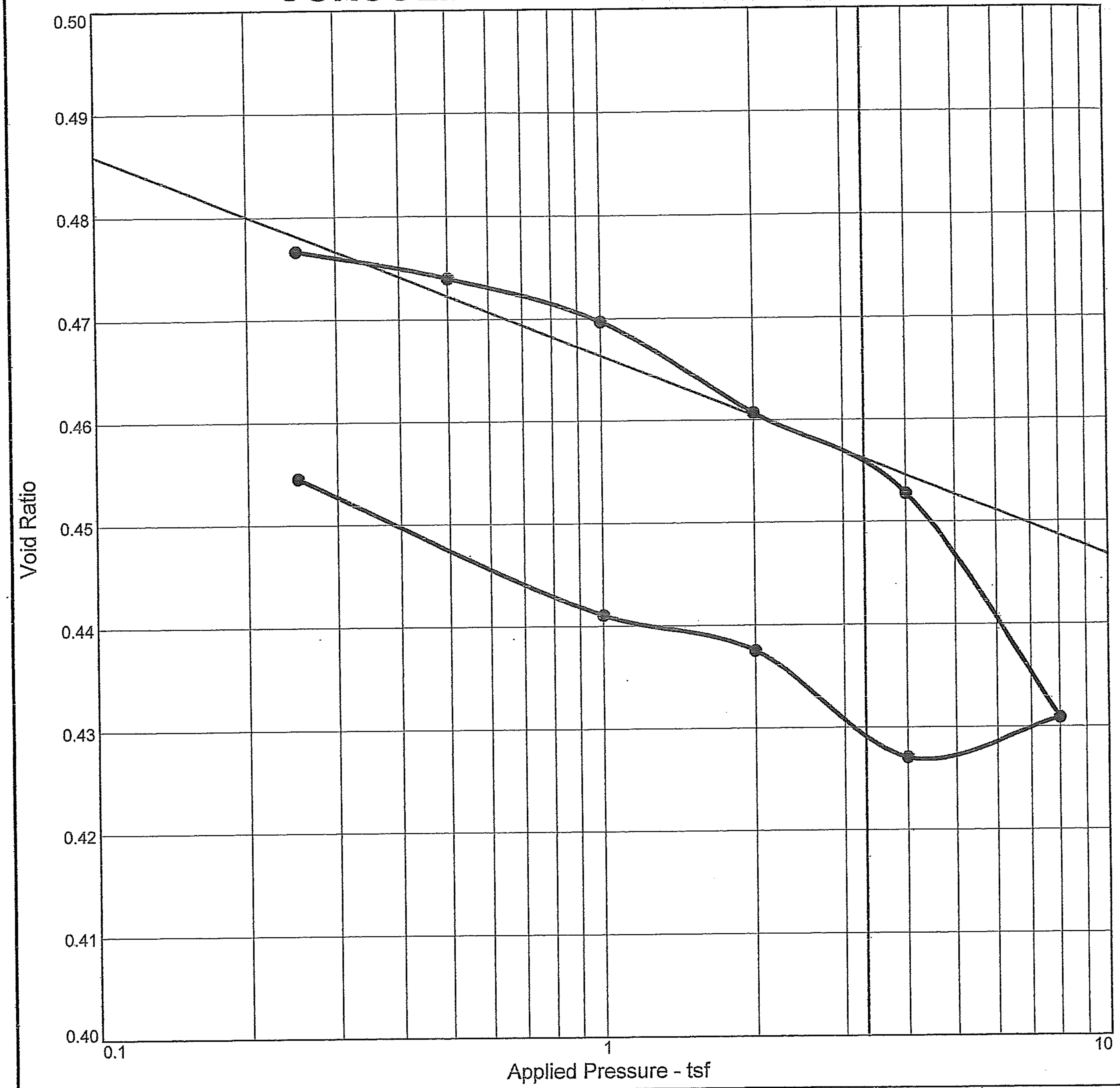
LOAD 1/4 ton rebound

DATE APPLIED: 2/2/17

Clock Time and Date	Elapsed Time, min	Original
8:11	0	
	0.1	
	0.25	393
	0.5	389
	1	384
	2	378
	4	370
	8	360
	15	346
	30	329
	60	
	~~120~~ 180	279
	240	
	~~480~~ 540	256
	960	
	1440	248

Dial Readings x ______

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	P_c (tsf)	C_c	Initial Void Ratio
Saturation	Moisture							
93.6 %	16.8 %	112.1	45	30	2.65	3.86	0.02	0.476

MATERIAL DESCRIPTION	USCS	AASHTO
Sandy clay, gray		

Project No. 0771-365- **Client:**

Project: Hardin County Expansion

Location: WC-5 **Depth:** 97.0'

M L Testing, LLC

Bluff Dale, TX

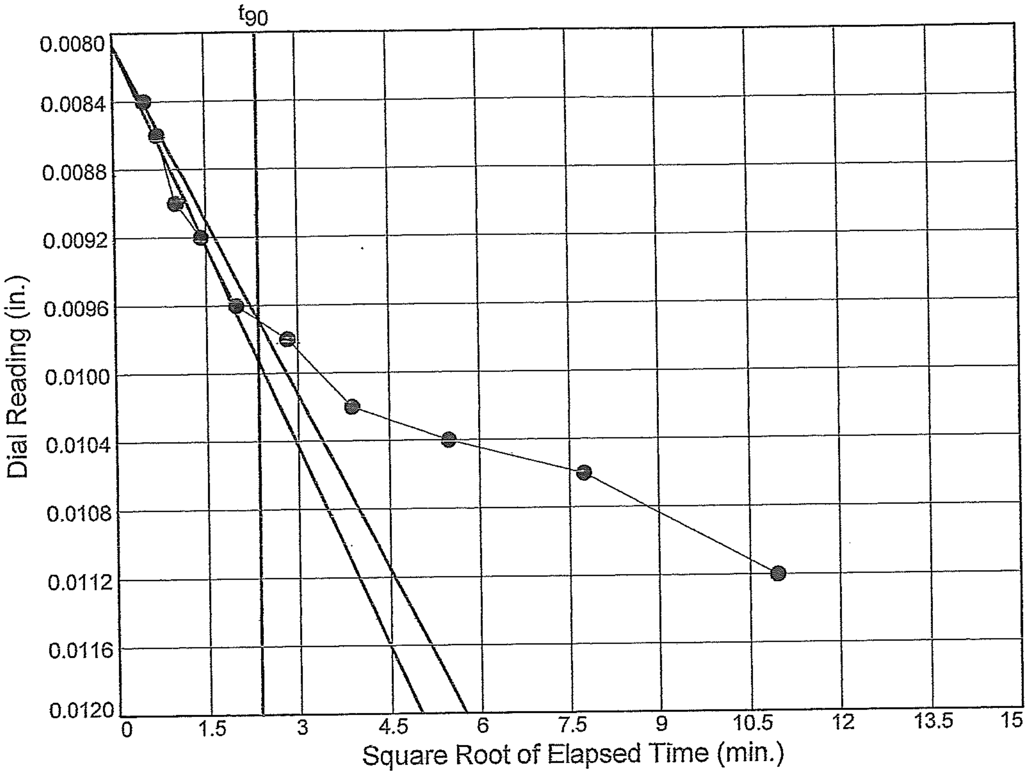
Remarks:

Figure

Dial Reading vs. Time

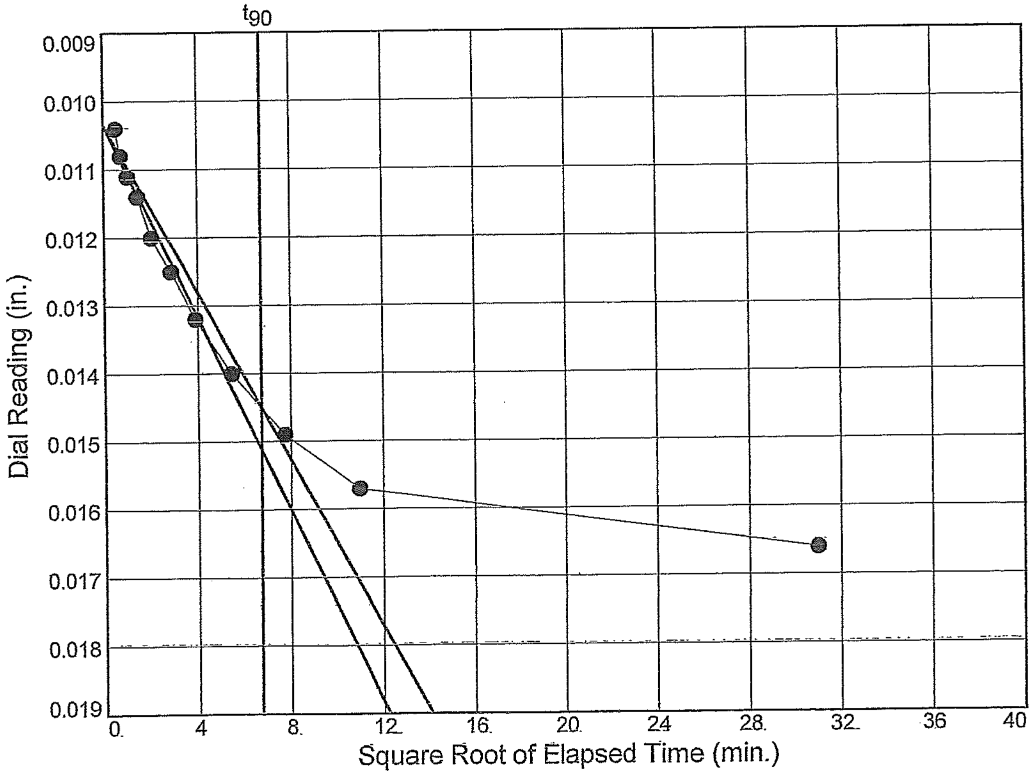
Project No.: 0771-365-11-07-24
Project: Hardin County Expansion

Location: WC-5 Depth: 97.0'



Load No.= 4
Load= 2.00 tsf
D_0 = 0.0081
D_{90} = 0.0097
D_{100} = 0.0099
T_{90} = 5.54 min.

C_v @ T_{90}
0.377 ft.^{2}/day



Load No.= 5
Load= 4.00 tsf
D_0 = 0.0104
D_{90} = 0.0145
D_{100} = 0.0150
T_{90} = 45.71 min.

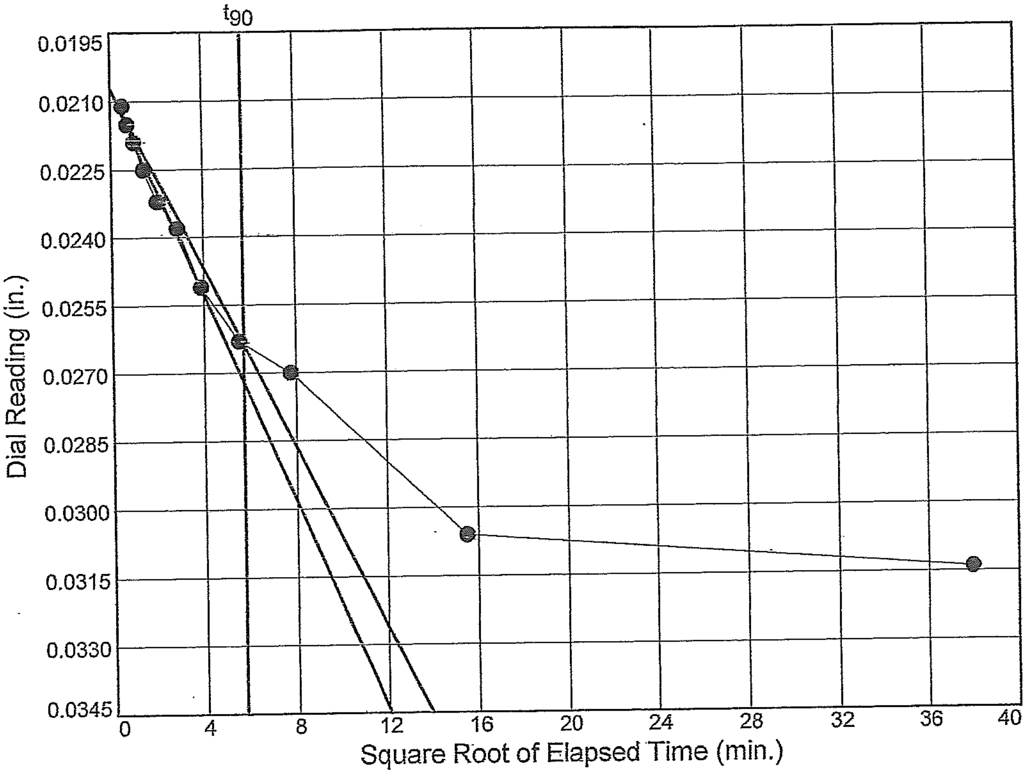
C_v @ T_{90}
0.045 ft.^{2}/day

Figure

Dial Reading vs. Time

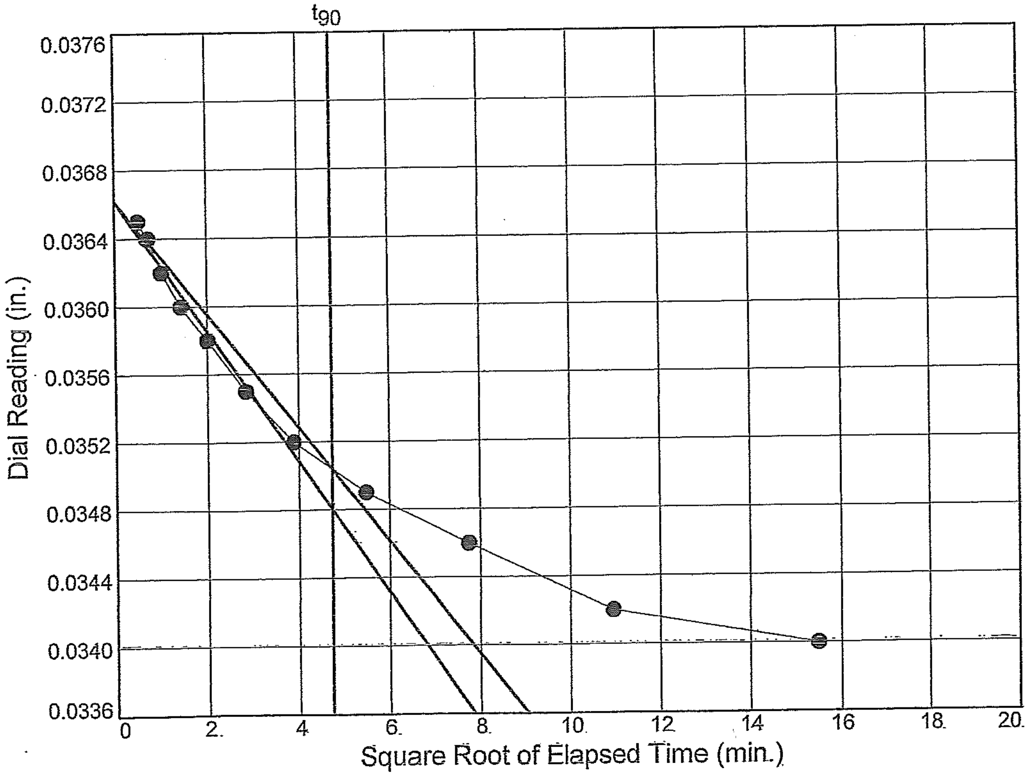
Project No.: 0771-365-11-07-24
Project: Hardin County Expansion

Location: WC-5 **Depth:** 97.0'



Load No.= 6
Load= 8.00 tsf
D_0 = 0.0207
D_{90} = 0.0264
D_{100} = 0.0270
T_{90} = 32.45 min.

C_V @ T_{90}
0.062 ft.^{2}/day



Load No.= 7
Load= 4.00 tsf
D_0 = 0.0366
D_{90} = 0.0350
D_{100} = 0.0349
T_{90} = 22.43 min.

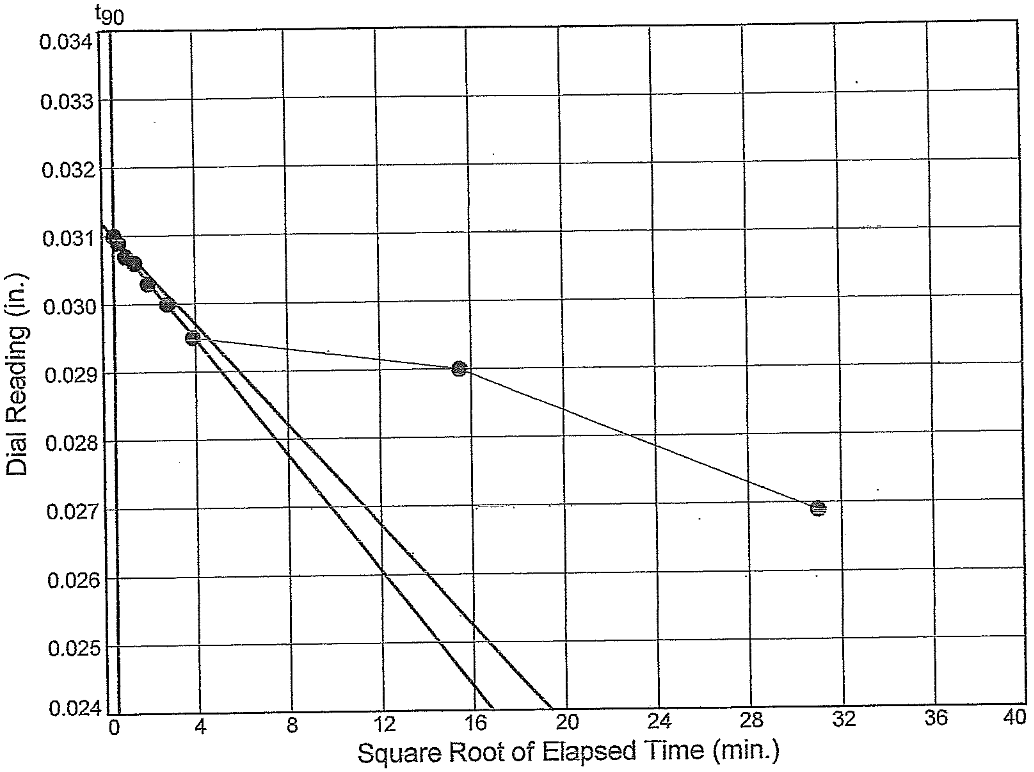
C_V @ T_{90}
0.089 ft.^{2}/day

Figure

Dial Reading vs. Time

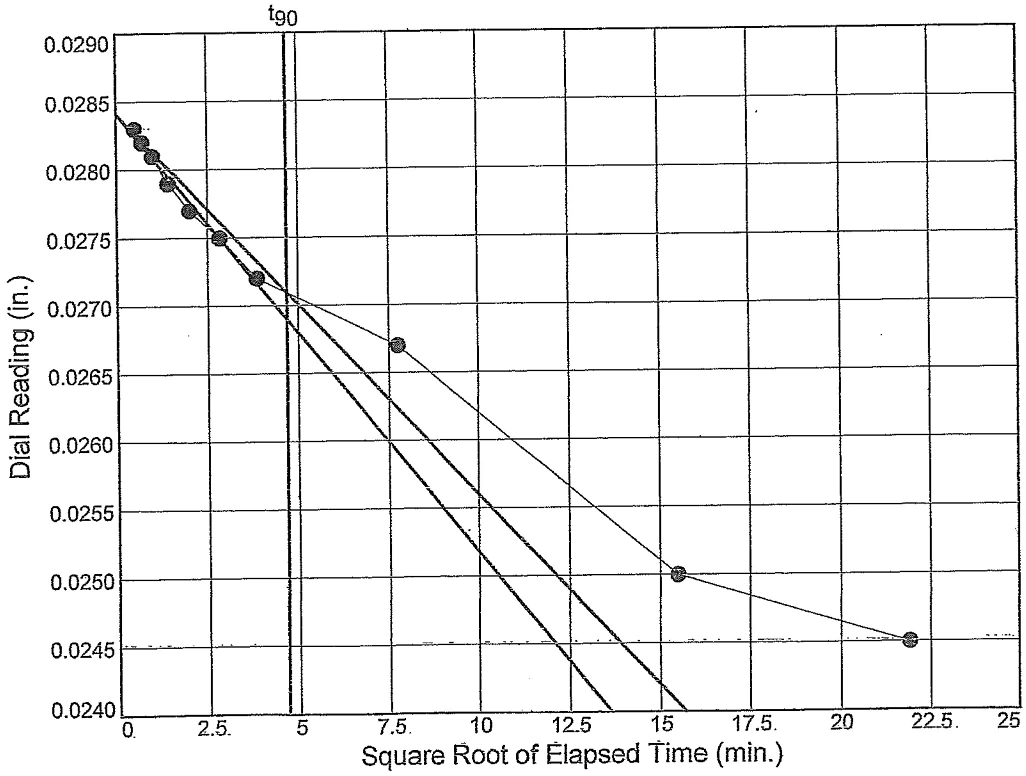
Project No.: 0771-365-11-07-24
Project: Hardin County Expansion

Location: WC-5 Depth: 97.0'



Load No.= 8
Load= 2.00 tsf
D_0 = 0.0312
D_{90} = 0.0310
D_{100} = 0.0310
T_{90} = 0.25 min.

C_v @ T_{90}
7.983 ft.^{2}/day



Load No.= 9
Load= 1.00 tsf
D_0 = 0.0284
D_{90} = 0.0271
D_{100} = 0.0269
T_{90} = 21.93 min.

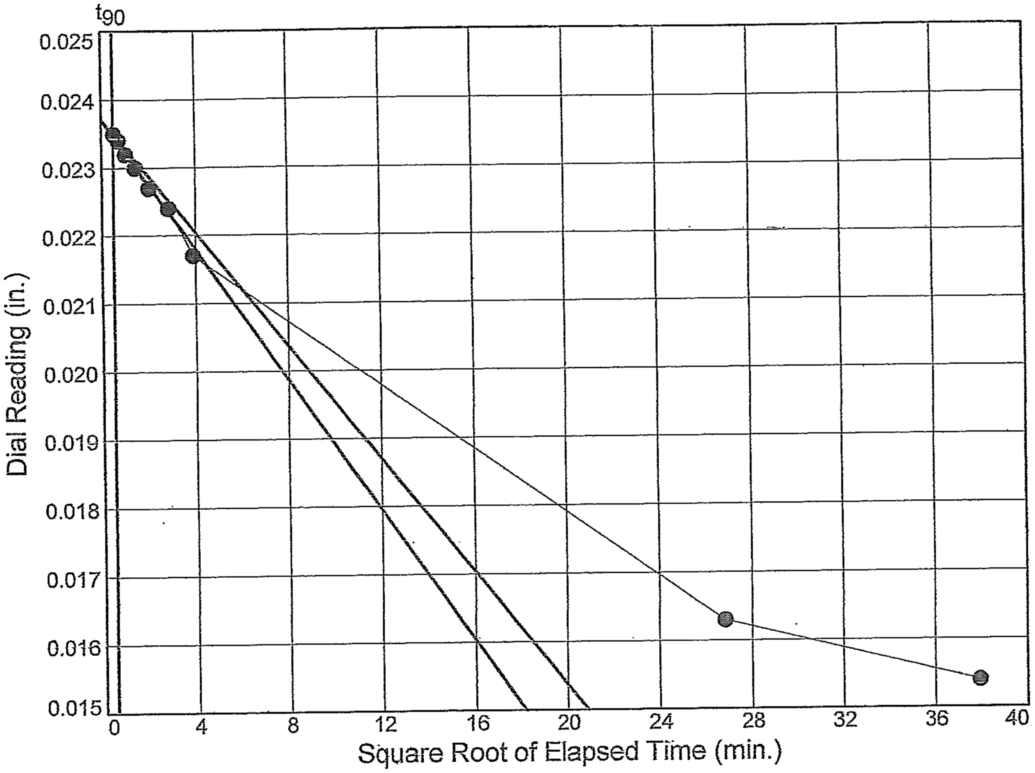
C_v @ T_{90}
0.092 ft.^{2}/day

Figure

Dial Reading vs. Time

Project No.: 0771-365-11-07-24
Project: Hardin County Expansion

Location: WC-5 Depth: 97.0'



Load No.= 10

Load= 0.25 tsf

D_0 = 0.0237

D_{90} = 0.0235

D_{100} = 0.0235

T_{90} = 0.25 min.

C_V @ T_{90}

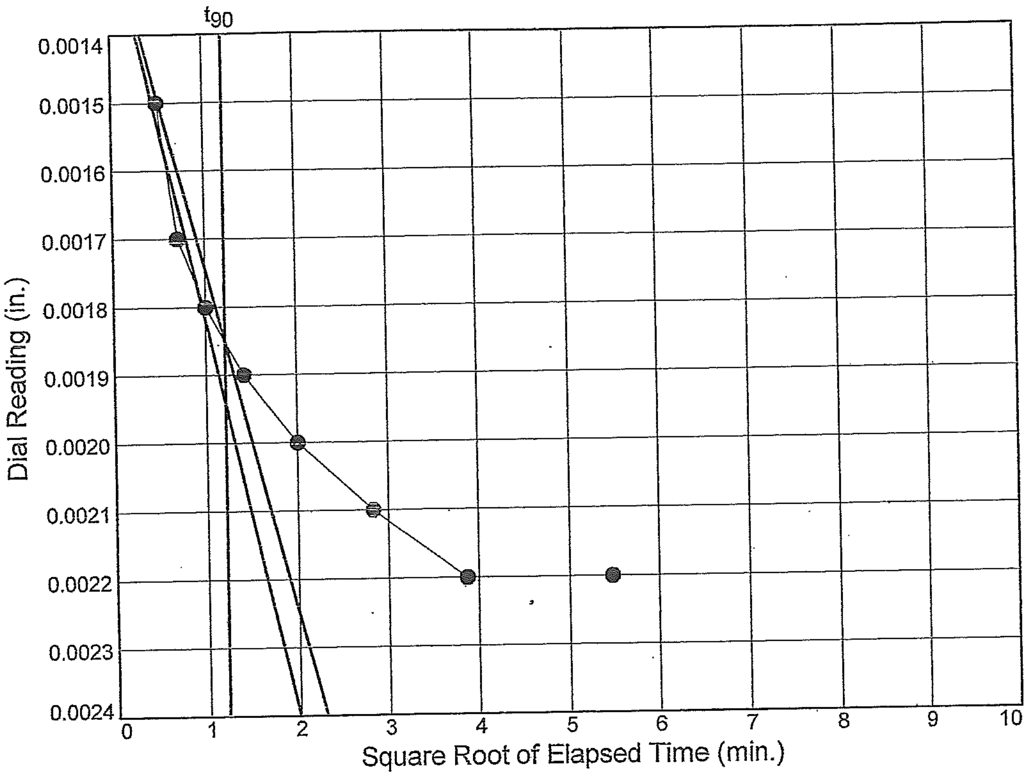
8.157 ft.^{2}/day

Figure

Dial Reading vs. Time

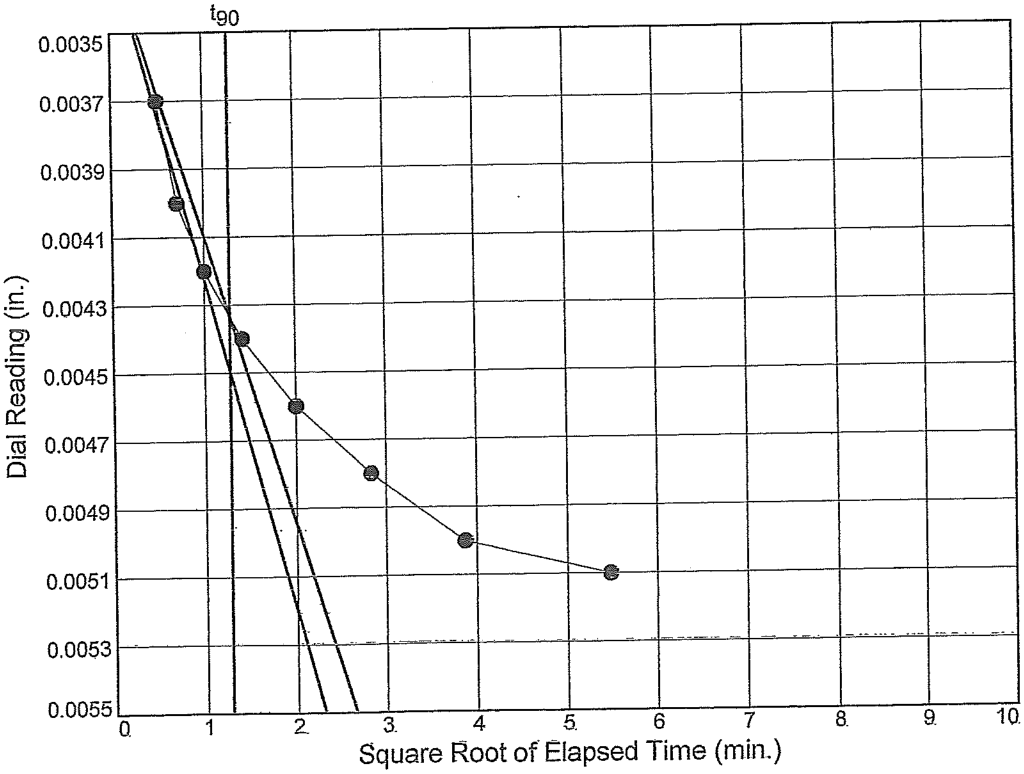
Project No.: 0771-365-11-07-24
Project: Hardin County Expansion

Location: WC-5 Depth: 97.0'



Load No.= 2
Load= 0.50 tsf
D_0 = 0.0012
D_{90} = 0.0019
D_{100} = 0.0019
T_{90} = 1.46 min.

C_V @ T_{90}
1.447 ft.^{2}/day



Load No.= 3
Load= 1.00 tsf
D_0 = 0.0032
D_{90} = 0.0043
D_{100} = 0.0045
T_{90} = 1.63 min.

C_V @ T_{90}
1.291 ft.^{2}/day

Figure

M L Testing, LLC

CONSOLIDATION TEST DATA SHEET

Project : Hardin County Expansion
Project No : 0771-365-11-07-24
Location :
Date : 1/12/17
Material : Sa Cl Gr
Technician : MLT
Boring No : WC-5
Sample Depth : 97'

SPECIFIC GRAVITY OF SOILS = SG = ______ ASSUMED ______ TEST RESULTS ______
HEIGHT OF SPECIMEN, cm 2.54
RING WEIGHT, gm 539.0

	BEFORE	AFTER
WET WEIGHT OF SAMPLE + RING + TARE, g	707.7	~~277.6~~ 816.6
DRY WEIGHT OF SAMPLE + RING + TARE, g		~~251.2~~ 790.2
TARE WEIGHT, g		106.8

INITIAL SOIL PARAMETERS

PERCENT SATURATION ______ VOID RATIO ______ POROSITY ______

WATER CONTENT ______ DRY DENSITY ______

CHANGE IN SAMPLE HEIGHT START OF TEST TO END TEST = ΔH = ______ cm

FINAL SOIL PARAMETERS

PERCENT SATURATION ______ VOID RATIO ______ POROSITY ______

WATER CONTENT ______ DRY DENSITY ______

LOAD		DIAL READING		TIME FOR 50% CONSOLIDATION	Δe	VOID RATIO	Cv
No.	tsf	U=0%	U=100%				
1	0.0625						
2	0.125						
3	0.25						
4	0.5						
5	1						
6	2						
7	4						
8	8						
9	12						
10	20						
REBOUND							
9	12						
8	8						
7	4						
6	2						
5	1						
4	0.5						
3	0.25						
2	0.125						
1	0.0625						

CONSOLIDATION DATA SHEET

LOADING TEST DATA

PROJECT: Hardin County Expansion

LOCATION:

MATERIAL:

BORING NO.: WC-5

JOB NO.: 0771-365-11-07-24

DATE:

TECHNICIAN:

DEPTH: 97'

LOAD 1/4 ton

DATE APPLIED: 1/12/17

Clock Time and Date	Elapsed Time, min	Original
8:20	0	
	0.1	
	0.25	7
	0.5	8
	1	8
	2	8
	4	
	8	
	15	
	30	
	60	
	120	
	240	
	480	
	960	
	1440	

LOAD 1/2 ton load

DATE APPLIED: 1/12/17

Clock Time and Date	Elapsed Time, min	Original
8:24	0	
	0.1	
	0.25	23
	0.5	25
	1	26
	2	27
	4	28
	8	29
	15	30
	30	30
	60	
	120	
	240	
	480	
	960	
	1440	

Dial Readings x ______

CONSOLIDATION DATA SHEET

LOADING TEST DATA

PROJECT: Hardin County Expansion

LOCATION:

MATERIAL:

BORING NO.: WC-5

JOB NO.: 0771-365-11-07-24

DATE:

TECHNICIAN:

DEPTH: 97'

LOAD: 1 ton load

DATE APPLIED: 1/12/17

Clock Time and Date	Elapsed Time, min	Original
8:57	0	
	0.1	
	0.25	53
	0.5	56
	1	58
	2	60
	4	62
	8	64
	15	66
	30	67
	60	
	120	
	240	
	480	
	960	
	1440	

LOAD: 2 ton load

DATE APPLIED: 1/12/17

Clock Time and Date	Elapsed Time, min	Original
9:33	0	
	0.1	
	0.25	108
	0.5	110
	1	114
	2	116
	4	120
	8	122
	15	126
	30	128
	60	133
	120	136
	240	
	480	
	960	
	1440	

Dial Readings x ______

CONSOLIDATION DATA SHEET

LOADING TEST DATA

PROJECT: Hardin County Expansion

LOCATION:

MATERIAL:

BORING NO.: WC-5

JOB NO.: 0771-365-11-07-24

DATE:

TECHNICIAN:

DEPTH: 97'

LOAD 4 ton load

DATE APPLIED: 11/12/17

Clock Time and Date	Elapsed Time, min	Original
14:03	0	
	0.1	
	0.25	174
	0.5	178
	1	181
	2	184
	4	190
	8	195
	15	202
	30	210
	60	219
	120	227
	240	
	480	
	960	236
	1440	

LOAD 8 ton load

DATE APPLIED: 11/13/17

Clock Time and Date	Elapsed Time, min	Original
7:06am	0	
	0.1	
	0.25	281
	0.5	285
	1	289
	2	295
	4	302
	8	308
	15	321
	30	333
	60	340
	120	
	240	376
	480	~~376~~
	960	
	1440	384

Dial Readings x ______

CONSOLIDATION DATA SHEET

LOADING TEST DATA

PROJECT: Hardin County Expansion

JOB NO.: 0771-365-11-07-24

LOCATION:

DATE:

MATERIAL:

TECHNICIAN:

BORING NO.: WC-5

DEPTH: 97'

LOAD 4 ton, rebound

DATE APPLIED: 1/14/17

Clock Time and Date	Elapsed Time, min	Original
8:55am	0	
	0.1	
	0.25	375
	0.5	374
	1	372
	2	370
	4	368
	8	365
	15	362
	30	359
	60	356
	120	352
	240	350
	480	
	960	
	1440	

LOAD 2 ton rebound

DATE APPLIED: 1/14/17

Clock Time and Date	Elapsed Time, min	Original
13:27pm	0	
	0.1	
	0.25	340
	0.5	339
	1	337
	2	336
	4	333
	8	330
	15	325
	30	
	60	
	120	
	240	320
	480	
	960	299
	1440	

Dial Readings x ______

CONSOLIDATION DATA SHEET

LOADING TEST DATA

PROJECT: Hardin County Expansion

JOB NO.: 0771-365-11-07-24

LOCATION:

DATE:

MATERIAL:

TECHNICIAN:

BORING NO.: WC-5

DEPTH: 97'

LOAD 1 ton rebound

DATE APPLIED: 1/15/17

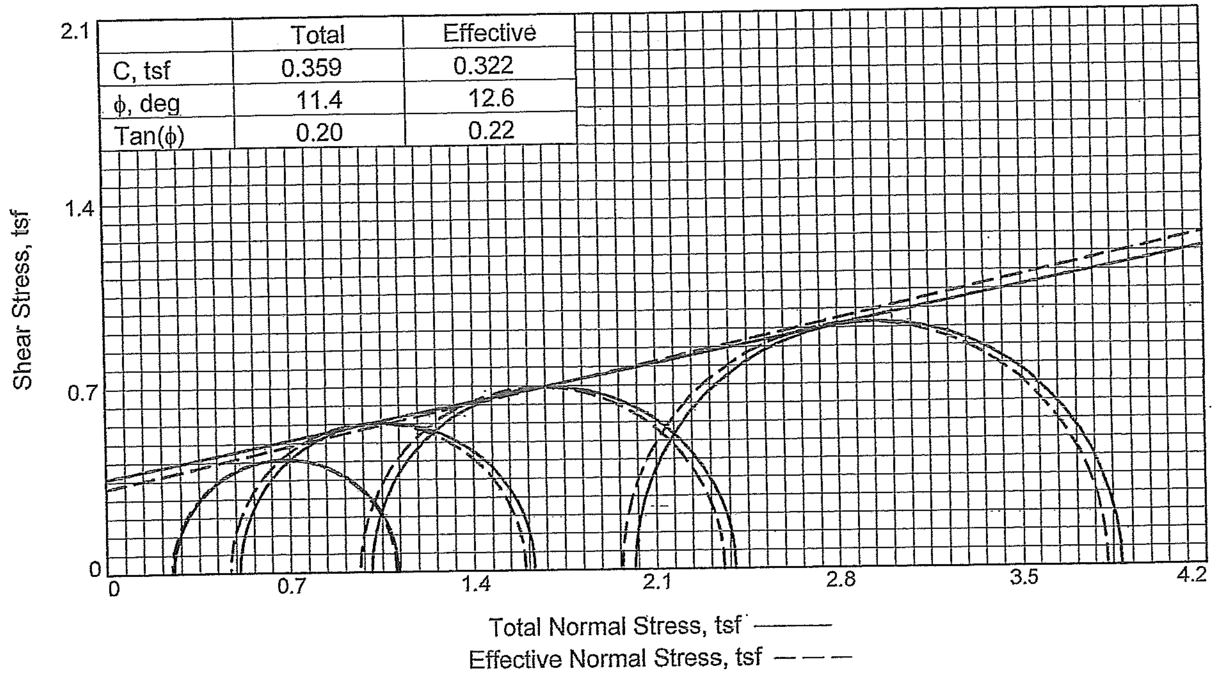
Clock Time and Date	Elapsed Time, min	Original
7:51am	0	
	0.1	
	0.25	293
	0.5	292
	1	291
	2	289
	4	287
	8	285
	15	282
	30	
	60	277
	120	
	240	260
	480	255
	960	
	1440	

LOAD 1/4 ton rebound

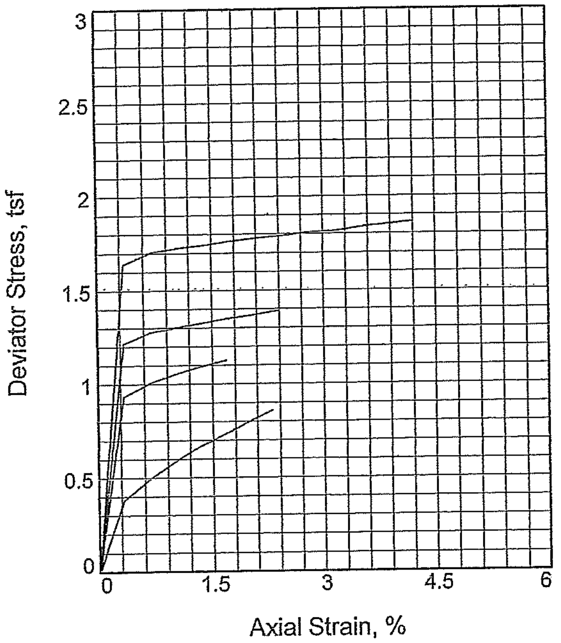
DATE APPLIED: 1/15/17

Clock Time and Date	Elapsed Time, min	Original
16:26pm	0	
	0.1	
	0.25	245
	0.5	244
	1	242
	2	240
	4	237
	8	234
	15	227
	30	
	60	
	120	
	~~240~~ 720	173
	480	
	960	
	1440	164

Dial Readings x ______



	Total	Effective
C, tsf	0.359	0.322
ϕ, deg	11.4	12.6
Tan(ϕ)	0.20	0.22



	Sample No.	1	2	3	4
Initial	Water Content, %	19.6	19.6	19.6	19.6
	Dry Density, pcf	108.7	108.7	108.7	108.7
	Saturation, %	99.7	99.7	99.7	99.7
	Void Ratio	0.5216	0.5216	0.5216	0.5216
	Diameter, in.	1.40	1.40	1.40	1.40
	Height, in.	3.05	3.05	3.05	3.05
At Test	Water Content, %	19.7	19.7	19.7	19.7
	Dry Density, pcf	108.7	108.7	108.7	108.7
	Saturation, %	100.0	100.0	100.0	100.0
	Void Ratio	0.5216	0.5216	0.5216	0.5216
	Diameter, in.	1.40	1.42	1.43	1.45
	Height, in.	3.05	2.98	2.93	2.86
	Strain rate, in./min.				
	Back Pressure, psi	10.00	10.00	10.00	10.00
	Cell Pressure, psi	13.50	17.00	24.00	38.00
	Fail. Stress, tsf	0.86	1.13	1.39	1.86
	Total Pore Pr., tsf	0.73	0.76	0.76	0.77
	Ult. Stress, tsf				
	Total Pore Pr., tsf				
	$\bar{\sigma}_1$ Failure, tsf	1.10	1.60	2.36	3.83
	$\bar{\sigma}_3$ Failure, tsf	0.24	0.47	0.96	1.97

Type of Test:
CU with Pore Pressures

Sample Type: Undisturbed

Description: Clay, silty brown-red sandy

Assumed Specific Gravity= 2.65

Remarks:

Figure ______

Client:

Project: Hardin County Expansion

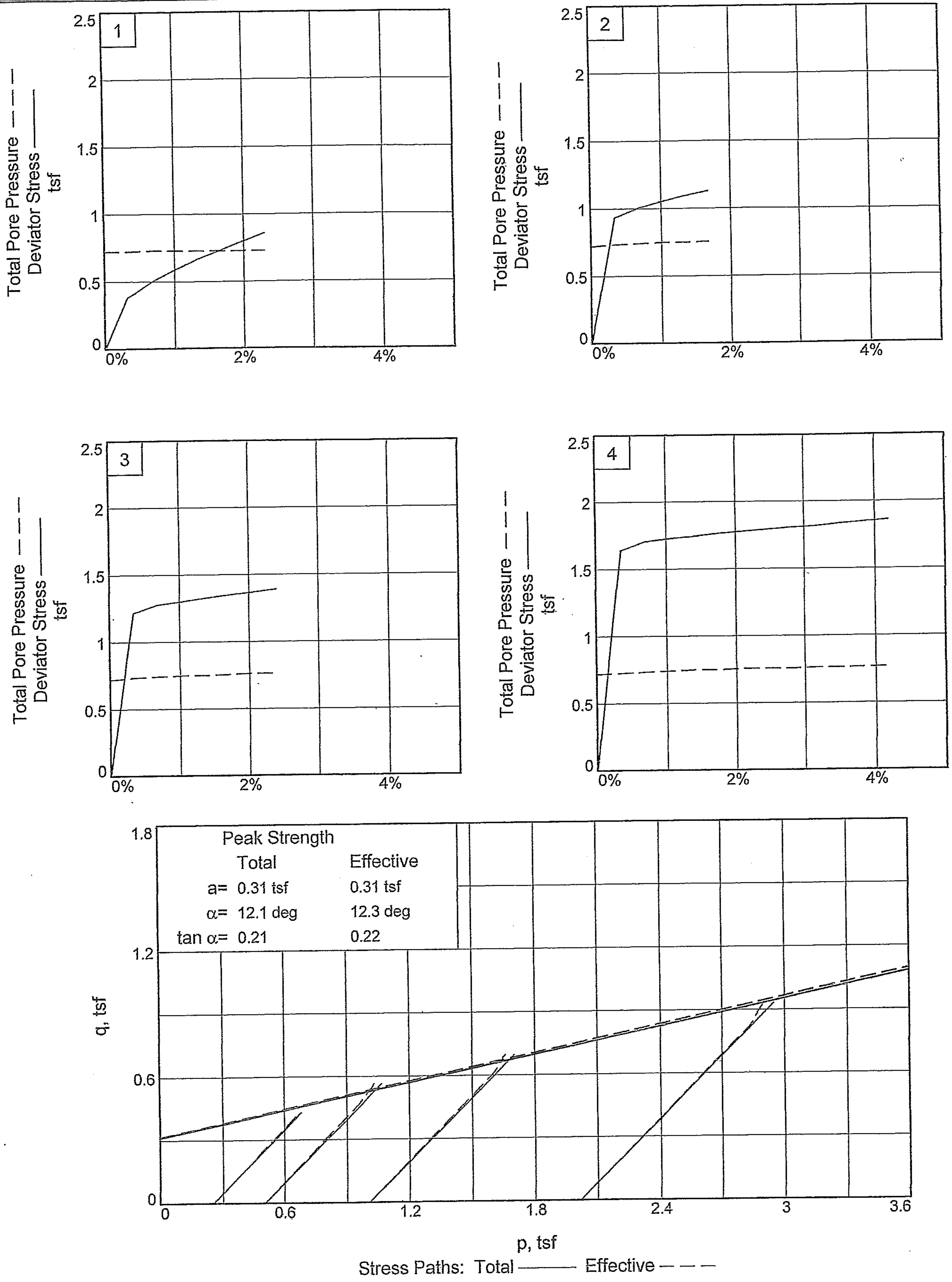
Location: WC-4

Depth: 16.0'

Proj. No.: 0771-365-11-07-24

Date Sampled: 1/30/2017

TRIAXIAL SHEAR TEST REPORT
M L Testing, LLC
Bluff Dale, TX



Client:

Project: Hardin County Expansion

Location: WC-4 **Depth:** 16.0'

Project No.: 0771-365-11-07-24

Figure ______

M L Testing, LLC

TRIAXIAL SHEAR TEST WORKSHEET

Project: Hardin County & Expansion　Job No.: 0771-365-11-07-24

Location: ______　Date: 1/30/17

Material: Si Cly [illegible] Sandy　Tech.: MLT

Boring No.: WC-4　Sample Depth: 16'

TYPE OF TEST: UU ______ CU R̄ ______ CD ______

Multi-Stage ______ Single Stage ______

254.1 / 227.6 / 93.6 C-5

Wt = 160.3		SAMPLE NUMBER		
		①	2	3
INTIAL	Sample Diameter, In.	1.4		
	Sample Height, In.	3.05		
	Water Content, %			
	Dry Density, pcf			
	Saturation, %			
	Void Ratio			
FINAL	Water Content, %			
	Dry Density, pcf			
	Saturation, %			
	Void Ratio			

Coefficient of Consolidation: ______　Volume Change: ______

Rate of Strain: ______

BETA COEFFICIENTS			
DATE	BETA	CONFINING PRESSURE	BACK PRESSURE

SKETCH OF FAILURE

VOLUME CHANGE

Intial Reading: ______ cc

Final Reading: ______ cc

TYPE OF FAILURE

Bulge: ☐

Vertical Split: ☐

Angular: ☐

Slickensided: ☐

TRIAXIAL SHEAR TEST WORKSHEET

Project: Hardin County Expansion
Job No.: 0771-365-11-07-24
Location:
Date: 1/30/17
Material:
Tech.: MLT
Boring No.: WC-4
Sample Depth: 16'
TYPE OF TEST: UU ______ CU $\overline{R}$ CD ______
Multi-Stage ______ Single Stage ______

		SAMPLE NUMBER		
		1	[✓ 2]	3
INTIAL	Sample Diameter, in.			
	Sample Height, in.			
	Water Content, %			
	Dry Density, pcf			
	Saturation, %			
	Void Ratio			
FINAL	Water Content, %			
	Dry Density, pcf			
	Saturation, %			
	Void Ratio			

Coeffcient of Consolidation: ______ Volume Change: ______
Rate of Strain: ______

BETA COEFFICIENTS			
DATE	BETA	CONFINING PRESSURE	BACK PRESSURE

SKETCH OF FAILURE

VOLUME CHANGE

Intial Reading: ______ cc
Final Reading: ______ cc

TYPE OF FAILURE

Bulge: ☐
Vertical Split: ☐
Angular: ☐
Slickensided: ☐

TRIAXIAL SHEAR TEST WORKSHEET

Project: Hardin County Expansion Job No.: 0771-365-11-87-24

Location: Date: 1/30/77

Material: Tech.: MT

Boring No.: WC-4 Sample Depth: 16'

TYPE OF TEST: UU CU R CD

Multi-Stage Single Stage

		SAMPLE NUMBER		
		1	2	3
INTIAL	Sample Diameter, in.			
	Sample Height, in.			
	Water Content, %			
	Dry Density, pcf			
	Saturation, %			
	Void Ratio			
FINAL	Water Content, %			
	Dry Density, pcf			
	Saturation, %			
	Void Ratio			

Coeffcient of Consolidation: Volume Change:

Rate of Strain:

BETA COEFFICIENTS			
DATE	BETA	CONFINING PRESSURE	BACK PRESSURE

SKETCH OF FAILURE

VOLUME CHANGE

Intial Reading: cc

Final Reading: cc

TYPE OF FAILURE

Bulge: ☐

Vertical Split: ☐

Angular: ☐

Slickensided: ☐

TRIAXIAL SHEAR TEST WORKSHEET

Project: Hardin County Expansion **Job No.:** 0771-365-11-07-24

Location: **Date:** 1/30/17

Material: **Tech.:** MLT

Boring No.: WSC-4 **Sample Depth:** 16'

TYPE OF TEST: UU ____ CU $\overline{R}$ CD ____

Multi-Stage ____ Single Stage ____

		SAMPLE NUMBER		
		1	2	3 (4)
INTIAL	Sample Diameter, in.			
	Sample Height, in.			
	Water Content, %			
	Dry Density, pcf			
	Saturation, %			
	Void Ratio			
FINAL	Water Content, %			
	Dry Density, pcf			
	Saturation, %			
	Void Ratio			

Coeffcient of Consolidation: ____ Volume Change: ____

Rate of Strain: ____

BETA COEFFICIENTS			
DATE	BETA	CONFINING PRESSURE	BACK PRESSURE

SKETCH OF FAILURE

VOLUME CHANGE

Intial Reading: ____ cc

Final Reading: ____ cc

TYPE OF FAILURE

Bulge: ☐

Vertical Split: ☐

Angular: ☐

Slickensided: ☐

HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS
TCEQ PERMIT NO. MSW-2214B

MAJOR PERMIT AMENDMENT APPLICATION

PART III – SITE DEVELOPMENT PLAN
APPENDIX IIIF
SURFACE WATER DRAINAGE PLAN

Prepared for

BFI Waste Systems of North America, LLC

March 2017
Revised August 2017

Revised December 2017

NEVZAT TURAN
84059

12-5-2017

Prepared by

Weaver Consultants Group, LLC
TBPE Registration No. F-3727
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Fort Worth, Texas 76109
817-735-9770

WBC Project No. 0120-758-11-02

This document is intended for permitting purposes only.

CONTENTS

DRAWINGS

NEVZAT TURAN
84059

12. 5 - 2017

CONTENTS (Continued)

STATE OF TEXAS
NEVZAT TURAN
84059
LICENSED PROFESSIONAL ENGINEER

12-5-2017

1 INTRODUCTION

1.1 Purpose

This appendix addresses §330.63(c)

The Surface Water Drainage Plan is prepared as part of a permit amendment application for the Hardin County Landfill consistent with Title 30 Texas Administrative Code (TAC) Chapter 330. This plan addresses surface water drainage design and erosion control. Permit level plans and details are presented for the proposed drainage system in this appendix. Appendix IIIF also includes a demonstration consistent with Title 30 TAC §330.305(a) confirming that the proposed landfill development will not adversely alter permitted drainage patterns.

This appendix includes the design of the final cover erosion control structures (i.e., chute and swale system), perimeter drainage channels, detention ponds, as well as hydrologic calculations. Consistent with Title 30 TAC §330.63(c) and §330.305(b) and (c), these facilities are designed to convey run-off produced from the 25-year storm event. In addition, an Erosion Control Plan for all phases of landfill development is included in Appendix IIIF-F. All drainage facilities will be constructed and maintained in accordance with this plan.

1.2 Drainage Demonstration

Section 4 of this appendix includes a demonstration that shows that the proposed landfill development will not adversely alter the existing permitted drainage patterns. As noted in Section 4, the proposed condition represents the proposed configuration of the site after the proposed vertical expansion of the landfill has been developed. Consistent with Title 30 TAC §330.63(c)(1)(C), §330.63(c)(1)(D)(iii), and §330.305(a), the proposed condition is compared to the existing permitted condition to demonstrate that the proposed expansion will not adversely alter the existing permitted drainage patterns.

1.3 Floodplain

The FEMA Flood Insurance Rate Map for the area of the landfill is provided on Figure I/II-11.1. As shown, no floodplains or floodways are exist within the permit boundary.

2 STORMWATER MANAGEMENT

2.1 Drainage System Layout

Stormwater runoff collected in swales located on the top dome and sideslopes of the landfill will be conveyed to drainage letdown structures (chutes) down the slopes to the perimeter channel system. The perimeter channels will be constructed before fill is placed above existing grade in each adjacent landfill sector. The perimeter drainage system will be constructed as the site is developed. Additional details regarding the existing condition of the perimeter drainage system and the sequence of development for the drainage system is listed below.

- Current site drains toward the south through perimeter channels on the east and west side of the fill area as permitted.
- Stormwater draining toward south either drains onto the non-landfill areas on the south of the limits of waste or into the depression areas within the future landfill footprint.
- Future landfill footprint areas eventually drain onto the non-landfill areas south side of the permit boundary.
- Consistent with the natural drainage patterns, the currently developed areas drain toward the southeast portion of the permit boundary as permitted.
- The final stage in the perimeter drainage system construction is shown on the landfill completion plan on Drawings I/IIA.3 and IIIF.1. A detailed drawing of the perimeter channels located along the permit boundary is provided on Drawings IIIF.1 through IIIF-7.

As shown on Drawing IIIF.1 – Drainage Structure Plan, runoff generated from most of the permit boundary will discharge to the Longston Branch of Cypress Creek, which flows in a south to north direction on the east side of the BFI property adjacent to the landfill boundary on the east. Stormwater discharge from the landfill will be attenuated by a detention pond located at the south side of the permit boundary before flowing off the permit boundary at the southeast. The other discharges from the west, north, and northeast of the permit boundary are insignificant and sheet flow at the permit boundary.

The facility has been designed to prevent discharge of pollutants into waters of the state or waters of the United States, as defined by the Texas Water Code and the Federal Clean Water Act, respectively. Hardin County Landfill has a current Texas Pollution Discharge Elimination System (TPDES) multi-sector general permit for industrial activity as stipulated under Section 402 of the Clean Water Act and under Chapter 26 of the Texas Water Code, the TPDES program. A copy of the multi-sector permit is included in Parts I/II, Appendix I/IIE. Any stormwater that has become contaminated by contact with the working face or with leachate will be handled in accordance with Appendix IIIC – Leachate and Contaminated Water Management Plan. The facility maintains a current Stormwater Pollution Prevention Plan prepared consistent with TPDES permit requirements.

2.2 Erosion and Sedimentation Control Plan

The Hardin County Landfill will use various interim and permanent erosion and sedimentation controls throughout the life of the site. The interim controls will be used around active areas and external embankment sideslopes and top dome surfaces. These controls will include temporary letdown structures, soil berms, and vegetation of intermediate cover areas to minimize the erosion potential from these areas. These interim controls will be used during all phases of landfill development to provide effective erosion stability for the external sideslopes and top dome surfaces. Refer to Appendix IIIF-F – Erosion Control Plan for All Phases of Landfill Operation for more information.

Permanent controls include swales and chutes that will be constructed upon completion of the final cover. As part of the final cover construction, an erosion layer capable of sustaining vegetation will be constructed. Areas that receive final cover will be vegetated in accordance with Appendix IIIJ – Closure Plan upon completion of final cover placement. Final cover vegetation will protect the erosion layer soil against erosive runoff velocities. A soil loss and sheet flow velocity demonstration for the erosion layer is included in Appendix IIIF-D. The erosion layer will include a vegetation layer that provides for a 90 percent ground coverage, to keep soil loss below the required design values. If there are areas that do not maintain at least 90 percent vegetative coverage, vegetation in these areas will be reestablished to maintain at least 90 percent vegetative cover.

Erosion will be controlled by vegetation in drainage structures with flow velocities less than or equal to 5 feet per second (fps). For drainage structures with flow velocities greater than 5 fps, rock riprap, gabions, or other surface reinforcing materials as designed will be used for surface reinforcement as depicted on the plans.

During site development, measures such as best management practices (BMPs) and sedimentation ponds will be employed to control erosion and sedimentation. BMPs may include the use of temporary rock riprap, silt fences, straw bales, check dams, interceptor swales and berms, temporary and permanent seeding and sodding, surface roughening, matting and mulching, sediment traps, and surface wetting for dust control (refer to Appendix IIIF-F for more information).

Sedimentation ponds used as erosion control BMPs may consist of (1) existing borrow areas converted to sedimentation ponds, (2) future cell excavation areas, (3) temporary ponds in undeveloped footprint areas, (4) permanent detention pond that will be installed at the south side of the permit boundary, and/or (5) temporary ponds outside the permitted footprint, all of which will be constructed to meet the requirements of the temporary sedimentation pond and located within the permit boundary. See Appendix IIIF-F for more information.

Runoff volume (25-year, 24-hour storm event) from the active fill area (i.e., working face of the landfill operation) will be contained by the containment berm (see Part III, Appendix IIIC – Leachate and Contaminated Water Management Plan for details) to prevent potential discharge of contaminated runoff from the site.

2.3 Stormwater System Maintenance Plan

In accordance with Title 30 TAC §330.305(e)(1), BFI will restore and repair constructed stormwater systems such as channels, drainage swales, and chutes in the event of wash-out or failure from extreme storm events. Stormwater BMPs installed during all phases of landfill development will also be replaced or repaired in the event of failure. Excessive sediment will be removed, as needed, so that the drainage structures (i.e., perimeter channels and detention ponds) function as designed. Site inspections by landfill personnel will be performed weekly or within 24 hours after any significant rainfall event (e.g., a rainfall event with 0.5 inch or more precipitation). Documentation of the inspection will be included in the Site Operating Record.

The following items will be evaluated during the inspections as further discussed in Appendix IIIF-F and Part IV – SOP:

- Erosion of daily and intermediate cover areas, final cover areas, perimeter ditches, chutes, swales, detention ponds, berms, and other drainage features.
- Settlement of intermediate cover areas, final cover areas, perimeter ditches, chutes, swales, and other drainage features.
- Silt and sediment build-up in perimeter ditches, chutes, swales, and detention ponds.
- Removed silt and sediment used as daily cover or to replenish intermediate cover soils.
- Obstructions in drainage features.
- Presence of erosion or sediment discharge at offsite stormwater discharge locations.
- Presence of sediment discharges along the site boundary in areas which have been disturbed by site activities.

- Presence of erosion over the bed and banks of the unnamed tributary of Itasca Creek. If any erosion problems are noted, necessary actions will be implemented to repair damaged locations.

Maintenance activities will be performed to correct damaged or deficient items noted during the site inspections. These activities will be performed as soon as possible after the inspection. The time frame for correction of damaged or deficient items will vary based on weather, ground conditions, and other site-specific conditions that may prevent access to the area requiring repair.

Maintenance activities will consist of the following, as needed:

- Vegetation reestablishment.
- Placement, grading, and stabilization of additional soils in eroded areas or in areas which have settled.
- Replacement or repair of riprap or other surface lining materials.
- Placement of additional riprap in eroded areas.
- Removal of obstructions from drainage features.
- Removal of silt and sediment build-up from drainage features.
- Repairs to erosion and sedimentation controls.
- Installation of additional erosion and sedimentation controls.

3 DRAINAGE SYSTEM DESIGN

3.1 Methodology

Drainage calculations for the final cover system erosion control structures and perimeter drainage system are based on the peak flow rates resulting from the 25-year frequency rainfall event for the area. The United States Army Corps of Engineers (USACE) HEC-HMS computer program was used to compute peak flow rates produced from the design storm for the completion conditions. The hydraulic methods employed in this study are consistent with those presented in the TCEQ Guidelines for Preparing a Surface Water Drainage Report for Municipal Solid Waste Facility (RG-H17, August 2006) and the TxDOT Bridge Division Hydraulic Manual, December 1985.

Water surface profiles were determined for the perimeter channels using the Channel Analysis Program (HYDROCALC HYDRAULICS Version 1.2a for Windows, Dodson & Associates, 1996) that is based on Manning's formula for uniform flow. The perimeter channels are designed to collect and route runoff from the 25-year frequency storm event to the detention ponds. Manning's *"n"* values for the channels and culverts were taken from the TxDOT Bridge Division Hydraulic Manual (Table 1, Page 3-5, and Table 2, Page 4-12), December 1983.

3.2 Hydrologic Analysis

The hydrologic analysis presented in this appendix is based on the currently approved drainage study for the permitted and pre-developed (natural) condition analyses included in Appendix IIIF-E and the proposed conditions drainage analysis included in Appendix IIIF-A. The currently approved analysis utilizes the Rational method for design flow calculations for various channels and the computer program "WSPRO" for pond routing. The proposed condition analysis has been developed using the HEC-HMS computer program to facilitate calculations of flow rates at various channels by considering flow combinations and routing of the estimated total flow through the pond located at the south side of the permit area. The drainage analysis presented in the appendix meets the requirements of §330.305(f)(1), which requires using the rational method for areas less than 200 acres as the flow rates calculated using HEC-HMS are comparable to the flow rates calculated using the rational method as demonstrated in Appendix IIIF-A.

3.2.1 Description of Computer Program

HEC-HMS was developed by the USACE Hydrologic Engineering Center to simulate the surface runoff response of a watershed. The HEC-HMS model represents a watershed as a network of hydrologic and hydraulic components. The modeling process results in the computation of stream-flow hydrographs at desired locations in the watershed. The use of HEC-HMS facilitated calculations of flows in channels, calculation of combined flows at channel junctions, routing through the channels, and total flow into and out of the detention pond using pond routing. The hydrologic analysis for the post-development condition is presented in Appendix IIIF-A. The hydrologic analysis for the permitted landfill condition is included in Appendix IIIF-E.

3.2.2 Watershed Subareas and Schematization

The landfill areas that contribute flow to detention pond were delineated into subareas to derive peak flow rates for the design of the perimeter channel and final cover drainage letdowns. Hydrographs are developed for each subarea and appropriately combined and routed through the swales and perimeter channels. The subareas are shown on Drawing IIIF.2 – Post-Development Drainage Area Plan.

3.2.3 Time Step

The time step, or program computation interval, is the time interval that the flow rates for the hydrographs are generated by the program. Time steps used for a design storm event hydrograph generation are 10 minutes.

3.2.4 Hypothetical Precipitation

The hypothetical precipitation for the storm was obtained from the National Weather Service (NWS) Technical Paper 40 (TP-40) (NWS, 1961) and NOAA Technical Memorandum NW3 Hydro-35 for the project area. For the design storm event analysis, a return period (frequency) of 25 years and a duration of 24 hours is used. The precipitation is assumed to be evenly distributed over the entire landfill area for each time interval.

3.2.5 Precipitation Losses

Precipitation losses (the precipitation that does not contribute to the runoff) are calculated using the Soil Conservation Service (SCS) Curve Number (CN) method. CN is a function of soil cover, land use, and antecedent moisture conditions. A CN of 88 was selected to represent the final covers. A CN of 84 was used for areas within the permit boundary that will not be developed. A CN of 99 was used for the detention pond area. Further discussion on selection of CN values is provided in Appendices IIIF-A.

3.2.6 Hydrograph Information

Three different types of hydrograph generation methods have been used in the drainage analyses: kinematic wave, the Snyder unit hydrograph, and SCS unit hydrograph. Kinematic wave and pond Elevation-Pond Surface Area methods were used for hydrograph routings. Example hydrograph development information for both distributed runoff and Snyder unit hydrograph methods is provided in Appendix IIIF-A.

Kinematic Wave Method

The kinematic wave, a distributed runoff method, is applicable to small-water catchments with uniformly sloped overland flow plains that drain into channels. Landfill final cover areas consist of relatively short (typically 120 feet on 4H:IV sideslopes) overland flow lengths that drain into landfill final cover swales. Distributed runoff estimation methods are applicable to landfill final cover areas because of the following:

- These methods were developed for uniform slopes that drain to collection channels. For a landfill final cover area, this translates to an overland flow segment, which is typically a 4H:IV slope that drains to a swale.
- These methods were developed for a network of relatively small drainage areas each of which can have different physical characteristics (slope, shape, etc.). Typically, to design the perimeter channels, landfill drainage areas need to be subdivided to determine a peak flow at several points to represent uniform physical conditions for each subdivided area.
- These methods are also inherently conservative because they are based on watershed physical characteristics (e.g., dimensions, slopes, and roughness) as opposed to other methods which use empirical information to represent varying physical conditions within the same basin. Also, this method is conservative because flow attenuation is not accounted for.
- This method is also more conservative than the rational method because watershed lag time is computed as a function of real flow time without any limitations such as using a minimum time of concentration (i.e., 10 minutes), which is common practice for the rational method.

The kinematic wave method has been used for estimating peak runoff rates from the landfill final cover areas. A hydrograph from each drainage area with channelized flow (e.g., landfill final cover areas to swales) was developed using the kinematic wave method to simulate both overland and channelized flow. This method utilizes a simplified form of the energy equation and is based on the characteristics of the drainage area, swale, or channel. This method uses physical (measurable) characteristics (e.g., flow lengths, slopes, surface roughness coefficients, channel cross sections) of a watershed to estimate peak discharges.

Snyder Unit Hydrograph Method

The Snyder unit hydrograph method has been used mainly for non-landfill drainage areas (e.g., areas that will not be developed as landfill). The method is applicable to basins with a wide range of characteristics. The Snyder unit hydrograph method has been perhaps the most widely-studied and widely-used unit hydrograph method. Several different methods have been developed to estimate Snyder unit hydrograph parameters (watershed lag and peaking coefficient). Espey 10-Minute method was used in this project to estimate Snyder unit hydrograph parameters. The Espey 10-Minute method was developed using flow records from 41 different watersheds in Texas and other states. The main advantage of the Espey 10-Minute method is that it is one of the best methods for small-size drainage basins.

SCS Unit Hydrograph Method

This method has been utilized only for the pond area to reflect the direct rainfall onto the pond. Soil Conservation Service (SCS), now Natural Resources Conservation Service (NRCS) developed a Dimensionless Unit Hydrograph (DUH) based on the analysis of large number of watersheds. The X-axis consists of dimensionless time units, and the Y-axis consists of dimensionless discharge units. The DUH is very useful for constructing a synthetic unit hydrograph for a wide variety of watersheds.

Hydrograph Routing

The kinematic wave channel routing method was used for routing of the flood wave through the drainage channels. This method is capable of accounting for hydrograph attenuation based on physical channel properties such as length, bottom slope, channel shape, and channel roughness.

Hydrographs at pond outlets were generated by routing the combined incoming flow hydrographs through the ponds. Pond routings were performed by using Elevation – Pond Surface Area relationships for the pond by defining pond surface area versus depth. Additionally, discharge structure (low level outlet and spillway) characteristics of the pond are used for pond routing.

3.3 Hydraulic Analysis

3.3.1 Swale and Channel Analysis

Drainage structure details are illustrated on Drawings IIIF.5 and IIIF.6. The swales and channels are designed to convey the peak flow rate generated by the design storm event. These swales and channels will also reduce maintenance at the site after closure by minimizing erosion. The drainage structures will be constructed as designed unless approval is received from TCEQ for an alternative design via a permit modification.

Hydraulic analyses of the swales and channels are conducted using Manning's uniform flow formula. The uniform flow assumption is applicable to long prismatic channels of uniform slope, as proposed at the site.

The general form of Manning's equation is

$$V = \frac{1.49\, R^{0.667}\, S^{0.5}}{n}$$

in which

V = *Velocity of flow, fps (feet per second)*
n = *Manning's "n" (unitless)*
R = $\frac{A}{P}$ = *Hydraulic radius, ft (feet)*
S = *Friction slope for nonuniform flow or channel slope for uniform flow, ft/ft*
A = *Area of water perpendicular to direction of flow, sf (square feet)*
P = *Wetted perimeter, ft.*

Using the relationship

$$Q = VA$$

Manning's equation can be written as

$$Q = \frac{1.49\, A\, R^{0.667} S^{0.5}}{n}$$

The uniform flow assumption equates the channel slope to the friction slope; therefore, the slope of the channel can be used for "S" in Manning's formula for computation of uniform flow.

Typical values for Manning's "*n*" are presented in the 2009 TXDOT *Bridge Division Hydraulic Manual* ("Suggested Manning's Roughness Coefficients" Table, Chapter 6, Section 1). A value of 0.030 is used for "*n*" for swales; a value of 0.040 is used for gabion-lined chutes; a value of 0.01 is used for FML chutes; and a value of 0.030 is used for perimeter channels. These values represent typical roughness coefficients to the proposed drainage structures, after vegetation has become established.

3.3.2 Letdown Structure (or Chute) Analysis

A typical chute detail is illustrated on Drawings IIIF.5 (gabion-lined chutes). The final cover drainage letdown structures are designed to convey the flow rate generated by the design storm event. Hydraulic analysis of the letdown structures is conducted under the principles of tumbling flow. Tumbling flow is a function of channel slope, discharge, spacing, and sizing of energy dissipating elements. The tumbling flow regime consists of a series of hydraulic jumps and overfalls that maintain critical velocity down the chute. The spacing and sizing of the energy dissipaters control the velocity and flow of the water in the chutes, thereby reducing erosive conditions at slope transitions with the perimeter road low water crossings and chute/perimeter channel confluences.

Appendix IIIF-C presents calculations for the energy dissipaters.

4 DRAINAGE PATTERNS

Consistent with Title 30 TAC §330.63(c)(1)(C), §330.63(c)(1)(D)(iii), and §330.305(a), this section provides a demonstration showing that the proposed changes to final cover grades will not adversely alter the existing permitted drainage patterns. The drawings depicting the two drainage conditions analyzed are listed below.

- Appendix IIIF-A (Post-Development Condition Hydrologic Calculations) – This condition represents the proposed configuration of the site after development of the expanded landfill is complete.
- Appendix IIIF-E (Permitted Condition Hydrologic Calculations) – This condition represents the existing permitted condition and natural drainage analysis developed as part of the currently approved Groundwater and Surface Water Protection and Drainage Plans (Attachment 6).

Supporting calculations are presented in Appendices IIIF-A for post-development conditions and IIIF-E for existing permitted conditions.

The following three sections discuss: (1) regional drainage associated with the site; (2) site drainage patterns; and (3) effect of the proposed development on peak flows, volumes, and velocities discharged from the site.

4.1 Regional Drainage Information

As shown on Figure 4.1, the Hardin County Landfill permit boundary is located near the headwater of Cypress Creek. The permit boundary area drains to Longston Branch located on the east side of the permit boundary. Longston Branch discharges in to Cypress Creek approximately 1 mile north side of the landfill. Cypress Creek discharges in to Village Creek approximately 7 miles east of the landfill (or approximately 3 miles east of Kountze). Village Creek discharges in to Neches River approximately 20 miles south east of Kountze, Texas. Longston Branch or Cypress Creek does not contain any water impoundments downstream of the site.

4.2 Site Drainage Patterns

The pre-development (natural), permitted and proposed site drainage patterns are shown on Figures IIIF-4-1 through 4-4, respectively. As shown on Figures 4-3 and 4-4, the proposed drainage patterns are consistent with the currently permitted drainage patterns. As shown on these two figures, most of the permit area drains toward the south and is discharged from the southeast corner of the permit boundary. The areas draining towards north and west consist only of insignificant areas that sheet flow across the permit boundary. The area draining east, as currently permitted, enters to the adjacent BFI property and continues to flow toward the south/southeast.

4.3 Effect of Site Development on Drainage from the Site

The purpose of this section is to evaluate the areas draining different directions from the permit boundary, peak flow rates, runoff volumes, and peak flow velocities of the pre-development, permitted, and post-development hydrologic conditions. A summary of areas draining in different directions from the permit area and peak flow rates exiting the permit boundary are provided in Table 4.1. Also, the drainage area delineations and the peak flow rates draining in different directions are shown on Figure IIIF-4-2 through IIIF-4-4 for the pre-development, permitted, and post-development conditions, respectively.

The proposed landfill final cover is configured in the same manner as the currently permitted final cover; therefore, runoff from the final cover areas for the post-development condition drains into the perimeter ditch system, similar to the currently permitted conditions. The main difference between the currently permitted and the post-development conditions is that the flow toward the adjacent BFI property on the east is diverted to south where a significantly increased flow attenuation is provided via a detention pond. This change does not affect the permitted conditions or off-site drainage, as (1) the flow entering the adjacent BFI property eventually drains to the south/southeast and (2) the peak flow rate for the post-development conditions is reduced to 54 cfs, which is reported to be 58.31 cfs and 76 cfs for the pre-development and the permitted conditions, respectively. Table 4-1 on page IIIF-16 lists the drainage areas contributing to each of these calculated flows and the footnotes to Table 4-1 provide additional detail.

The volumes and flow velocities for the permitted and post developed conditions are presented in Appendix IIIF-A. The volumes (refer to page IIIF-A-30) were estimated for the permitted conditions using the runoff coefficients utilized in the currently permitted drainage design and the storm event used for the post development analysis (refer to page IIIF-A-1). The apparent increase in the total volume of runoff is due to (1) the currently permitted analysis not encompassing the entire permit boundary and (2) a significantly large detention pond incorporated into the post-development drainage design on the south side of the permit boundary. Regardless of the increase in the total volume of runoff, the proposed attenuation of the runoff by the detention pond facilitates discharge of runoff in a controlled manner so that the post-development peak discharge is significantly lower than the estimated permitted peak discharge. Also, the velocity calculations presented on page IIIF-A-35 demonstrate that runoff velocities at the outfall (and sheet flows) will be non-erosive; therefore, erosion of offsite areas will not occur.

4.4 Summary

From the hydrological evaluations of the pre-developed, permitted, and post-development conditions, the existing drainage conditions at the permit boundary will not be adversely altered by the proposed development. Given that: (1) drainage patterns are not adversely altered, (2) total design stormwater peak discharge rate at the permit boundary is less than the permitted total stormwater peak discharge rate, (3) post-development runoff velocity at the permit boundary will not be erosive (or increase from the currently permitted condition), and (4) the stormwater discharge outfall locations are consistent with the permitted configuration, it is concluded that the proposed landfill development will not adversely alter permitted drainage patterns consistent with Title 30 TAC 330.63(c)(1)(C), §330.63(c)(1)(D)(iii), and §330.305(a).

Table 4-1
Pre-Development Permitted and Proposed Site Drainage Summary

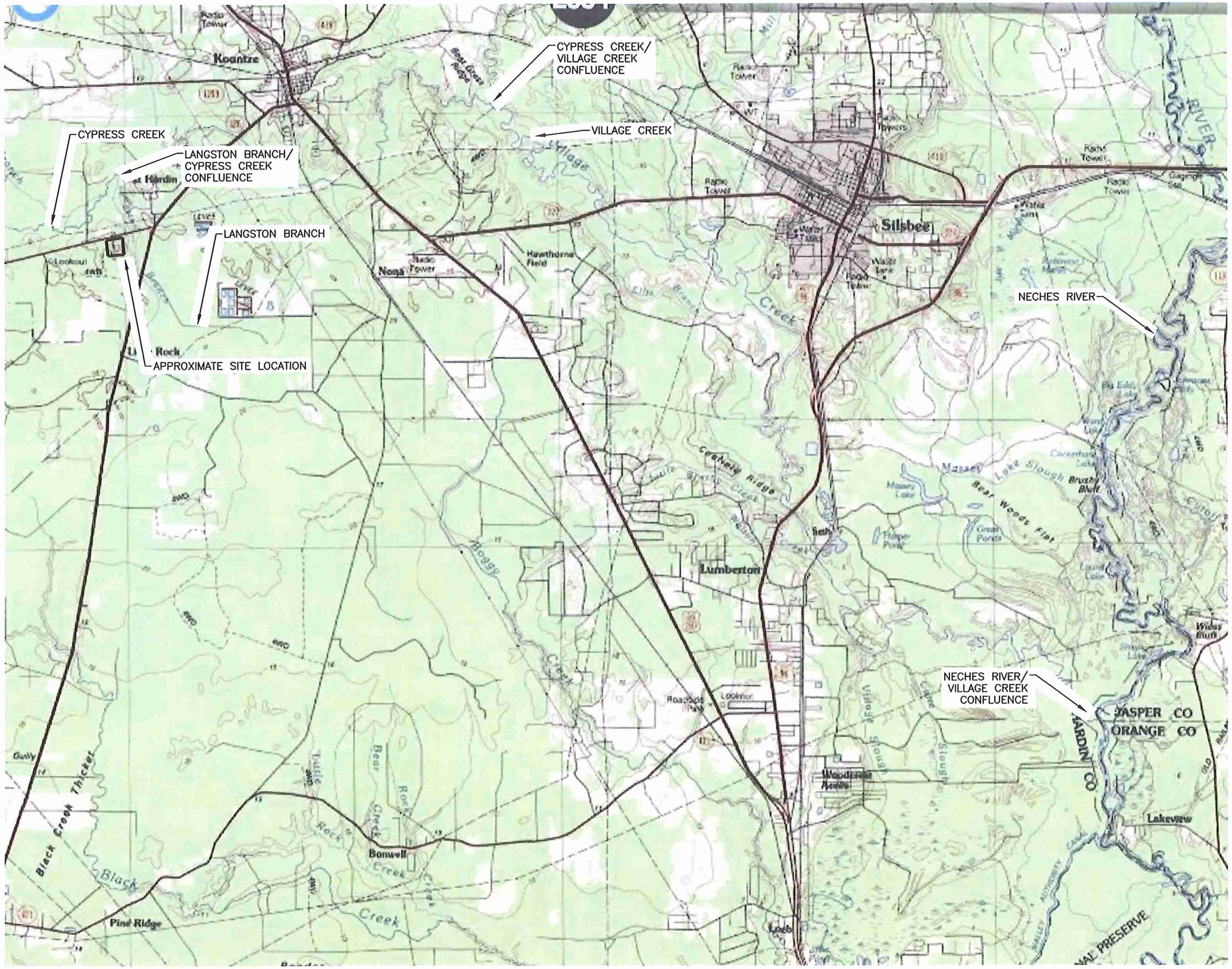
Discharge Direction	Natural (Pre-Development Conditions)			Permitted Conditions			Post-Development Conditions		
	Drainage Basin Label	Drainage Area (ac)	25-Yr Storm Peak Flow (cfs)	Drainage Basin Label	Drainage Area (ac)	25-Yr Storm Peak Flow (cfs)	Drainage Basin Label	Drainage Area (ac)	25-Yr Storm Peak Flow (cfs)
North	Q5	3.75	13.3	E	2.22	19.23	NA1	1.25	6.7
North	Q3	2.42	4.32				NA2	1.04	4.1
East	Q2	25.26	54.68	A	10.34	89.55	EA1	3.78	17.2
South	Q1	45.96	58.31	B, C, D	38.43	76	SD	73.15	54.0
West	Q4	1.03	3.36	F	0.1	0.87	WA1	1.22	5.1

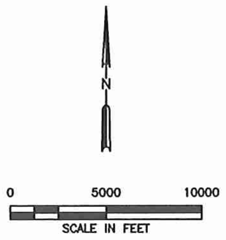
[1] The drainage information presented for the pre-development and the permitted conditions have been reproduced from the currently permitted drainage analysis shown on Figure IIIF-4-2 and IIIF-4-3 respectively.

[2] All currently permitted condition flow rates are reproduced from Figure IIIF-4-3 as reported except the peak flow rate draining to south (areas labeled as B, C, and D in the table has been obtained from the routing information included on page "Part III- Att 6-6" of the currently approved draiange design (refer to Appendix IIIF-E for the currently permitted draiange design information).

[3] Permitted condition flow rate to east (Drainage basin label "A") was estimated using the total area ("Contrib. Areas 2, 3, and 4) draining to east as presented in the currently approved drainage design (refer to Figure IIIF-4-3).

[4] Pre-development and post development total areas represent the permit boundary draiange area (approximatly 80 acres). The permitted condition draiange area represent the landfill area as presented in the approved draiange plan (Appendix IIIF-E). However, the total flow for the permitted condition at the southeast corner represent the areas outside the limits of waste on the south side as discussed in Section 1.2 (page "Part III - Att 6-3") of the permitted draiange plan. Therefore, the flow rates in the table represent the entire permit boundary for each condition.





LEGEND

— — - — — - — BFI EAST PROPERTY BOUNDARY

NOTES:

1. MAP OBTAINED FROM THE USGS.

NEVZAT TURAN
84059

12-5-2017

☐ DRAFT ☒ FOR PERMITTING PURPOSES ONLY ☐ ISSUED FOR CONSTRUCTION	PREPARED FOR BFI WASTE SYSTEMS OF NORTH AMERICA, LLC	REGIONAL DRAINAGE INFORMATION AREA PLAN HARDIN COUNTY LANDFILL HARDIN COUNTY, TEXAS
DATE: 03/2017 FILE: 0120-758-11 CAD: FIG IIIF-4-4-PROP COMP PLAN.DWG	DRAWN BY: SRF DESIGN BY: AE REVIEWED BY: NT	
Weaver Consultants Group TBPE REGISTRATION NO. F-3727		WWW.WCGRP.COM
		FIGURE IIIF-4.1

REVISIONS

NO.	DATE	DESCRIPTION
1	11/2017	OWNERSHIP CHANGE

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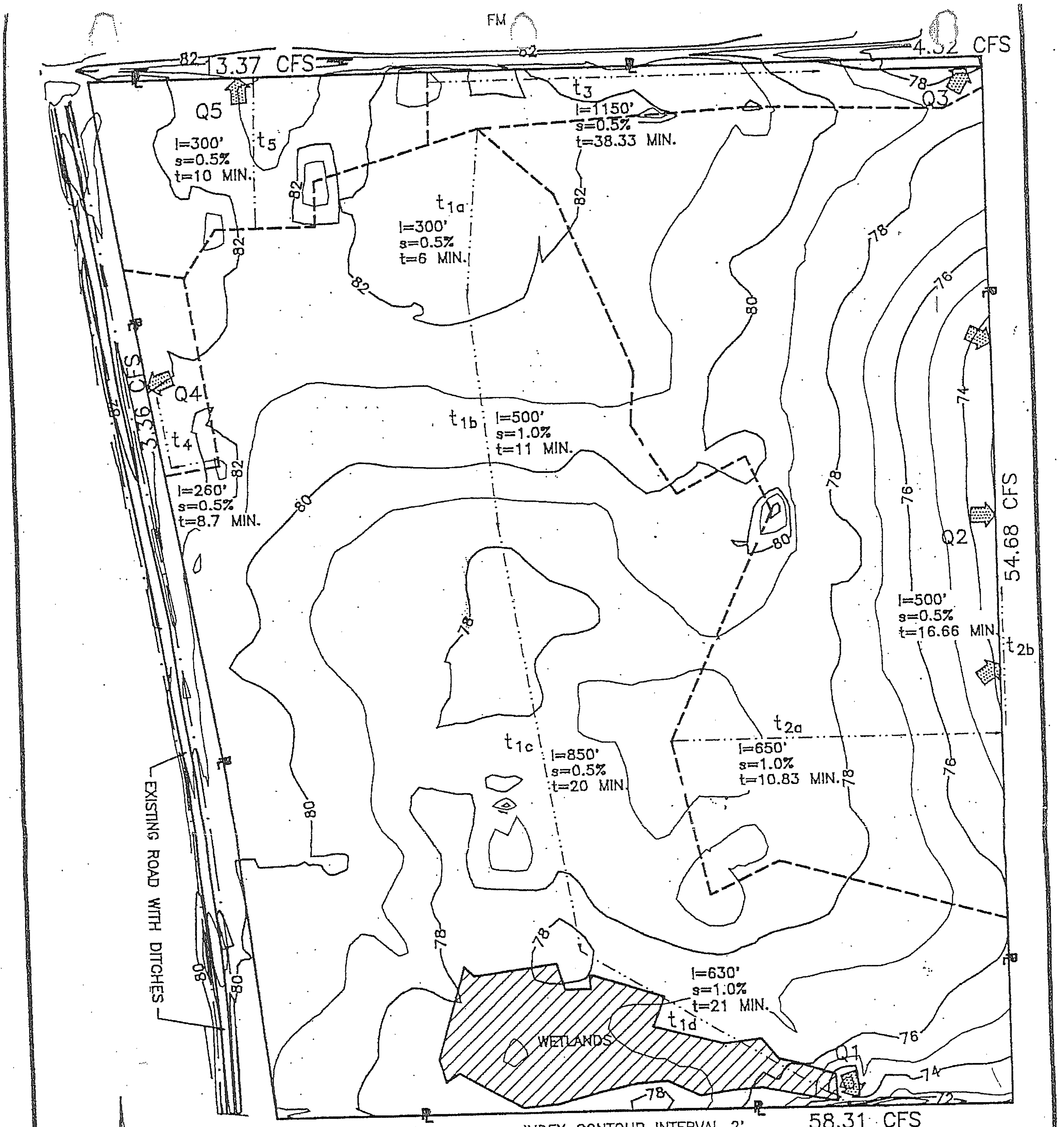

CONTOUR INTERVAL 1'   INDEX CONTOUR INTERVAL 2'

- INDICATES DIRECTION OF RUNOFF
- BOUNDARY OF RUNOFF AREA
- FLOW LINE OF DITCH
- TIME OF CONCENTRATION LINE
- PROPERTY LINE

l= LENGTH
s= SLOPE
t= TIME OF CONCENTRATION

NOTE:
NO PORTION OF SITE WITHIN 100-YEAR FLOODPLAN.

FOR 25 – YEAR STORM

$Q = CIA \quad I = \frac{b}{(t + d)^e}$ HARDIN COUNTY $b = 80 \quad d = 7.5 \quad e = 0.720$

	C	I (in/hour)	A (Ac)	FLOW (CFS)
Q1	0.35	3.625	45.96	58.31
Q2	0.35	6.185	25.26	54.68
Q3	0.35	5.096	2.42	4.32
Q4	0.35	10.19	1.03	3.36
Q5	0.35	10.19	3.75	13.37

N

Figure IIIF-4-2

SCALE IN FEET

200 0 200

SCALE: 1"=200'

FOR PERMITTING PURPOSES ONLY

BILLY L. SIMS
53016

KSA ENGINEERS, INC.
1313 S. JOHN REDDITT DR.–P.O. BOX 151508
LUFKIN, TEXAS 75915–1508 (409) 637–6061
ENGINEERS – SURVEYORS
DALLAS LONGVIEW LUFKIN TYLER

SURVEYED	DESIGNED	DRAWN	APPROVED	JOB NO.	DATE
N.A.	ALSII	DLM	BDS	HN-001	AUG. 1994

HARDIN COUNTY
LANDFILL PERMIT
APPLICATION–PART III
PROJECT NAME

SITE DEVELOPMENT PLAN
ATTACHMENT #6
EXISTING DRAINAGE
SHEET NAME

DRAWING SCALE:
VERT: NONE
HORIZ: 1"=200'
PLOT: AUG. 94
FN: EXDRAIN

SHEET NO.
33
OF

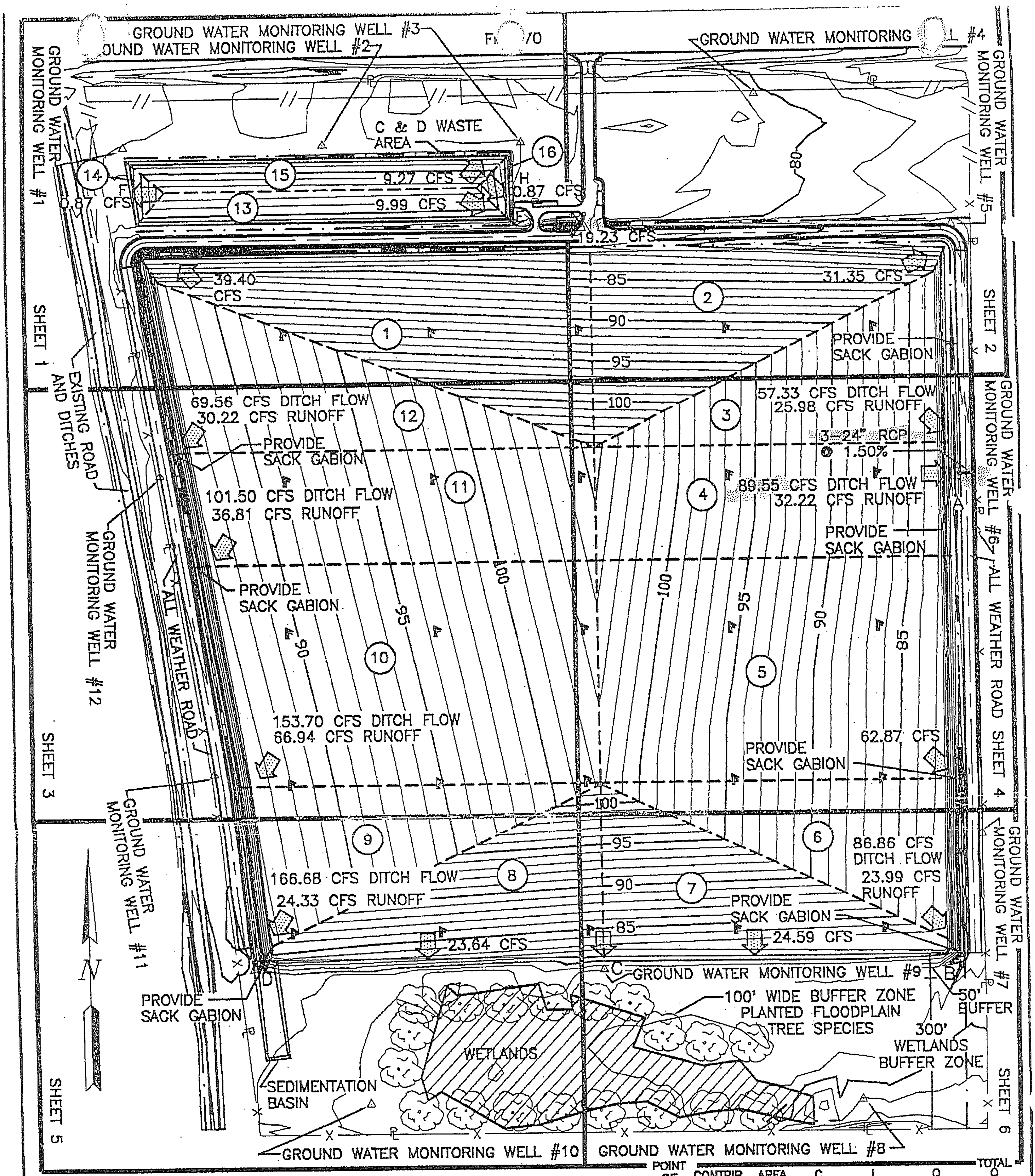

Figure IIIF-4-3

CONTOUR INTERVAL 1'   INDEX CONTOUR INTERVAL 5'

- — — — BOUNDARY OF RUNOFF AREA
- ⇨ INDICATES DIRECTION OF RUNOFF
- (#) RUNOFF AREA (SEE ATTACHMENT #6 & #13)
- —·—·— FLOW LINE OF DITCH (SEE ATTACHMENT #6)
- △ MONITORING WELL (SEE ATTACHMENT #11 & #13)
- —PL— PROPERTY LINE
- —X— BARBED WIRE FENCE(SEE ATTACHMENT #1)
- —//— CHAIN LINK FENCE(SEE ATTACHMENT #1)
- ▲ GAS VENT(SEE ATTACHMENT #14)

$$Q = CIA$$

$$I = \frac{b}{(t + d)^e}$$

HARDIN COUNTY
25-YEAR STORM

$b = 80$
$d = 7.5$
$e = 0.720$

NOTE:
SIDE SLOPES 4:1 HORIZONTAL TO VERTICAL
TOP SLOPES 2% MINIMUM AND 25% MAXIMUM
FOR CROSS SECTIONS SEE ATTACHMENT #2

POINT OF CONC.	CONTRIB. AREA	AREA (Ac)	C	I (In/hr)	Q (CFS)	TOTAL Q (CFS)
	2	3.62	0.85	10.19	31.35	31.35
	3	3.00	0.85	10.19	25.98	57.33
	4	3.72	0.85	10.19	32.22	32.22
A		10.34	0.85	10.19	89.55	89.55
	5	7.26	0.85	10.19	62.87	62.87
	6	2.77	0.85	10.19	23.99	86.86
B		10.03	0.85	10.19	86.86	86.86
	7	2.84	0.85	10.19	24.59	24.59
	8	2.73	0.85	10.19	23.64	23.64
C		5.57	0.85	10.19	48.24	48.24
	1	4.55	0.85	10.19	39.40	39.40
	12	3.49	0.85	10.19	30.22	69.56
	11	4.25	0.85	10.19	36.81	101.50
	10	7.73	0.85	10.19	66.94	153.70
	9	2.81	0.85	10.19	24.33	166.68
D		22.83	0.85	8.59	166.68	166.68
	13	1.05	0.85	10.19	9.09	9.09
	15	1.07	0.85	10.19	9.27	9.27
	16	0.10	0.85	10.19	0.87	10.13
E		2.22	0.85	10.19	19.23	19.23
F	14	0.10	0.85	10.19	0.87	0.87

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BILLY D. SIMS
53016

SCALE IN FEET
200   0   200

SCALE: 1"=200'

REVISED SEPT. 9, 1997
REVISED AUG. 16, 1996

KSA ENGINEERS, INC. 1313 S. JOHN REDDITT DR.–P.O. BOX 151508
LUFKIN, TEXAS 75915-1508 (409) 637-6061
ENGINEERS - SURVEYORS
DALLAS LONGVIEW LUFKIN TYLER

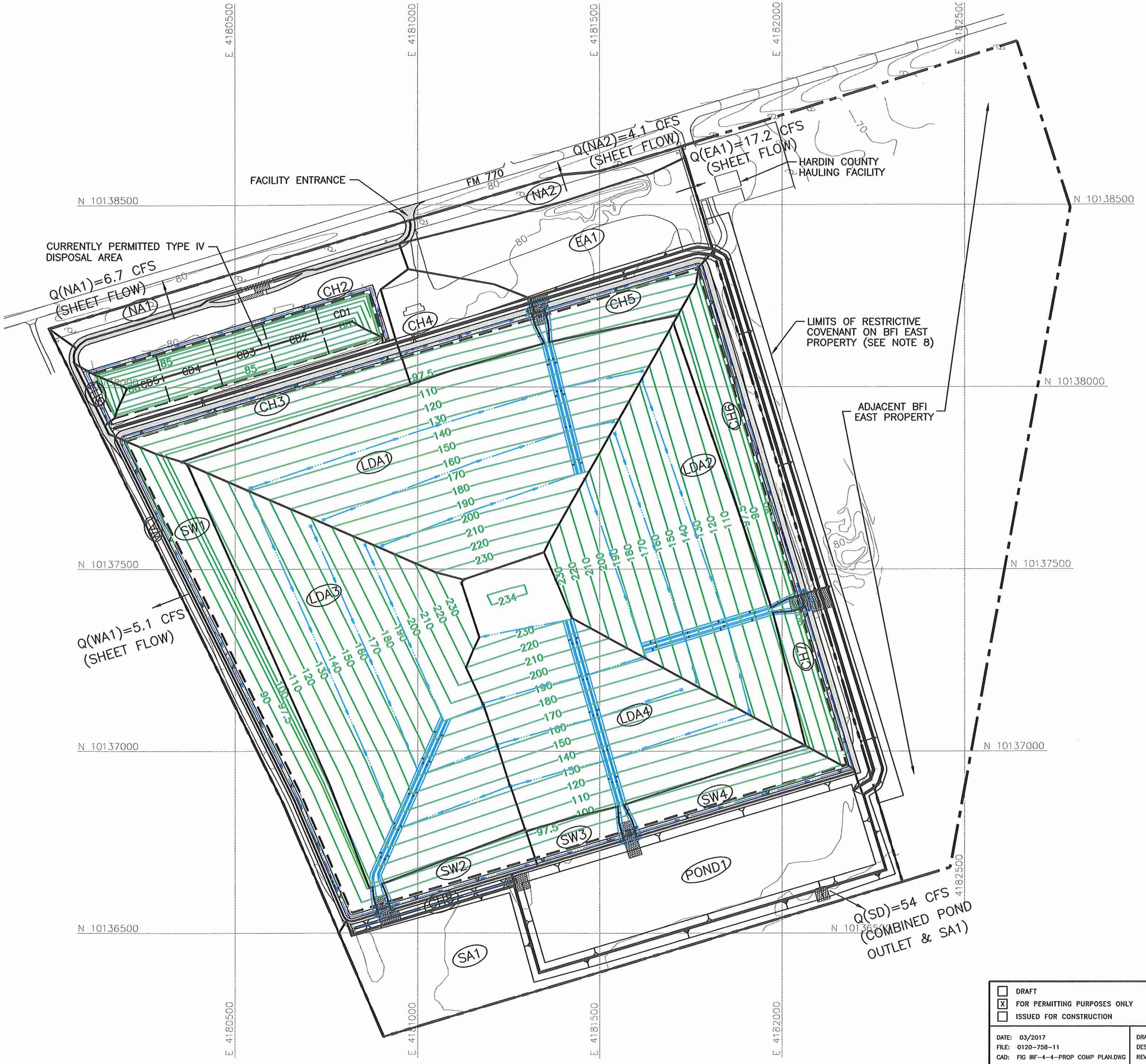
SURVEYED	DESIGNED	DRAWN	APPROVED	JOB NO.	DATE
N.A.	ALSII	DLM	BDS	HN-001	AUG. 1994

HARDIN COUNTY
LANDFILL PERMIT
APPLICATION–PARTIII
PROJECT NAME:

SITE DEVELOPMENT PLAN
ATTACHMENT # 6
FINAL CONTOUR MAP
SHEET NAME:

DRAWING SCALE:
VERT: NONE
HORIZ: 1"=200'
PLOT: SEPT. 97
FN: FINDRAIN

SHEET NO.
34
OF



N

0 150 300
SCALE IN FEET

LEGEND

— · — · —	BFI EAST PROPERTY BOUNDARY
—— · · ——	PERMIT BOUNDARY
— — — —	CURRENTLY PERMITTED LIMITS OF WASTE
—— —	CELL BOUNDARY
N 10137500	STATE PLANE COORDINATE GRID
—70—	EXISTING CONTOUR
—234—	PROPOSED FINAL CONTOUR (SEE NOTE 3)
—·—·—	PROPOSED DRAINAGE SWALE
═══	PROPOSED DRAINAGE CHUTE
———	DRAINAGE AREA BOUNDARY
LDA2	DRAINAGE AREA DESIGNATION

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS DEVELOPED BY WEAVER CONSULTANTS GROUP FROM AERIAL PHOTOGRAPHY FLOWN 05-17-2016. GRID SYSTEM IS TIED TO THE TEXAS STATE PLANE COORDINATE SYSTEM NAD 83, CENTRAL ZONE.
2. UPON APPROVAL OF THE APPLICATION (WITHIN ONE YEAR OR PRIOR TO INSTALLATION OF THE CHANNEL, WHICH EVER COMES FIRST), RESTRICTIVE COVENANT FOR THE DRAINAGE CHANNEL INSIDE THE ADJACENT BFI EAST PROPERTY WILL BE FILED WITH HARDIN COUNTY.

STATE OF TEXAS
NEVZAT TURAN
84059
LICENSED PROFESSIONAL ENGINEER

12-5-2017

O:\0120\758\2214B EXPANSION\IIIF\IIIF-4-4 PROPOSED COMPLETION PLAN (NT-flows).dwg, 11/15/2017 10:52:36 AM, rsellers, 1:2

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FILE: 0120-758-11
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DRAWN BY: SRF
DESIGN BY: AE
REVIEWED BY: NT

PREPARED FOR
BFI WASTE SYSTEMS OF NORTH AMERICA, LLC

REVISIONS

NO.	DATE	DESCRIPTION
1	11/2017	OWNERSHIP CHANGE

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TBPE REGISTRATION NO. F-3727

POST-DEVELOPMENT DRAINAGE AREA PLAN

HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS

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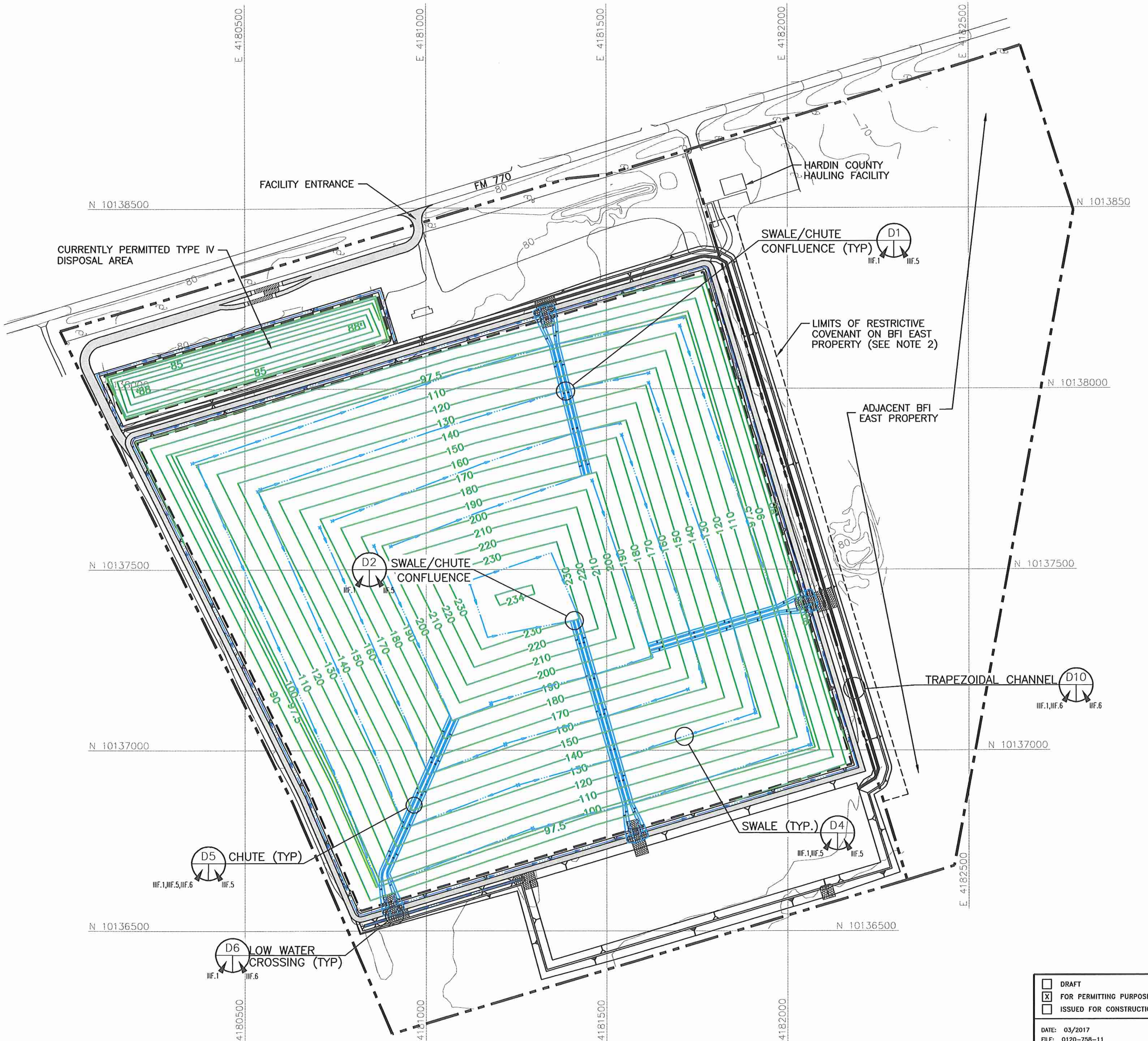
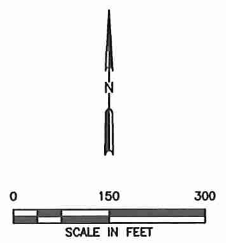
FIGURE IIIF-4-4

DRAWINGS





LEGEND

— · — ·	BFI EAST PROPERTY BOUNDARY
— · · —	PERMIT BOUNDARY
— — — —	CURRENTLY PERMITTED LIMITS OF WASTE
N 10137500	STATE PLANE COORDINATE GRID
70	EXISTING CONTOUR
234	FINAL CONTOUR
— · → —	DRAINAGE SWALE
═	DRAINAGE CHUTE

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS DEVELOPED BY WEAVER CONSULTANTS GROUP FROM AERIAL PHOTOGRAPHY FLOWN 05-17-2016. GRID SYSTEM IS TIED TO THE TEXAS STATE PLANE COORDINATE SYSTEM NAD 83, CENTRAL ZONE.
2. UPON APPROVAL OF THE APPLICATION (WITHIN ONE YEAR OR PRIOR TO INSTALLATION OF THE CHANNEL, WHICH EVER COMES FIRST), RESTRICTIVE COVENANT FOR THE DRAINAGE CHANNEL INSIDE THE ADJACENT BFI EAST PROPERTY WILL BE FILED WITH HARDIN COUNTY.

NEVZAT TURAN
84059

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DRAWN BY: SRF
DESIGN BY: AE
REVIEWED BY: NT

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

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BFI WASTE SYSTEMS OF NORTH AMERICA, LLC

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DRAINAGE STRUCTURE PLAN

HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS

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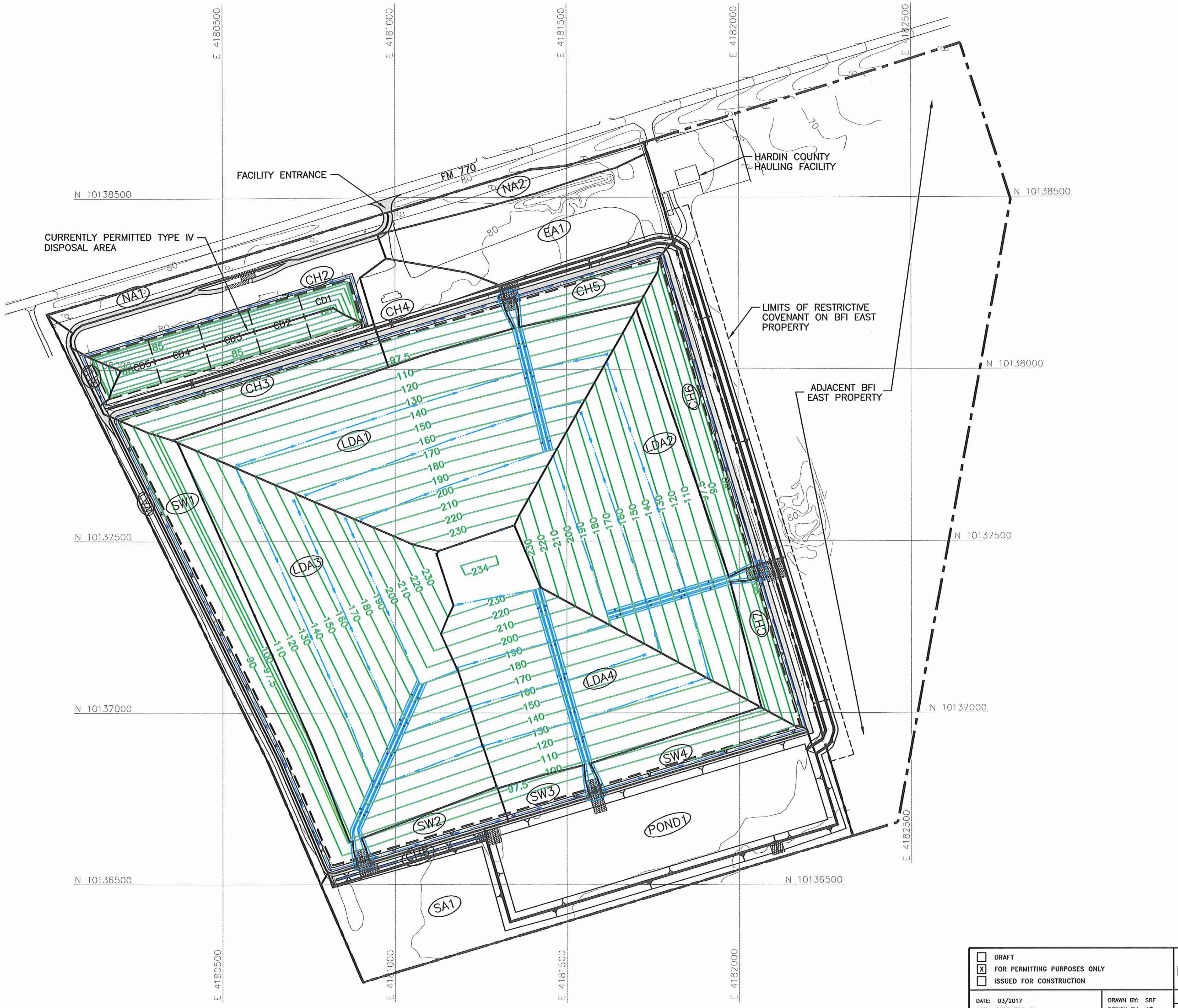
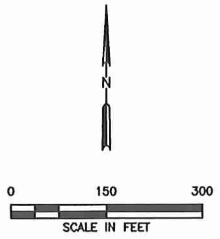
DRAWING IIIF.1





LEGEND

— · — · —	BFI EAST PROPERTY BOUNDARY
—— · · ——	PERMIT BOUNDARY
— — — —	CURRENTLY PERMITTED LIMITS OF WASTE
—— – ——	CELL BOUNDARY
N 10137500	STATE PLANE COORDINATE GRID
70	EXISTING CONTOUR
234	FINAL CONTOUR
→	DRAINAGE SWALE
▨	DRAINAGE CHUTE
——	DRAINAGE AREA BOUNDARY
LDA2	DRAINAGE AREA DESIGNATION

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS DEVELOPED BY WEAVER CONSULTANTS GROUP FROM AERIAL PHOTOGRAPHY FLOWN 05-17-2016. GRID SYSTEM IS TIED TO THE TEXAS STATE PLANE COORDINATE SYSTEM NAD 83, CENTRAL ZONE.

NEVZAT TURAN
84059

12. 5-2017

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DATE: 03/2017
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DRAWN BY: SRF
DESIGN BY: AE
REVIEWED BY: NT

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TBPE REGISTRATION NO. F-3727

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REVISIONS

NO.	DATE	DESCRIPTION
1	11/2017	OWNERSHIP CHANGE

POST-DEVELOPMENT DRAINAGE AREA PLAN

HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS

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DRAWING IIIF.2

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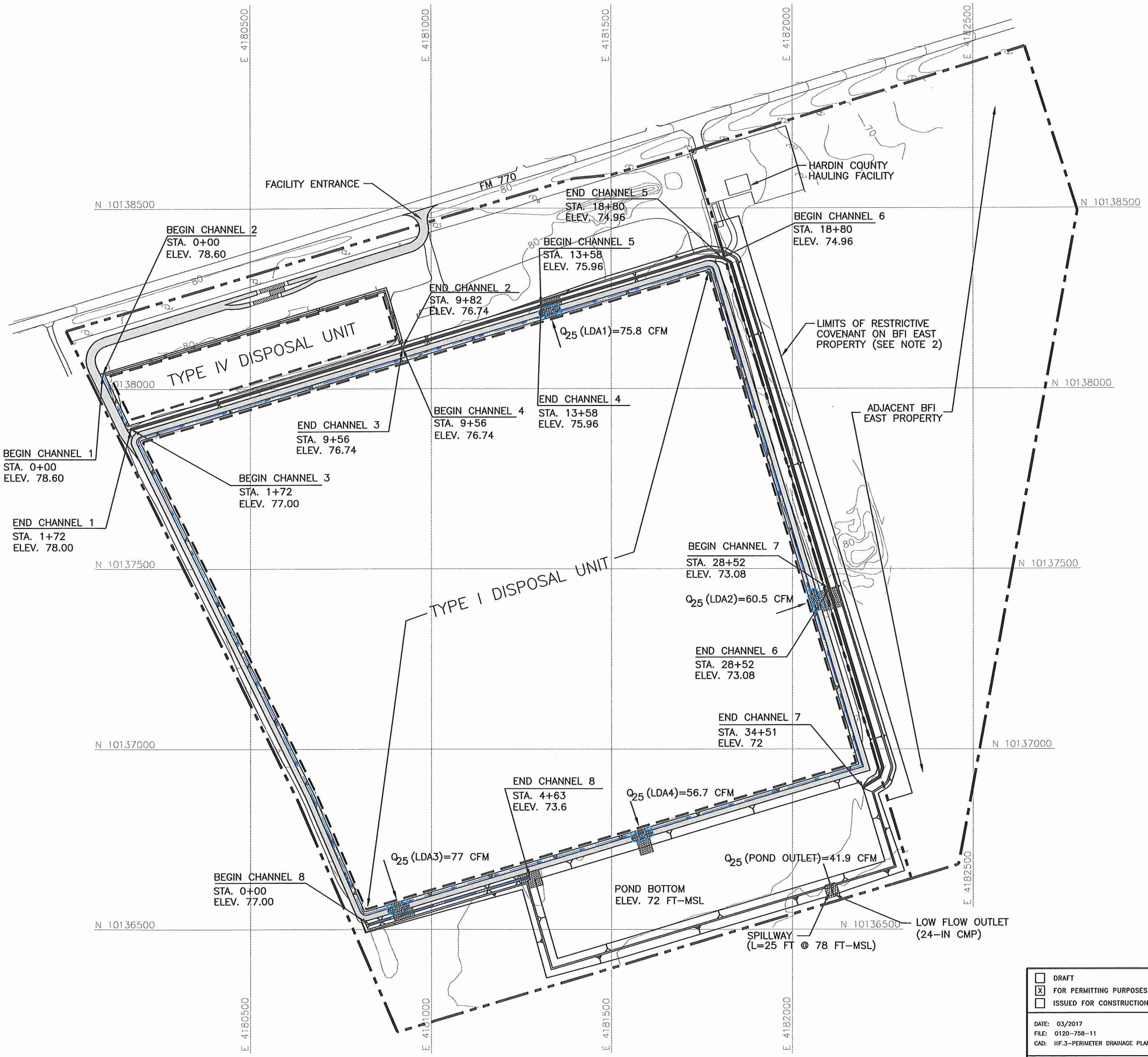

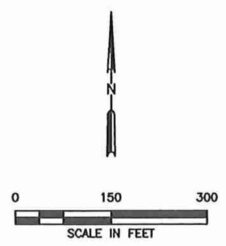

LEGEND

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—— - - ——	PERMIT BOUNDARY
— — — — —	CURRENTLY PERMITTED LIMITS OF WASTE
N 10137500	STATE PLANE COORDINATE GRID
——70——	EXISTING CONTOUR
STA.	CHANNEL STATION (FT)
ELEV.	ELEVATION (FT–MSL)
Q_{25}(LDA2)	25–YR FREQUENCY PEAK FLOW RATE (CUBIC FEET PER SECOND)

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS DEVELOPED BY WEAVER CONSULTANTS GROUP FROM AERIAL PHOTOGRAPHY FLOWN 05–17–2016. GRID SYSTEM IS TIED TO THE TEXAS STATE PLANE COORDINATE SYSTEM NAD 83, CENTRAL ZONE.
2. CHANNEL PROFILE INFORMATION IS PROVIDED ON IIIF–4.

NEVZAT TURAN
84059

12-5-2017

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☒ FOR PERMITTING PURPOSES ONLY
☐ ISSUED FOR CONSTRUCTION

DATE: 03/2017
FILE: 0120–758–11
CAD: IIIF.3–PERIMETER DRAINAGE PLAN.DWG

DRAWN BY: SRF
DESIGN BY: AE
REVIEWED BY: NT

Weaver Consultants Group
TBPE REGISTRATION NO. F–3727

PREPARED FOR
BFI WASTE SYSTEMS OF NORTH AMERICA, LLC

REVISIONS

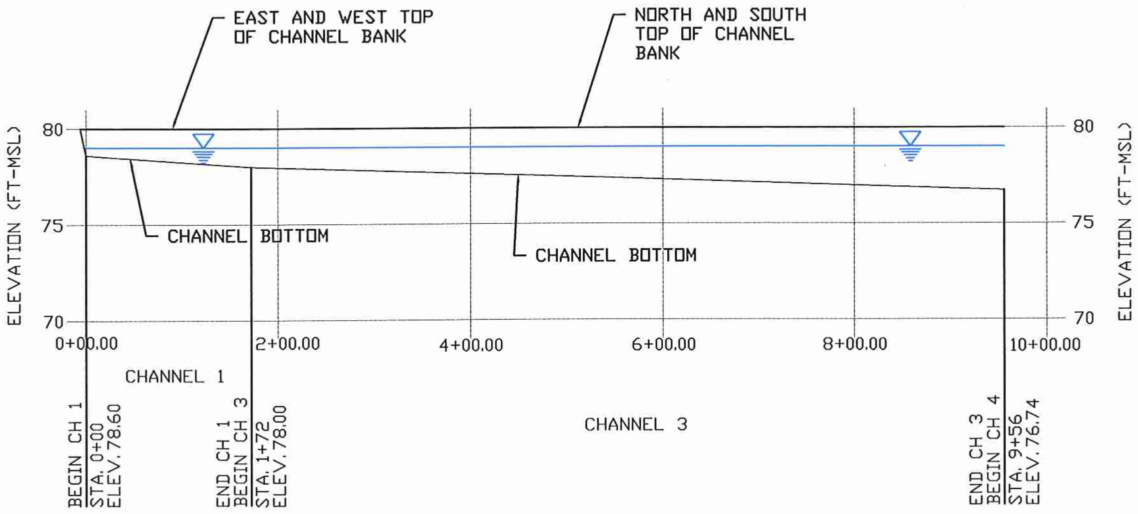
NO.	DATE	DESCRIPTION
1	11/2017	OWNERSHIP CHANGE

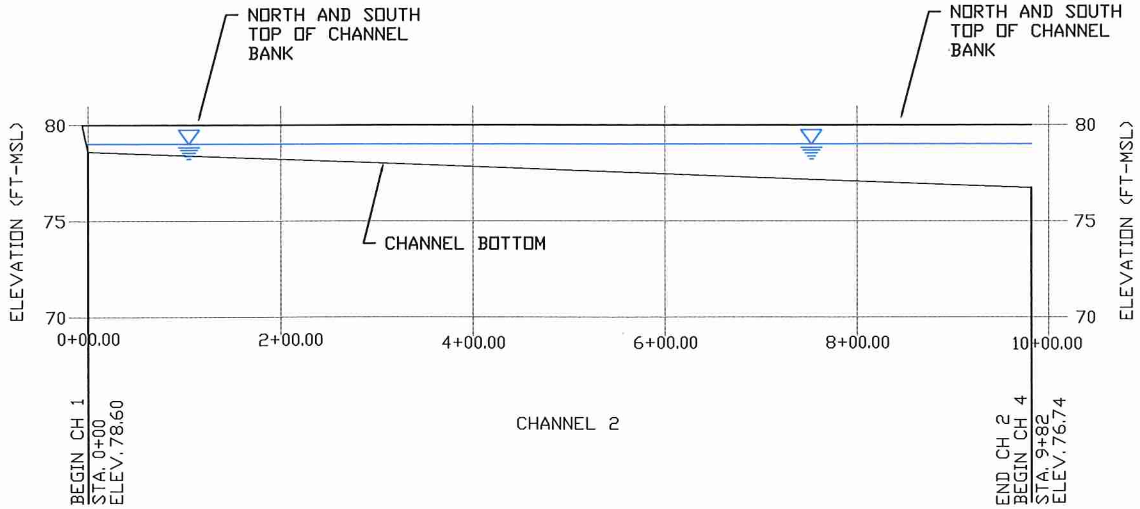
PERIMETER DRAINAGE PLAN

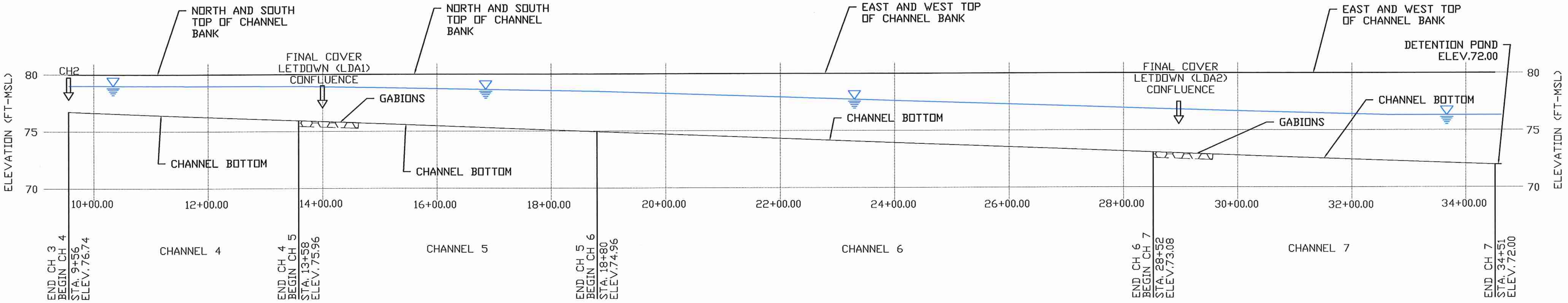
HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS

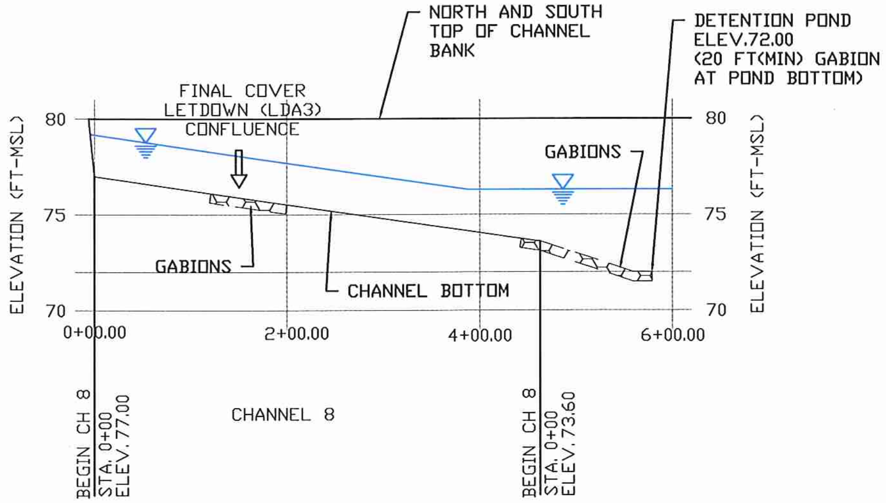
WWW.WCGRP.COM

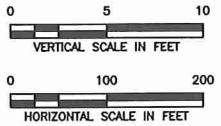
DRAWING IIIF.3











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84059

12-5-2017

NOTES:

1. UNLESS OTHERWISE IS SHOWN CHANNELS ARE LINED WITH GRASS ONLY. AREAS WITH HIGHER 5 FT/S VELOCITIES WILL BE LINED WITH TURF REINFORCEMENT AND GABIONS. FLOW VELOCITIES CALCULATED ARE LESS THAN 5 FT/S IN CHANNELS. GABIONS ARE SHOWN AT FINAL COVER DRAINAGE LETDOWN / PERIMETER CHANNEL CONFLUENCES AS WELL AS CH8 DISCHARGE LOCATION TO THE POND.

2. WATER SURFACE SHOWN FOR CHANNELS 1, 2, AND 3 IS BASED ON THE DOWNSTREAM CHANNEL WATER SURFACE TO REFLECT IN CHANNEL STORAGE AND DEMONSTRATE THAT CHANNEL BANKS FOR THESE CHANNELS WILL HAVE SUFFICIENT ELEVATIONS TO CONTAIN FLOW.

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DATE: 03/2017
FILE: 0120-758-11
CAD: IIIF.4-CHANNEL PROFILE.DWG

DRAWN BY: SRF
DESIGN BY: AE
REVIEWED BY: NT

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1	11/2017	OWNERSHIP CHANGE

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PERIMETER CHANNEL PROFILE

HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS

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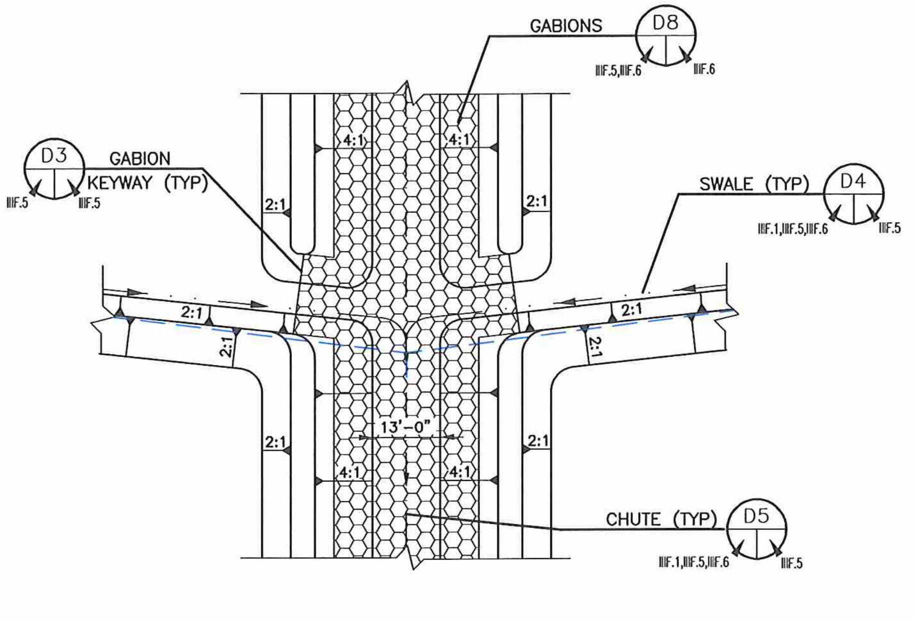
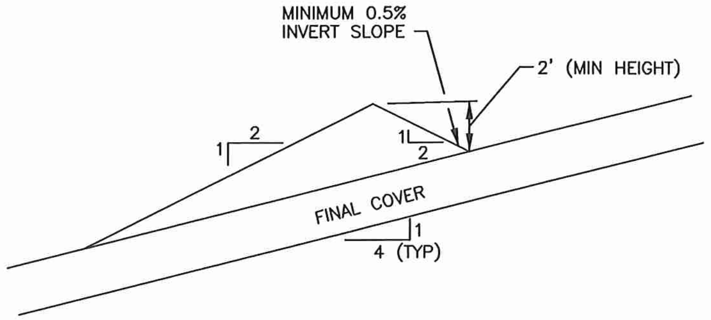
DRAWING IIIF.4

O:\0120\758\2214B EXPANSION\IIIF\IIIF.4-PERIMETER CHANNEL PROFILE.dwg, 11/15/2017 10:26:27 AM, rsellers, 1:2



SWALE/CHUTE CONFLUENCE (TYP) D1 IIF.1 IIF.6

0 20 40
SCALE IN FEET



SWALE (TYP) D4 IIF.1,IIF.5 IIF.5

0 4 8
SCALE IN FEET

NOTES:

1. REFER TO DRAWING IIIF.1–DRAINAGE STRUCTURE PLAN FOR LOCATION OF DETAILS.
2. BEDDING MATERIAL WILL CONSIST OF CLAYEY SOILS COMPACTED TO PROVIDE FIRM BASE THAT WILL BE OVERLAIN BY 8 oz/sy GEOTEXTILE PRIOR TO PLACEMENT OF GABIONS.
3. CHUTE DETAILS ARE SHOWN WITH 8 FEET OF BOTTOM WIDTH. SEE PAGE IIIF–C–9.
4. EROSION LAYER WILL BE CAPABLE OF SUSTAINING VEGETATIVE GROWTH.
5. BOTTOM OF GABION (i.e., GEOTEXTILE) WILL HAVE POSITIVE DRAINAGE TOWARD THE CENTER OF THE DRAINAGE LETDOWN WHICH DRAINS TOWARD THE TOE OF 25 PERCENT FINAL COVER TO PREVENT WATER ACCUMULATION AT THE BASE OF GABIONS.

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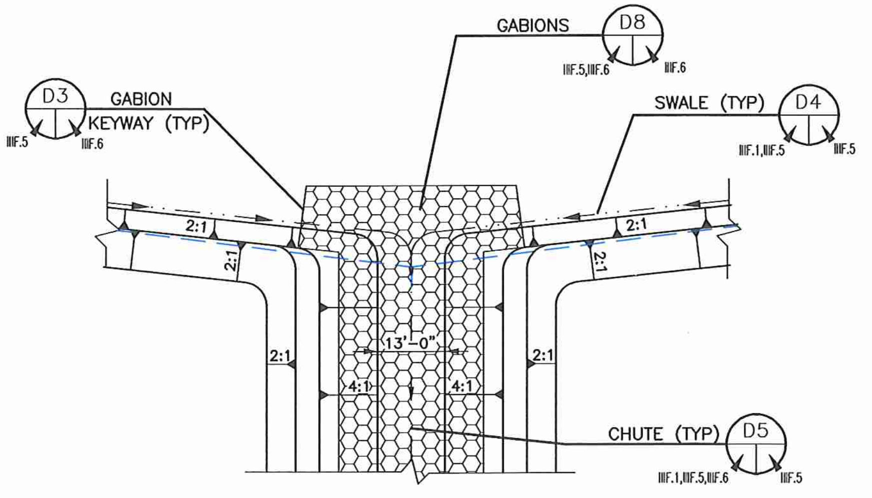
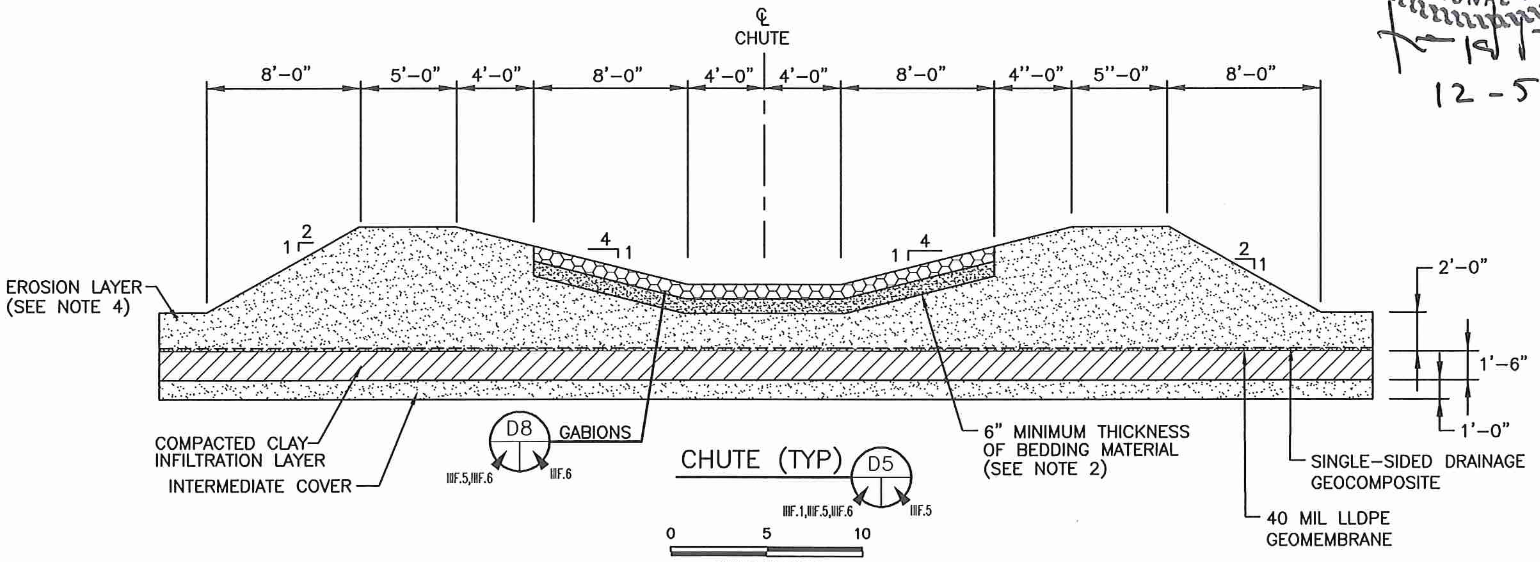


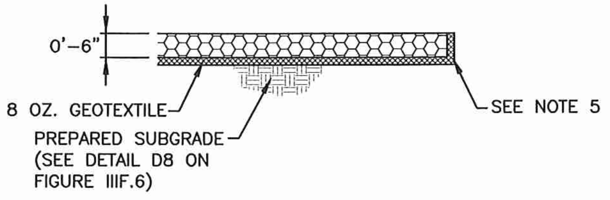
SWALE/CHUTE CONFLUENCE (TYP) D2 IIF.1 IIF.5

0 20 40
SCALE IN FEET



CHUTE (TYP) D5 IIF.1,IIF.5,IIF.6 IIF.5

0 5 10
SCALE IN FEET



GABION KEYWAY (TYP) D3 IIF.5 IIF.5

0 2 4
SCALE IN FEET

O:\0120\758\2214B EXPANSION\IIIF\IIIF.5-DRAINAGE DETAILS.dwg, 11/15/2017 10:26:52 AM, rsellers, 1:2

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DRAINAGE DETAILS

HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS

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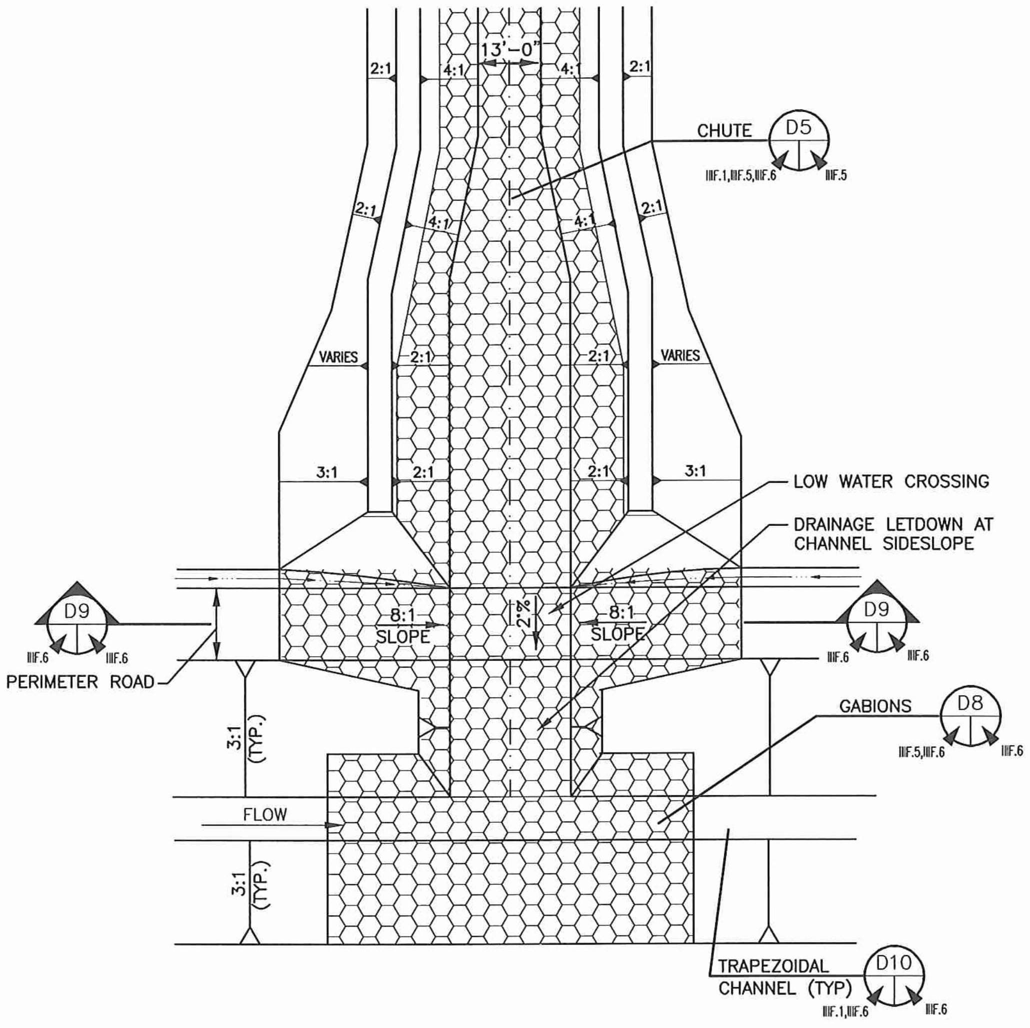
DRAWING IIIF.5

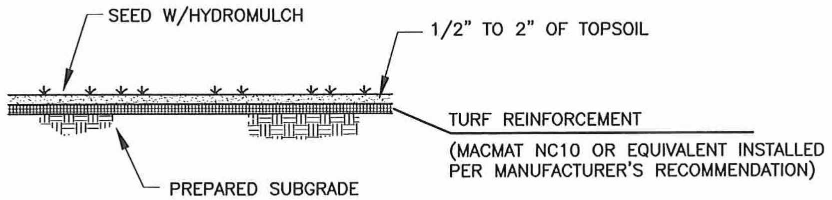


LOW-WATER CROSSING D6

IIIF.1 IIIF.6

0 20 40

SCALE IN FEET

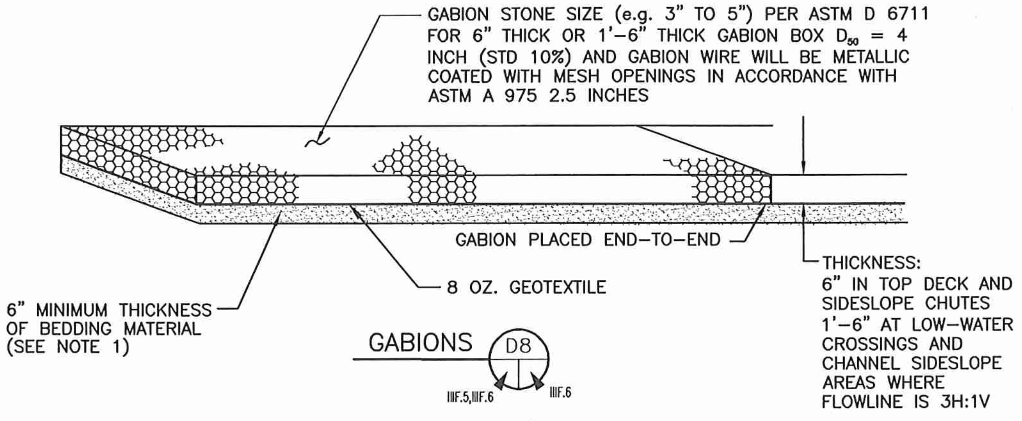


TURF REINFORCEMENT D7

NOT TO SCALE

(SEE NOTE 2)

IIIF.6 IIIF.6

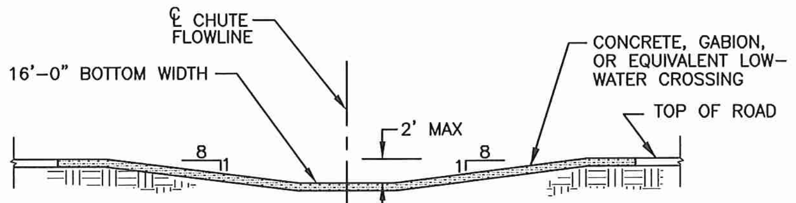


GABIONS D8

IIIF.5,IIIF.6 IIIF.6

0 2 4

SCALE IN FEET



LOW-WATER CROSSING SECTION D9

IIIF.6 IIIF.6

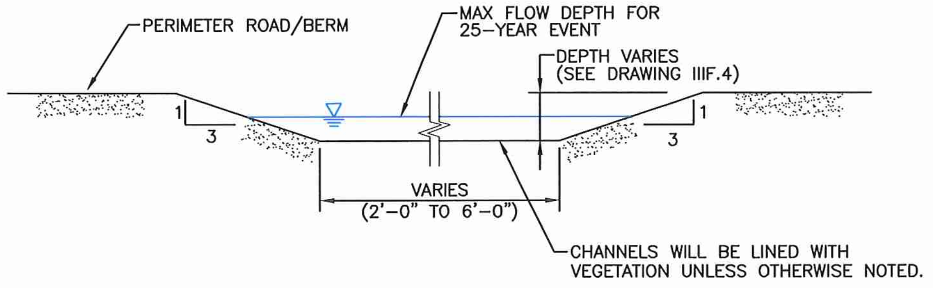
0 10 20

SCALE IN FEET

NEVZAT TURAN

84059

12-5-2017



TRAPEZOIDAL CHANNEL (TYP) D10

IIIF.1,IIIF.6 IIIF.6

0 2 4

SCALE IN FEET

NOTES:

1. BEDDING MATERIAL WILL CONSIST OF CLAYEY SOILS COMPACTED TO PROVIDE FIRM BASE THAT WILL BE OVERLAIN BY 8 oz/sy GEOTEXTILE PRIOR TO PLACEMENT OF GABIONS.
2. TURF REINFORCEMENT MAT WILL BE USED ON THE BERMS WITH MORE THAN 3H:1V SLOPES.

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DATE: 03/2017
FILE: 0120-758-11
CAD: IIIF.6-DRAINAGE DETAILS.DWG

DRAWN BY: JRP
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BFI WASTE SYSTEMS OF NORTH AMERICA, LLC

REVISIONS

NO.	DATE	DESCRIPTION
1	08/2017	FIRST NOD RESPONSE
2	11/2017	OWNERSHIP CHANGE

DRAINAGE DETAILS

HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS

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DRAWING IIIF.6

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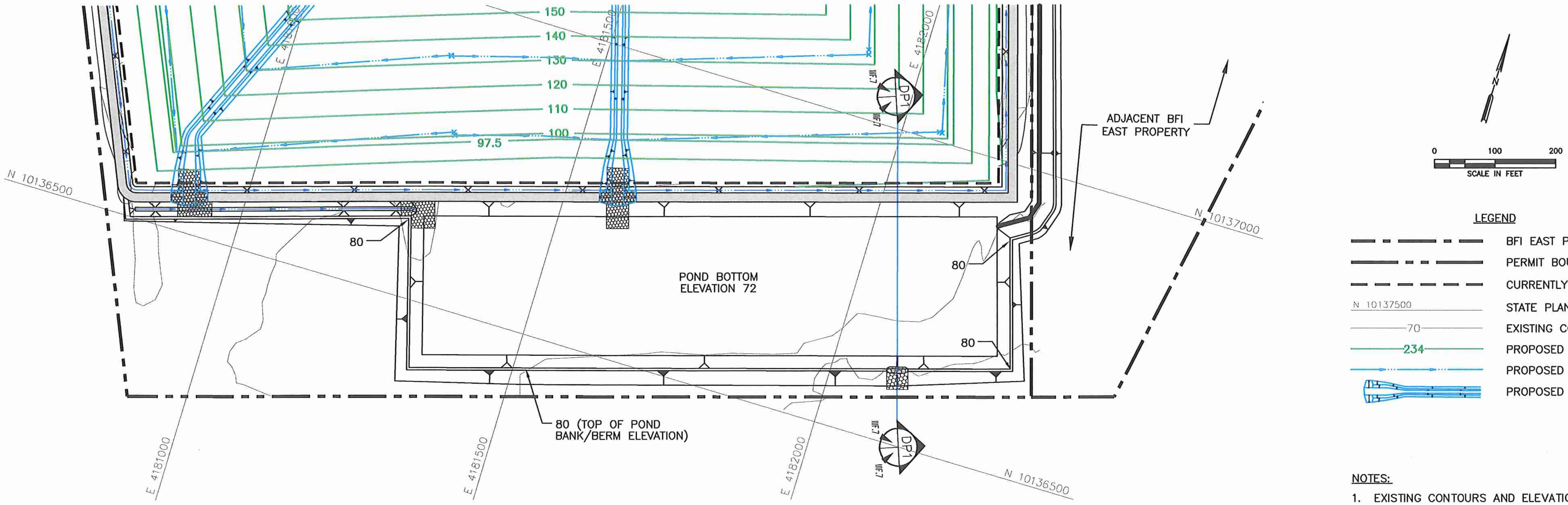
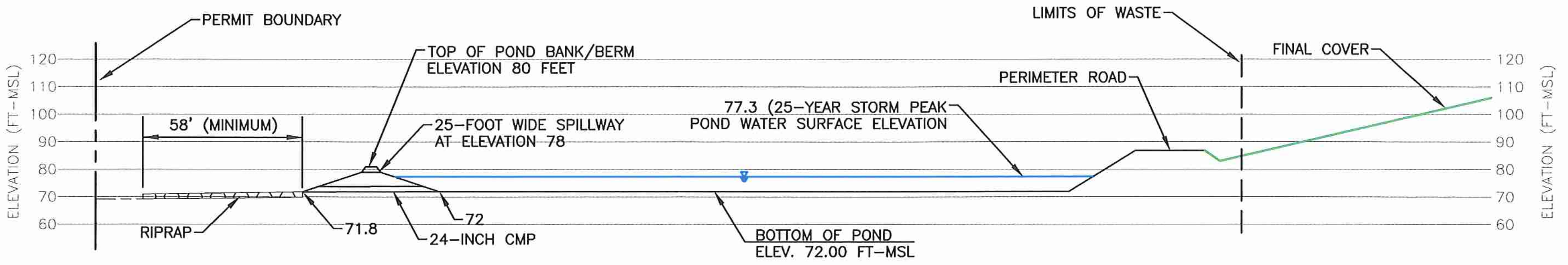


LEGEND

— — · — —	BFI EAST PROPERTY BOUNDARY
—— · · ——	PERMIT BOUNDARY
— — — —	CURRENTLY PERMITTED LIMITS OF WASTE
N 10137500	STATE PLANE COORDINATE GRID
70	EXISTING CONTOUR
234	PROPOSED FINAL CONTOUR
···→···	PROPOSED DRAINAGE SWALE
═══	PROPOSED DRAINAGE CHUTE

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS DEVELOPED BY WEAVER CONSULTANTS GROUP FROM AERIAL PHOTOGRAPHY FLOWN 05-17-2016. GRID SYSTEM IS TIED TO THE TEXAS STATE PLANE COORDINATE SYSTEM NAD 83, CENTRAL ZONE.



DETENTION POND P1 SECTION DP1 IIIF.7 IIIF.7

0 30 60 SCALE IN FEET

NEVZAT TURAN
84059

12-5-2017

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DATE: 03/2017
FILE: 0120-758-11
CAD: IIIF.7-DETENTION POND DETAILS.DWG

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DESIGN BY: AE
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REVISIONS

NO.	DATE	DESCRIPTION
1	11/2017	OWNERSHIP CHANGE

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

DETENTION POND INFORMATION

HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS

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DRAWING IIIF.7

APPENDIX IIIF-A

POST-DEVELOPMENT CONDITION HYDROLOGIC CALCULATIONS

STATE OF TEXAS
NEVZAT TURAN
84059
LICENSED PROFESSIONAL ENGINEER

12-5-2017

Includes pages IIIF-A-1 through IIIF-A-43

CONTENTS

STATE OF TEXAS
NEVZAT TURAN
84059
LICENSED
PROFESSIONAL ENGINEER

12-5-2017

HYPOTHETICAL STORM DATA

Hypothetical Storm Data

Precipitation data taken from TP-40 and Hydro 35 rainfall data.

Time	5 min	15 min	60 min	2 hr	3 hr	6 hr	12 hr	24 hr
25-Year Event	0.77	1.69	3.8	4.8	5.5	7	8.3	10

TP-40 (*U.S. Department of commerce, May 1961)* was used to identify precipitation values for storm durations from 60 minutes to 24 hours.

Hydro 35 (*National Oceanic and Atmospheric Administration, June 1977)* was used to estimate precipitation for the 5 and 15 minute duration storm events.

PRECIPITATION LOSS DATA

Required: Determine the SCS curve numbers for both on-site and off-site drainage areas for use in the HEC-HMS analysis.

References:

1. Dodson's and Associates, Inc., *ProHec-1 Plus Program Documentation*, 1995.
2. United States Department of Agriculture, Soil Conservation Service, "Soil Survey of Tarrant County, Texas".
3. The Hydrologic Evaluation of Landfill Performance (HELP) Model - Engineering Documentation for version 3. EPA/600/R-94/168b, September 1994.

Note: Approximate non landfill areas within the permit boundary on SCS map (page IIIF-A-8).

Solution: Based on the soil survey information found in Ref. 2, hydrologic group D soils predominate the soils within the permit boundary drainage area (see pages IIIF-A-5 and IIIF-A-6).

The on-site subbasins near the site were considered pasture land in fair conditions. A curve number was selected using the table on page IIIF-A-7.

Use: CN = 84

The final cover system was assumed to be in place and the erosion layer will control precipitation loss. A curve number for erosion layer may be selected using the chart on page IIIF-A-9.

Use: CN = 89 for topslopes
Use: CN = 88 for sideslopes

The pond area is assumed to collect all precipitation for directly on to the pond surface and 1% reduction is shown to account for the extended area of the banks and the top of berm.

Use: CN = 99

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Properties and Qualities Ratings

Open All | Close All

Soil Chemical Properties

Soil Erosion Factors

Soil Health

Soil Physical Properties

Soil Qualities and Features

- AASHTO Group Classification (Surface)
- Depth to a Selected Soil Restrictive Layer
- Depth to Any Soil Restrictive Layer
- Drainage Class
- Frost Action
- Frost-Free Days
- Hydrologic Soil Group

View Description | View Rating

View Options

Map ☐

Table ☑

Description of Rating ☑

Rating Options ✓

☐ Detailed Description

Advanced Options

Aggregation Method: Dominant Condition ▼

Component Percent Cutoff

Tie-break Rule: ○ Lower ● Higher

View Description | View Rating

- Map Unit Name
- Parent Material Name
- Representative Slope
- Unified Soil Classification (Surface)

Water Features

Tables — Hydrologic Soil Group — Summary By Map Unit

Summary by Map Unit — Hardin County, Texas (TX199)

Map unit symbol	Map unit name	Rating	Acres in AOI	Percent AOI
AnhA	Anahuac-Aris complex, 0 to 1 percent slopes	D	—	
ArsA	Aris-Levac complex, 0 to 1 percent slopes	D	—	
AspA	Aris-Spindletop complex, 0 to 1 percent slopes	D	—	
BatA	Batson very fine sandy loam, 0 to 1 percent slopes	B	—	
BeaA	Beaumont clay, 0 to 1 percent slopes	D	—	
BelB	Belrose loamy fine sand, 0 to 3 percent slopes	A	—	
BemA	Belrose-Caneyhead complex, 0 to 1 percent slopes	A	—	
BevA	Bevil clay, 0 to 1 percent slopes	D	—	
CamA	Camptown silt loam, 0 to 1 percent slopes, frequently ponded	C	—	
CapA	Camptown-Batson complex, 0 to 1 percent slopes	C	—	
CowA	Cowmarsh mucky clay, 0 to 1 percent slopes, frequently flooded, frequently ponded	D	—	
DAMX	Dam	D	—	
EvaA	Evadale silt loam, 0 to 1 percent slopes	C/D	—	
EvdA	Evadale-Aldine complex, 0 to 1 percent slopes	C/D	—	
EvgA	Evadale-Gist complex, 0 to 1 percent slopes	C/D	—	
HatA	Hatliff-Pluck-Kian complex, 0 to 1 percent slopes, frequently flooded	A	—	
JasA	Jasco silt loam, 0 to 1 percent slopes	D	—	
JayA	Jayhawker silt loam, 0 to 1 percent slopes	C/D	—	
KefB	Kenefick very fine sandy loam, 0 to 3 percent slopes	C	—	
KenA	Kenefick-Caneyhead complex, 0 to 1 percent slopes, frequently ponded	C	—	
KibB	Kirbyville fine sandy loam, 0 to 2 percent slopes	C/D	—	
KinB	Kirbyville-Niwana complex, 0 to 2 percent slopes	C/D	—	
KouB	Kountze very fine sandy loam, 0 to 2 percent slopes	C/D	—	
LalA	Labelle-Levac complex, 0 to 1 percent slopes	C	—	
LasA	Labelle-Spindletop complex, 0 to 1 percent slopes	C	—	
LeaA	League clay, 0 to 1 percent slopes	D	—	
LelA	Lelavale silt loam, 0 to 1 percent slopes, ponded	C/D	—	
LetA	Leton loam, 0 to 1 percent slopes, occasionally flooded, frequently ponded	D	—	
McnC	McNeely sand, 1 to 5 percent slopes	A	—	
NonA	Nona-Dallardsville complex, 0 to 1 percent slopes	C/D	—	
Oa	Oil wasteland	D	—	
OliA	Olive silt loam, 0 to 1 percent slopes	D	—	
OlvA	Olive-Dallardsville complex, 0 to 1 percent slopes, frequently ponded	D	—	
OtaB	Otanya very fine sandy loam, 1 to 3 percent slopes	B	—	
OtbC	Otanya very fine sandy loam, 3 to 5 percent slopes	C	—	
PlaA	Plank silt loam, 0 to 1 percent slopes	D	—	
SilC	Silsbee fine sandy loam, 3 to 5 percent slopes	C	—	
SilD	Silsbee loamy fine sand, 5 to 12 percent slopes	C	—	
SimA	Simelake clay, 0 to 1 percent slopes, frequently flooded	D	—	
SipA	Simelake-Pluck complex, 0 to 1 percent slopes, frequently flooded	D	—	
SolA	Sorter silt loam, 0 to 1 percent slopes	B/D	—	
SomA	Sorter-Dallardsville complex, 0 to 1 percent slopes	D	—	
SovA	Sourlake loam, 0 to 1 percent slopes, frequently flooded	C/D	—	
SpuB	Spurger very fine sandy loam, 0 to 3 percent slopes	C	—	
SpyA	Spurger-Caneyhead complex, 0 to 1 percent slopes	C	—	
TelB	Texla silt loam, 0 to 2 percent slopes	D	—	
TurB	Turkey sand, 1 to 3 percent slopes	A	—	

Summary by Map Unit — Hardin County, Texas (TX199)

Map unit symbol	Map unit name	Rating	Acres in AOI	Percent AOI
TybA	Tyden-Babco complex, 0 to 1 percent slopes, frequently ponded	C/D	—	
VamA	Vamont clay, 0 to 1 percent slopes	C/D	—	
VigA	Vidor-Gist complex, 0 to 1 percent slopes	C/D	—	
VtaA	Votaw fine sand, 0 to 1 percent slopes	B	—	
W	Water	D	—	
WarA	Waller-Dallardsville complex, 0 to 1 percent slopes	D	—	
Totals for Area of Interest			**574,630.3**	**100.**

Description — Hydrologic Soil Group

Hydrologic soil groups are based on estimates of runoff potential. Soils are assigned to one of four grou according to the rate of water infiltration when the soils are not protected by vegetation, are thoroughly wet, and receive precipitation from long-duration storms.

The soils in the United States are assigned to four groups (A, B, C, and D) and three dual classes (A/D, B/D, and C/D). The groups are defined as follows:

Group A. Soils having a high infiltration rate (low runoff potential) when thoroughly wet. These consist mainly of deep, well drained to excessively drained sands or gravelly sands. These soils have a high rat water transmission.

Group B. Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderat deep or deep, moderately well drained or well drained soils that have moderately fine texture to moderately coarse texture. These soils have a moderate rate of water transmission.

Group C. Soils having a slow infiltration rate when thoroughly wet. These consist chiefly of soils having layer that impedes the downward movement of water or soils of moderately fine texture or fine texture. These soils have a slow rate of water transmission.

Group D. Soils having a very slow infiltration rate (high runoff potential) when thoroughly wet. These consist chiefly of clays that have a high shrink-swell potential, soils that have a high water table, soils t have a claypan or clay layer at or near the surface, and soils that are shallow over nearly impervious material. These soils have a very slow rate of water transmission.

If a soil is assigned to a dual hydrologic group (A/D, B/D, or C/D), the first letter is for drained areas an the second is for undrained areas. Only the soils that in their natural condition are in group D are assign to dual classes.

Rating Options — Hydrologic Soil Group

Aggregation Method: Dominant Condition

Component Percent Cutoff: *None Specified*

Tie-break Rule: Higher

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TABLE 5.3 Values of SCS Curve Number for Rural Areas

Source: [McCuen, 1982]

	Hydrologic Soil Group			
Land Use Description	**A**	**B**	**C**	**D**
Fallow:				
Straight Row	77	86	91	94
Row Crops:				
Straight Row, Poor Condition	72	81	88	91
Straight Row, Good Condition	67	78	85	89
Contoured, Poor Condition	70	79	84	88
Contoured, Good Condition	65	75	82	86
Contoured and Terraced, Poor Condition	66	74	80	82
Contoured and Terraced, Good Condition	62	71	78	81
Small Grain:				
Straight Row, Poor Condition	65	76	84	88
Straight Row, Good Condition	63	75	83	87
Contoured, Poor Condition	63	74	82	85
Contoured, Good Condition	61	73	81	84
Contoured and Terraced, Poor Condition	61	72	79	82
Contoured and Terraced, Good Condition	59	70	78	81
Close-Seeded Legumes or Rotation Meadow				
Straight Row, Poor Condition	66	77	85	89
Straight Row, Good Condition	58	72	81	85
Contoured, Poor Condition	64	75	83	85
Contoured, Good Condition	55	69	78	83
Contoured and Terraced, Poor Condition	63	73	80	83
Contoured and Terraced, Good Condition	51	67	76	80
Pasture or Range:				
Poor Condition	68	79	86	89
Fair Condition	49	69	79	84
Good Condition	39	61	74	80
Contoured, Poor Condition	47	67	81	88
Contoured, Fair Condition	25	59	75	83
Contoured, Good Condition	6	35	70	79
Meadow, Good Condition	30	58	71	78
Woods or Forest Land:				
Poor Condition	45	66	77	83
Fair Condition	36	60	73	79
Good Condition	25	55	70	77
Farmsteads:	59	74	82	86

Initial and Uniform Loss Rate

An initial loss in inches (*STRTL*) and a constant loss rate (*CNSTL*) in inches per hour are specified for this method. All rainfall is lost until the volume of initial loss is satisfied. After the initial loss is satisfied, rainfall is lost at the constant rate.

This section provides guidance in selecting the values used for the initial loss and uniform loss rate in two ways:

1. By consulting previous studies of actual rainfall events for a particular watershed or region.
2. By relating the parameters to the SCS Curve Number, which can be estimated using the information presented earlier in this chapter.

Previous studies by the U.S. Army Corps of Engineers or other public agencies may provide guidance on selecting appropriate values for the initial loss and uniform loss rate for a particular location. Tables 5.4 through 5.6 list the values of initial and

IIIF-A-8

HYDROGRAPH DEVELOPMENT INFORMATION

HYDROGRAPH DEVELOPMENT INFORMATION

Landfill Areas

Direct runoff methods, (i.e., kinematic wave) have been used for the landfill final cover areas. The kinematic wave method has been used to model the 3 percent topslope areas and 25 percent side slope areas before the flow is intercepted by the drainage swales. The kinematic wave method is a physically based method using slope, surface roughness, catchment lengths and areas. This method does not consider attenuation for flood wave; as a consequence, this method provides for a conservative analysis. The following typical parameters for the kinematic wave method have been developed for landfill areas.

Kinematic wave parameters for overland flow:

Slope:	Varies from 0.04 to 0.25 ft/ft landfill slopes
N:	0.3 Manning's friction coefficient (based on using a value between dense grass (N = 0.24) and Bermuda grass (N = 0.41) listed in Soil Conservation Services TR-55)
L:	Represents a typical distance between swales for overland flow for each drainage area. For example, as shown on Sheet IIIF-A-4, the swale spacing on 4H:1V sideslopes is 120 feet.

Percentage of drainage area represented by this element is 100 percent.

Kinematic Wave routing for channels:

- Channel length (ft): The length of the channel section.
- Channel slope (ft/ft): Varies from 0.0016 to 0.005 (0.005 for swales).
- Channel roughness coefficient: 0.03 for grass lined channels and swales.
- Channel type: A trapezoidal channel was used with varying width and 3:1 side slopes ("V" ditch with varying side slopes for swales).

Non-Landfill Final Cover Areas

Hydrographs for the majority of non-landfill final cover areas within and near the permit boundary (e.g., pond areas) were developed using the Snyder unit hydrograph method.

The Espey "10-Minute" method has been used to estimate the Snyder parameter. Snyder parameter estimations are provided on page IIIF-A-16.

As discussed in Section 2 of Appendix IIIF, hydrographs for the areas inside the permit boundary (NA1, NA2, WA1, SA1, EA1, CH1, AND CH2) were developed using the Snyder unit hydrograph method. The percent imperviousness ranges from 0 percent to 50 percent, for the non-landfill on-site areas. Pond areas are assumed to be 99 percent impervious, and areas with significant channel surface or paved surfaces were assigned higher percentages of impervious area, as shown on IIIF-A-16.

Drainage Areas

The drainage areas used for this analysis is shown on Sheet IIIF-A-22. The routing scheme for the post-development condition is shown in the HEC-HMS output file presented in Appendix IIIF-A-26.

DISTRIBUTED RUNOFF METHOD
KINEMATIC WAVE EXAMPLE

Prep By: NT
Date: 3-31-17

HARDIN COUNTY LANDFILL
0120-758-11-02
KINEMATIC WAVE PARAMETERS

Chkd By: JRP
Date: 3-31-17

Drainage area "LDA3" is used in this example (refer to Sheet IIIF-A-15 for location of drainage area).

Watershed Specific Parameters:

A =	11.51	acres	Watershed Area (acres)
A =	0.0180	sq-miles	Watershed Area (sq-miles)
CN=	88		SCS Curve Number (see sheet IIIF-A-9 for more information)

Kinematic Wave parameter for overland flow:

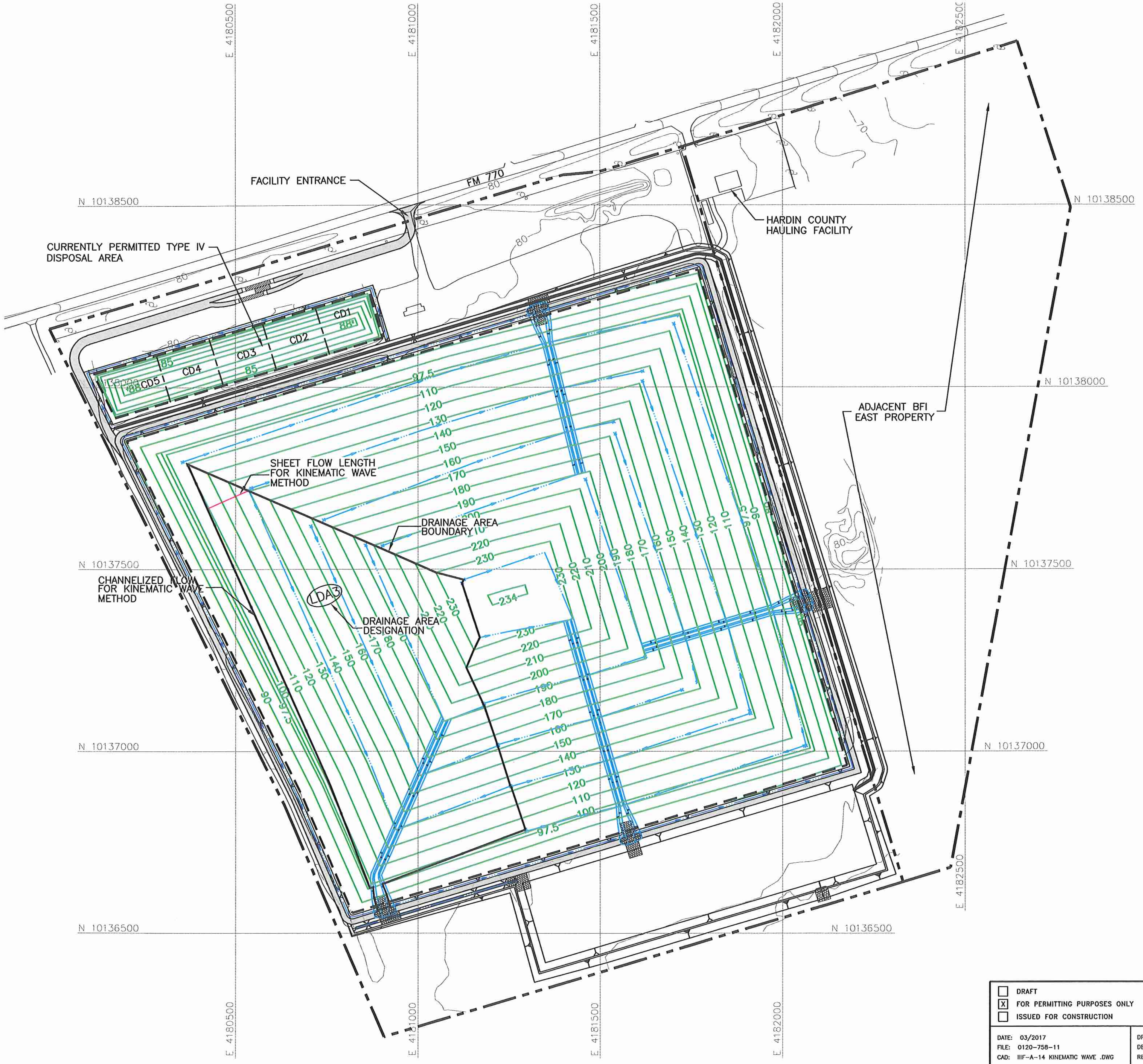
L=	120	ft	Typical overland flow (ft)
S=	0.25	ft/ft	Landfill slope (ft/ft)
N=	0.30		Manning's Coefficient

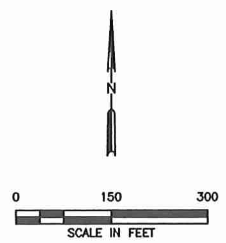
Percentage of the drainage area represented by this element is 100 percent

Muskingum-Cunge routing data for the swale:

L=	1283.13	ft	Typical swale length (ft)
S=	0.005	ft/ft	Swale bottom slope (ft/ft)
N=	0.03		Manning's Coefficient
Channel=	TRAP		Swale Type*

* A trapezoidal channel with no bottom width was used to simulate a triangular channel.





LEGEND

— — · —	BFI EAST PROPERTY BOUNDARY
—— · · ——	PERMIT BOUNDARY
— — — —	CURRENTLY PERMITTED LIMITS OF WASTE
—— — · ——	CELL BOUNDARY
N 10137500	STATE PLANE COORDINATE GRID
70	EXISTING CONTOUR
234	PROPOSED FINAL CONTOUR (SEE NOTE 3)
··→··	PROPOSED DRAINAGE SWALE
▨	PROPOSED DRAINAGE CHUTE

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS DEVELOPED BY WEAVER CONSULTANTS GROUP FROM AERIAL PHOTOGRAPHY FLOWN 05-17-2016. GRID SYSTEM IS TIED TO THE TEXAS STATE PLANE COORDINATE SYSTEM NAD 83, CENTRAL ZONE.

NEVZAT TURAN
84059

12-5-2017

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☐ ISSUED FOR CONSTRUCTION

DATE: 03/2017
FILE: 0120-758-11
CAD: IIIF-A-14 KINEMATIC WAVE .DWG

DRAWN BY: SRF
DESIGN BY: AE
REVIEWED BY: NT

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

PREPARED FOR
BFI WASTE SYSTEMS OF NORTH AMERICA, LLC

REVISIONS

NO.	DATE	DESCRIPTION
1	11/2017	OWNERSHIP CHANGE

KINEMATIC WAVE PARAMETERS

HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS

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SHEET IIIF-A-14

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ESPEY 10-MINUTE METHOD PARAMETERS

Prep By: NT
Date: 3-31-17

Chkd By: JRP
Date: 3-31-17

HARDIN COUNTY LANDFILL
0120-758-11-02
UNIT HYDROGRAPH DATA
PROPOSED EXPANSION CONDITION

Snyder's Hydrograph Coefficients (Espey's 10 Minute Method)

Proposed Expansion Conditions

Area No.	Area (acres)	Max. Flow Length (L) (ft)	S (ft/ft)	I (%)	Manning "n"	Φ^1	T_r^2 (min)	T_{lag}^3 (min)	T_{lag} (hr)	Area[4] (sq mi)	q_p^5 (cfs/sq mi)	C_p^6
NA1	1.25	337	0.0089	25	0.04	0.86	17.0	12.0	0.20	0.0020	1954.4	0.61
NA2	1.04	469	0.0064	2	0.04	0.86	31.4	26.4	0.44	0.0016	1022.0	0.70
WA1	1.22	799	0.0017	50	0.04	0.86	27.6	22.6	0.38	0.0019	1166.4	0.69
SA1	5.99	315	0.0254	0	0.04	0.86	52.7	47.7	0.80	0.0094	547.5	0.68
EA1	3.77	820	0.0073	20	0.04	0.86	22.8	17.8	0.30	0.0059	1365.7	0.63
CH1	0.23	129	0.0618	10	0.04	0.86	9.9	4.9	0.08	0.0004	3727.5	0.48
CH2	3.41	324	0.0185	5	0.04	0.86	18.7	13.7	0.23	0.0053	1692.8	0.61

[1] Conveyance efficiency coefficient from Dodson & Associates Inc., *ProHec-1 Program Documentation* , 1995, pages 6-19 and 6-20.

[2] $T_r = 3.1(L^{0.23})(S^{-0.25})(I^{-0.18})(\Phi^{1.57})$

[3] $T_{lag} = T_r - \Delta t/2$

[4] From area summary sheet

[5] $q_p = 31600(A^{-0.04})(T_r^{-1.07})$

[6] $C_p = 49.375(A^{-0.04})(T_r^{-1.07})(T_{lag})$

T_r = surface runoff to unit hydrograph peak (min)
L = distance along main channel from study point to watershed boundary (ft)
S = main channel slope (ft/ft)
I = impervious cover within the watershed (%)
T_{lag} = watershed lag time (min)
Δt= computation interval (minutes)
q_p = unit hydrograph peak discharge (cfs/sq mi)
C_p = Snyder's peaking coefficient

HARDIN COUNTY LANDFILL
0120-758-11-02
ESPEY 10 MINUTE SAMPLE CALCULATION

Snyder Unit Hydrograph uses lag time (T_{lag}) and peaking coefficient accounting for flood wave and watershed storage conditions.

Drainage area "SA1" is used in this example.

Estimated Watershed specific parameters

A =	5.99	acres	watershed area
L =	315	feet	maximun flow length with this watershed
S =	0.0254	feet/feet	watershed slope
I =	0	percent (%)	watershed imperviousness
n =	0.03		Manning's coefficient

Calculate Tr: time beginning of surface runoff to the unit hydrograph peak in minutes

$T_r = 3.1(L^{0.23})(S^{-0.25})(I^{-0.18})(\Phi^{1.57})$

Estimate : conveyance efficiency coefficient
See figure 6.12 on page IIIF-A-18 for estimating
Φ = for 0.6 percent impervious cover and n = 0.01

$\Phi = 0.9$

$T_r = 3.1(315^{0.23})(.010^{-0.25})(0.01^{-0.18})(0.9^{1.57})$

$T_r = 52.7$ min

Calculate T_{lag}: watershed lag time

$T_{lag} = Tr - (\Delta t/2)$

$T_{lag} = 47.7$ minutes

$T_{lag} = 0.80$ hours

Δt is calculation interval, and 10 minutes is used in the HEC - HMS modeling in this project

$A = A/640$

$A = 0.0094$ square miles

Calculate q_p: peak discharge of unit hydrograph per unit area (cfs/sq. mi).

$q_p = 31600(A^{-0.04})(T_r^{-1.07})$

$q_p = 31600(0.0351^{-0.04})(29.6^{-1.07})$

$q_p = 547.6$ cfs/sq. mi

Calculate Peaking coefficient C_p:

$C_p = 49.375(A^{-0.04})(T_r^{-1.07})(T_{lag})$

$C_p = 49.375(0.0351^{-0.04})(29.6^{-1.07})(0.45)$

$C_p = 0.68$

compute the value of Snyder's peaking coefficient C_p for use in HEC-1 analyses. First, the watershed lag time T_L is determined by subtracting one-half of the computation interval from the time to rise ($T_L = T_r - \Delta t/2$). Then, C_p may be computed by substituting the known values of T_L and q_p into Snyder's equation for peak unit hydrograph flow rate and solving for C_p.

$$C_p = \frac{q_p \times T_L}{640} \tag{6.30}$$

Espey "10-Minute" Method for Estimating Snyder Parameters

In another study, Espey [1977] derived the following equation for computing the time from the beginning of surface runoff to the unit hydrograph peak:

$$T_r = 3.10\, L^{0.23} S^{-0.25} I^{-0.18}\, \Phi^{1.57} \tag{6.31}$$

in which:

T_r = time from beginning of surface runoff to unit hydrograph peak (minutes)

L = total distance along main channel from study point to watershed boundary (feet)

S = main channel slope between the reference point and a point 0.2L downstream from the upstream watershed boundary (feet per foot)

I = impervious cover within the watershed (percent)

Φ = description of conveyance efficiency of the watershed drainage system.

The conveyance efficiency coefficient Φ is determined using the relationships illustrated on Figure 6.12.

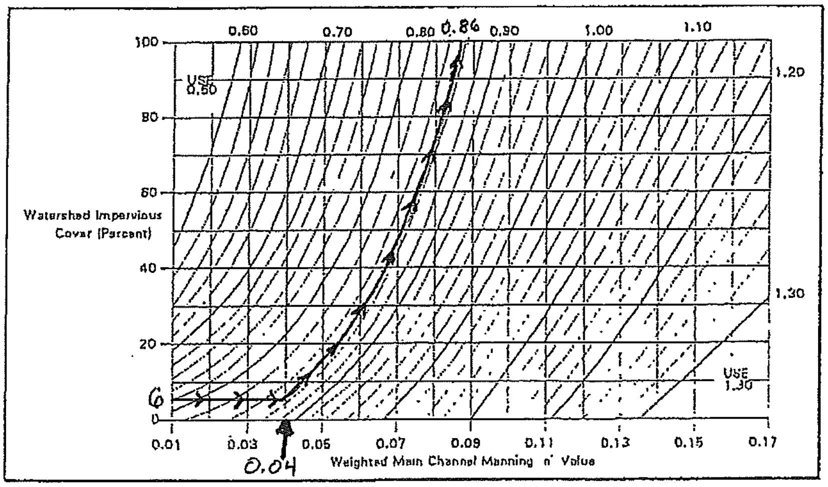


FIGURE 6.12 Determination of Conveyance Efficiency Coefficient Φ

This equation was derived from records for 41 watersheds in Texas, Tennessee, Mississippi, Pennsylvania, North Carolina, Colorado, Kentucky, and Indiana. The range in the watershed characteristics used to develop the equations for urban areas were:

Area : From 0.0128 square miles to 15.00 square miles

L : From 555 feet to 35,600 feet

S : From 0.0005 ft. per ft. to 0.0295 ft. per ft.

I: From 2% to 100%

Φ : From 0.60 to 1.30

Again, note that the time to rise T_r is not the same as the watershed lag time T_p. The difference between the two is that T_r is defined as the time from the beginning of effective rainfall to the peak of the unit hydrograph, while T_L is the time from the centroid of the effective rainfall to the peak of the unit hydrograph. For the purposes of HEC-1 analyses, however, T_L may be determined simply by subtracting one-half the computation time interval from the computed value of T_r (T_R - $\Delta t/2$).

The relationship developed by Espey to compute the peak flow rate of the unit hydrograph is as follows:

6.32 $$Q_u = 31600 \quad A^{0.96} T_r^{-1.07}$$

in which:

Q_u = unit hydrograph peak discharge (cfs)

A = drainage area (square miles)

T_r = time of rise from beginning of surface runoff to unit hydrograph peak (minutes)

Riverside County Method for Estimating Snyder Parameters

Three watershed lag equations have been derived for use in rural areas of Riverside County, California by the Riverside County Flood Control and Water Conservation District [Anonymous, 1963]. These equations differ slightly from those developed at the Tulsa District of the U.S. Army Corps of Engineers in that lag is defined as the time from the beginning of rainfall to the point on the unit hydrograph corresponding to one-half of the total runoff volume.

Each equation is applicable to a different topographic region:

6.33 $$T_L = 1.20\left(\frac{L \times L_{ca}}{\sqrt{S}}\right)^{0.38} \quad \text{(Mountain Areas)}$$

6.34 $$T_L = 0.72\left(\frac{L \times L_{ca}}{\sqrt{S}}\right)^{0.38} \quad \text{(Foothill Areas)}$$

6.35 $$T_L = 0.38\left(\frac{L \times L_{ca}}{\sqrt{S}}\right)^{0.38} \quad \text{(Valley Areas)}$$

in which:

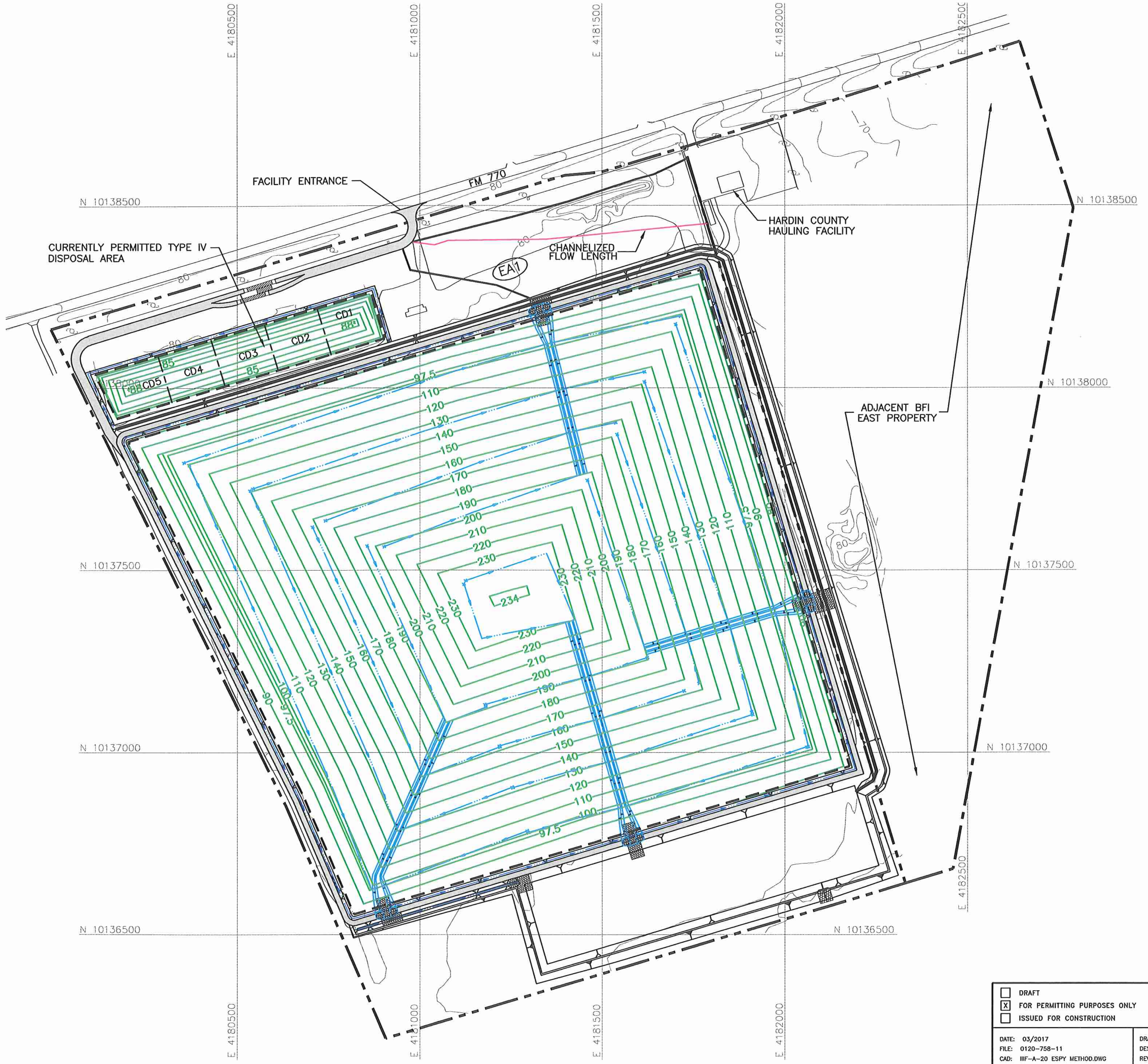
T_L = watershed lag in hours

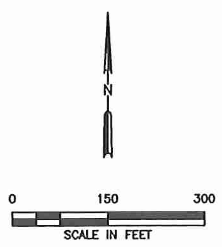
L = watershed length in miles

L_{ca} = length to centroid in miles

S = watershed slope in feet per mile.

The sizes of the watersheds studied in developing these equations ranged from 2.3 square miles to 645 square miles.





LEGEND

— - — - —	BFI EAST PROPERTY BOUNDARY
—— - - ——	PERMIT BOUNDARY
— — — — —	CURRENTLY PERMITTED LIMITS OF WASTE
—— — ——	CELL BOUNDARY
N 10137500	STATE PLANE COORDINATE GRID
—70—	EXISTING CONTOUR
—234—	PROPOSED FINAL CONTOUR (SEE NOTE 3)
—→—	PROPOSED DRAINAGE SWALE
▭	PROPOSED DRAINAGE CHUTE
——	DRAINAGE AREA BOUNDARY
EA1	DRAINAGE AREA DESIGNATION

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS DEVELOPED BY WEAVER CONSULTANTS GROUP FROM AERIAL PHOTOGRAPHY FLOWN 05-17-2016. GRID SYSTEM IS TIED TO THE TEXAS STATE PLANE COORDINATE SYSTEM NAD 83, CENTRAL ZONE.

NEVZAT TURAN
84059

12-5-2017

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PREPARED FOR
BFI WASTE SYSTEMS OF NORTH AMERICA, LLC

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DESIGN BY: AE
REVIEWED BY: NT

REVISIONS

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ESPEY "10-MINUTE" METHOD PARAMETERS

HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS

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SHEET IIIF-A-20

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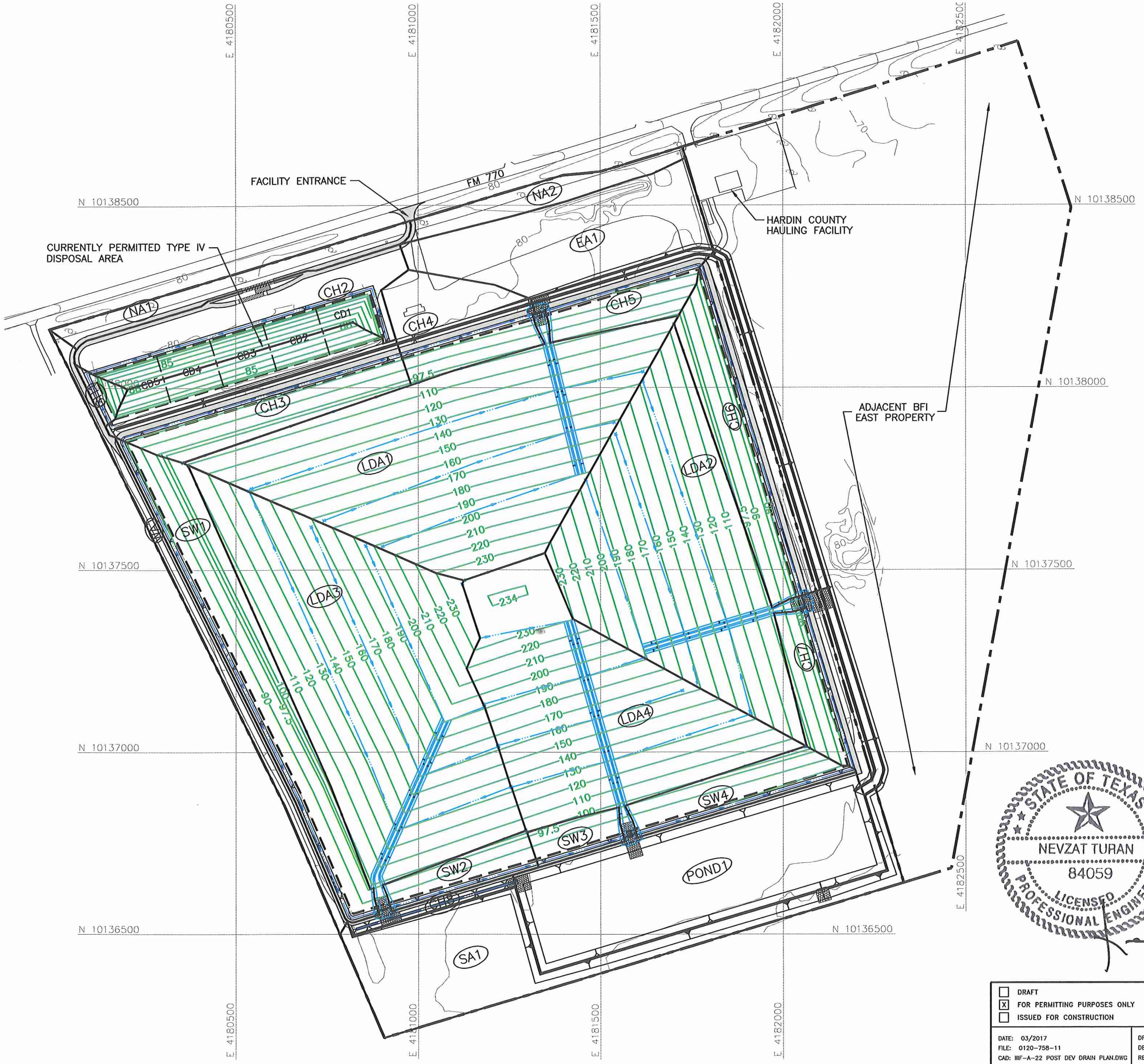
POST-DEVELOPMENT HEC-1 ANALYSIS
DRAINAGE AREAS



	LEGEND
— - - —	BFI EAST PROPERTY BOUNDARY
— · · —	PERMIT BOUNDARY
— — — —	CURRENTLY PERMITTED LIMITS OF WASTE
— — - —	CELL BOUNDARY
N 10137500	STATE PLANE COORDINATE GRID
70	EXISTING CONTOUR
234	PROPOSED FINAL CONTOUR (SEE NOTE 3)
→ →	PROPOSED DRAINAGE SWALE
	PROPOSED DRAINAGE CHUTE
——	DRAINAGE AREA BOUNDARY
LDA2	DRAINAGE AREA DESIGNATION

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS DEVELOPED BY WEAVER CONSULTANTS GROUP FROM AERIAL PHOTOGRAPHY FLOWN 05-17-2016. GRID SYSTEM IS TIED TO THE TEXAS STATE PLANE COORDINATE SYSTEM NAD 83, CENTRAL ZONE.

DRAINAGE AREA NO.	AREA (ACRES)	DRAINAGE AREA NO.	AREA (ACRES)
NA1	1.25	CH7	1.77
NA2	1.04	CH8	0.38
WA1	1.216	LDA1	10.69
EA1	3.78	LDA2	8.74
SA1	4.54	LDA3	11.51
CH1	0.22	LDA4	7.89
CH2	3.41	SW1	3.88
CH3	3.85	SW2	0.88
CH4	2.10	SW3	0.57
CH5	1.47	SW4	1.32
CH6	3.56	POND 1	6.34
TOTAL OF THE LISTED AREAS =			80.41

NEVZAT TURAN
84059

12-5-2017

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DATE: 03/2017
FILE: 0120-758-11
CAD: IIIF-A-22 POST DEV DRAIN PLAN.DWG

DRAWN BY: SRF
DESIGN BY: AE
REVIEWED BY: NT

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

PREPARED FOR
BFI WASTE SYSTEMS OF NORTH AMERICA, LLC

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NO.	DATE	DESCRIPTION
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POST-DEVELOPMENT DRAINAGE AREA PLAN

HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS

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SHEET IIIF-A-22

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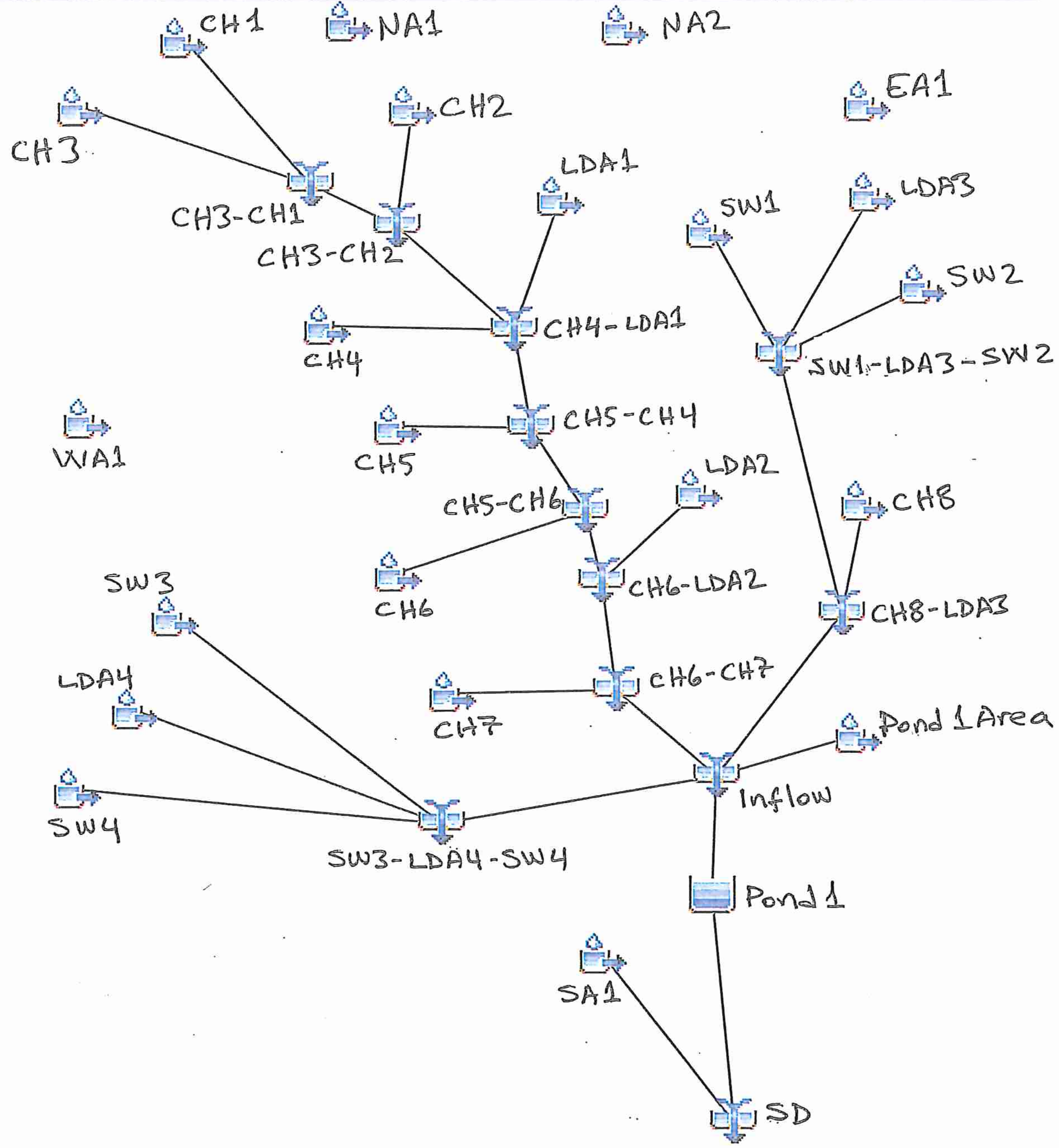
HEC-HMS OUTPUT – POST-DEVELOPMENT
25-YEAR, 24-HOUR STORM EVENT

HEC-HMS

Project : Hardin County LF-Post

Basin Model : HCLF-Post

Mar 27 19:06:26 CDT 2017



Subbasin Areas

Subbasin Designation	Square Miles
LDA1	0.016711
CH3	0.006023
CH1	0.000348
CH2	0.005326
CH4	0.003282
CH5	0.0023
CH6	0.00557
LDA2	0.01366
CH7	0.002771
LDA3	0.017981
SW1	0.006064
SW2	0.001379
CH8	0.0006
LDA4	0.01233
SW4	0.002055
SW3	0.00089
Pond1 Area	0.0099
SA1	0.0071
NA1	0.001954
WA1	0.0019
NA2	0.001623
EA1	0.0059

Kinematic Wave Transformation Input Parameters

Subbasin Designation	Length (ft)	Slope (ft/ft)	Roughness	Contributing Area (%)	Routing Steps
LDA1	120	0.25	0.3	100	5
CH3	141.7376	0.2	0.3	100	5
CH4	119.321	0.2	0.3	100	5
CH5	358.1459	0.019936	0.3	100	5
CH6	173.4218	0.2	0.3	100	5
LDA2	120	0.25	0.3	100	5
CH7	159.2612	0.186064	0.3	100	5
LDA3	120	0.25	0.3	100	5
SW1	135	0.25	0.3	100	5
SW2	94.2573	0.25	0.3	100	5
CH8	412.1696	0.007279	0.3	100	5
LDA4(Plane 1)	101	0.03	0.3	15	5
LDA4(Plane 2)	120	0.25	0.3	85	5
SW4	95.5731	0.25	0.3	100	5
SW3	89.773	0.25	0.3	100	5

Snyder Unit Hydrograph Input Parameters

Subbasin Designation	Lag Time (hr)	Peaking Coefficient
CH1	0.78	0.78
CH2	0.23	0.61
SA1	0.8	0.68
NA1	0.2	0.61
WA1	0.42	0.7
NA2	0.44	0.7
EA1	0.3	0.63

SCS Unit Hydrograph Input Parameters

Subbasin Designation	Graph Type	Lag Time (minute)
Pond1 Area	Standard	0.1

SCS Curve Numbers

Subbasin Designation	Curve Number	Impervious Cover (%)
LDA1	88	0
CH3	88	10
CH1	84	0
CH2	88	0
CH4	88	10
CH5	88	10
CH6	88	10
LDA2	88	0
CH7	88	10
LDA3	88	0
SW1	88	10
SW2	88	10
CH8	88	5
LDA4 (Plane 1)	89	0
LDA4 (Plane 2)	88	0
SW4	88	10
SW3	88	0
Pond1 Area	99	0
SA1	84	0
NA1	84	25
WA1	84	30
NA2	84	2

Pond Routing Information:

Elevation (ft-msl)	Area (ac)
72	4.95
72.5	5.03
74	5.28
76	5.61
78	5.95
79	6.12

HEC-HMS Hardin County Post Development Model Element Inventory

Subbasin Designation	Element Type	Description
LDA1	Subbasin	Letdown 1 Area
CH3	Subbasin	CHANNEL 3 Area
CH1	Subbasin	Channel 1 Area
CH3-CH1	Junction	Channel 1 & Channel 3 Confluence
CH2	Subbasin	Channel 2
CH3-CH2	Junction	Channel 2 & Channel 3 Confluence
CH4	Subbasin	Channel 4 Area
CH4-LDA1	Junction	Channel 4 & Letdown 1 Confluence
CH5	Subbasin	Channel 5 Area
CH5-CH4	Junction	Channel 4 & Channel 5 Confluence
CH6	Subbasin	Channel 6 Area
CH5-CH6	Junction	Channel 6 & Channel 5 Confluence
LDA2	Subbasin	Letdown 2 Area
CH6-LDA2	Junction	Channel 5 Letdown 2 Confluence
CH7	Subbasin	Channel 7 Area
CH7-CH6	Junction	Channel 6 Channel 7 Confluence
LDA3	Subbasin	Letdown 3 Area
SW1	Subbasin	West lower swale
SW2	Subbasin	South swale at the west end
SW1-LDA3-SW2	Junction	Flow combination for 3 areas flowing into channel 8
CH8	Subbasin	Channel 8 Area
CH8-LDA3	Junction	Channel 8 Letdown 3 Confluence
LDA4	Subbasin	Letdown 4 area (south slope)
SW4	Subbasin	South swale (east end)
SW3	Subbasin	Small swale on the south side (middle)
SW3-LDA4-SW4	Junction	Combined flow from letdown 4
Pond1 Area	Subbasin	Estimates direct rainfall onto the pond
Inflow	Junction	Total Flow into the Pond
Pond1	Reservoir	Total Flow Routed thru the Pond
SA1	Subbasin	Area downstream of the pond
SD	Junction	South Discharge
NA1	Subbasin	Sheet flow discharge on the north west
WA1	Subbasin	Sheet flow discharge to west
NA2	Subbasin	Sheet flow discharge on the north east
EA1	Subbasin	Sheet flow discharge on the north east

Global Summary of HEC-HMS Simulation Results

Hydrologic Element	Drainage Area (SqMi)	Peak Discharge (cfm)	Time of Peak	Volume (in)
LDA1	0.0167105	75.8	01Jan2017, 12:10	8.46
CH3	0.006023	26.9	01Jan2017, 12:10	7.99
CH1	0.000348	0.7	01Jan2017, 12:50	8.03
CH3-CH1	0.006371	27.1	01Jan2017, 12:10	7.99
CH2	0.005326	17.4	01Jan2017, 12:20	8.53
CH3-CH2	0.011697	40.9	01Jan2017, 12:10	8.24
CH4	0.003282	15.1	01Jan2017, 12:10	8.21
CH4-LDA1	0.0316895	131.9	01Jan2017, 12:10	8.35
CH5	0.0023	7.6	01Jan2017, 12:10	8.68
CH5-CH4	0.0339895	139.5	01Jan2017, 12:10	8.37
CH6	0.00557	23.2	01Jan2017, 12:10	8.58
CH5-CH6	0.0395595	162.6	01Jan2017, 12:10	8.4
LDA2	0.0136597	60.5	01Jan2017, 12:10	8.45
CH6-LDA2	0.0532192	223.1	01Jan2017, 12:10	8.41
CH7	0.002771	11.7	01Jan2017, 12:10	8.15
CH7-CH6	0.0559902	234.8	01Jan2017, 12:10	8.4
LDA3	0.0179811	77	01Jan2017, 12:10	8.37
SW1	0.0060641	26.9	01Jan2017, 12:10	8.68
SW2	0.001379	6.5	01Jan2017, 12:10	8.74
SW1-LDA3-SW2	0.0254242	110.4	01Jan2017, 12:10	8.47
CH8	0.0006	1.8	01Jan2017, 12:20	8.63
CH8-LDA3	0.0260242	112	01Jan2017, 12:10	8.47
LDA4	0.01233	56.7	01Jan2017, 12:10	8.53
SW4	0.0020553	9.1	01Jan2017, 12:10	8.66
SW3	0.000890173	4.1	01Jan2017, 12:10	8.52
SW3-LDA4-SW4	0.0152755	69.8	01Jan2017, 12:10	8.55
Pond1 Area	0.0099	44.1	01Jan2017, 12:10	9.88
Inflow	0.10719	460.8	01Jan2017, 12:10	8.58
Pond1	0.10719	41.9	01Jan2017, 13:40	7.69
SA1	0.0071	12.7	01Jan2017, 12:50	8.03
SD	0.11429	54	01Jan2017, 13:00	7.71
NA1	0.0019538	6.7	01Jan2017, 12:20	8.52
WA1	0.0019	5.1	01Jan2017, 12:30	8.62
NA2	0.0016232	4.1	01Jan2017, 12:30	8.07
EA1	0.0059	17.2	01Jan2017, 12:20	8.42

VOLUME CALCULATIONS

EXCESS RAINFALL VOLUME CALCULATION

The volume generated by the site and the surrounding properties is calculated for the 25-year storm event. A summary of the design information that is included in this Appendix and related appendices are listed below.

- Excess rainfall and drainage areas used in the volume calculations were taken from the HEC-HMS analysis located in Appendix IIIF-A (post-development).
- Post-development condition volume information is summarized on page IIIF-A-32.

Prep By: NT
Date: 3/31/17

HARDIN COUNTY LANDFILL
0120-758-11-02
EXCESS RAINFALL
VOLUME CALCULATIONS

Chkd By: JRP
Date: 3/31/17

Required: Determine the volume generated by the site and offsite areas using the excess rainfall calculated in the HEC-HMS analysis of the postdevelopment site conditions.

Method:

1. Use the excessive rainfall data generated by the HEC-HMS analysis (see Appendix IIIF-A) to determine the volume produced by the site for the postdevelopment conditions.
2. Use areas and runoff coefficient in the currently permitted draiange analysis (Sheet 34 - Attachment # 6, Final Contour Map) provided in Appendix IIIF-E to summarize the volume of runoff discharged for the currently permitted conditions.

Prep By: NT
Date: 3/31/17

HARDIN COUNTY LANDFILL
0120-758-11-02
EXCESS RAINFALL
VOLUME CALCULATIONS

Chkd By: JRP
Date: 3/31/17

1. Postdevelopment Conditions

1. a. Total Volume of Flow for Areas Discharging to North:

Area No.	Area (sq mi)	Total Excess Rainfall (in)	Area (ac)	Volume (ac-ft)
NA1	0.0020	8.52	1.25	0.9
NA2	0.0016	8.07	1.04	0.7
Total Volume of Discharge at the north permit boundary=				1.6

Discharge from the north permit boundary only occurs as sheet flow.

1. b. Total Volume of Flow for Areas Discharging to East:

Area No.	Area (sq mi)	Total Excess Rainfall (in)	Area (ac)	Volume (ac-ft)
EA1	0.0059	8.42	3.76	2.6

Discharge from the east permit boundary (on to adjacent IESI property) only occurs as a sheet flow.

1. c. Total Volume of Flow for Areas Discharging to South:

Area No.	Area (sq mi)	Total Excess Rainfall (in)	Area (ac)	Volume (ac-ft)
SD	0.1143	7.71	73.15	47.0

South discharge is combination of sheet flow from areas labeled as SA1 and discharge from the pond.

1. d. Total Volume of Flow for Areas Discharging to West

Area No.	Area (sq mi)	Total Excess Rainfall (in)	Area (ac)	Volume (ac-ft)
WA1	0.0019	8.07	1.22	0.8

West discharge consist of sheet flow generated from the west side of the perimeter road.

2. Permitted Conditions

The following areas and labels are reproduced from Sheet 34 - Final Contour Map included in Appendix IIIF-E.

2.a. Total Volume of Flow for Areas Discharging to North:
Discharge number "E" : This discharge represent areas 13, 15, and 16 on Sheet 34.

Discharge Point	SubArea	Area (ac)	Rainfall (in)	Runoof %	Excess Rainfall (in)	Volume (ac-ft)
E	13	1.05	10	0.85	8.50	0.7
	15	1.07	10	0.85	8.50	0.8
	16	0.10	10	0.85	8.50	0.1
	Total Area (ac):	2.22			Total Discharge to North:	1.6

EXCESS RAINFALL
VOLUME CALCULATIONS

2.b. Total Volume of Flow for Areas Discharging to East

Discharge number "A" : This discharge represent areas 2, 3, and 4 on Sheet 34.

Discharge Point	SubArea	Area (ac)	Rainfall (in)	Runoof %	Excess Rainfall (in)	Volume (ac-ft)
A	2	3.62	10	0.85	8.50	2.6
	3	3.00	10	0.85	8.50	2.1
	4	3.72	10	0.85	8.50	2.6
	Total Area (ac):	10.34			Total Discharge to East:	7.3

2.c. Total Volume of Flow for Areas Discharging to South

Discharge number "B, C, D" : This discharge represent areas 5, 6, 7, 8, 1, 12, 11, 10, and 9 on Sheet 34.

Discharge Point	SubArea	Area (ac)	Rainfall (in)	Runoof %	Excess Rainfall (in)	Volume (ac-ft)
B	5	7.26	10	0.85	8.50	5.1
	6	2.77	10	0.85	8.50	2.0
C	7	2.84	10	0.85	8.50	2.0
	8	2.73	10	0.85	8.50	1.9
D	1	4.55	10	0.85	8.50	3.2
	12	3.49	10	0.85	8.50	2.5
	11	4.25	10	0.85	8.50	3.0
	10	7.73	10	0.85	8.50	5.5
	9	2.81	10	0.85	8.50	2.0
	Total Area (ac):	38.43			Total Discharge to East:	27.2

2.d. Total Volume of Flow for Areas Discharging to South

Discharge number "F" : This discharge represent area 14 on Sheet 34.

Discharge Point	SubArea	Area (ac)	Rainfall (in)	Runoof %	Excess Rainfall (in)	Volume (ac-ft)
F	14	0.10	10	0.85	8.50	0.07
					Total Discharge to East:	0.07

VELOCITY CALCULATIONS

Required: 1. Flow Velocity exiting the landfill permit boundary

- Flow for the 25-year storm event was obtained from the HEC-HMS files included in this Attachment and are summarized below

Area Label	Exiting Label	Slope ft/ft	Flow Vel. (fps)
NA1	DCPN1	0.0089	1.60
NA2	DCPN2	0.0064	1.40
EA1	DCPE	0.0073	1.50
WA1	DCPW	0.0017	1.10
Pond 1*	DCPS	0.0149	1.30

* Runoff discarges from the permit boudary as a sheet flow for each direction. The pond outlet description, provided in Appendix IIIF-B, includes discharge velocity calculations for the outlet. The velocity shown is from the permit boundary, which is downstream of the pond. There is no run on to the permit area.

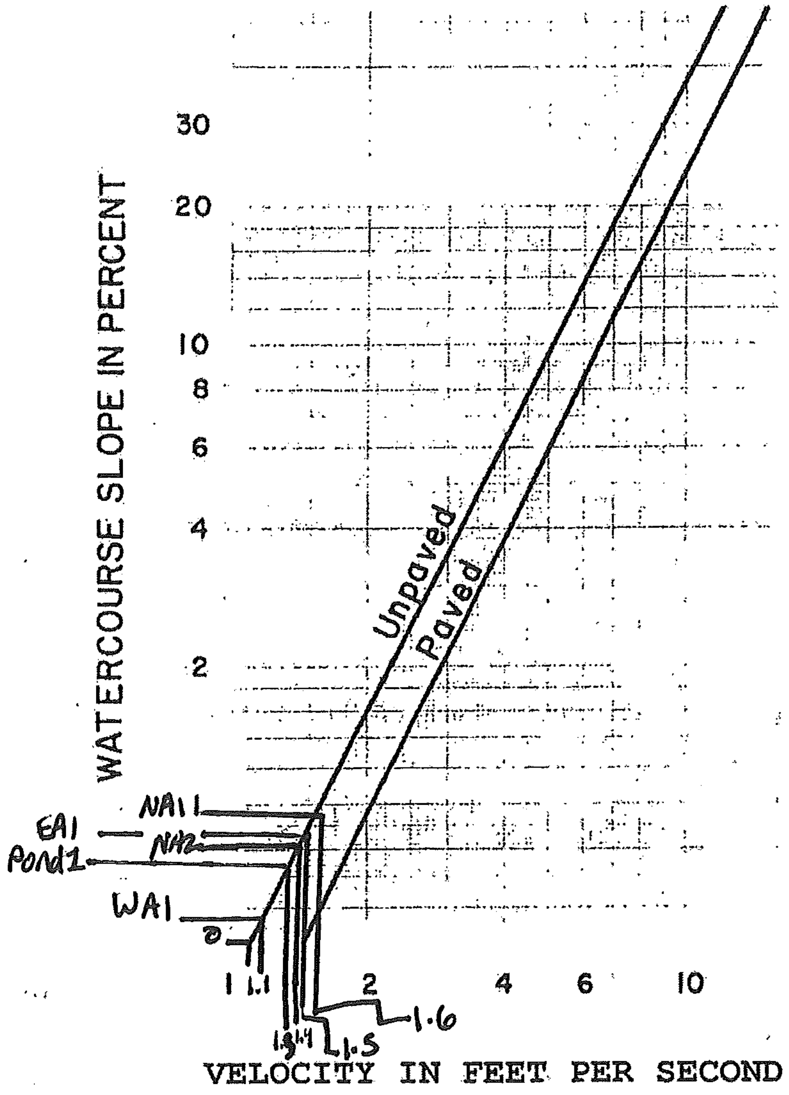


Figure 1. Average Velocities for Estimating Travel Time for Shallow Concentrated Flow.

(NENTC TN,, HYDROLOGY N4, DEC 2003) (5)

RATIONAL METHOD
HEC-HMS COMPARISON

Required: Provide procedure for estimating peak flow rates using the rational method.

Given:

1. Drainage area (See Drawing IIIF-2 for example drainage area LDA4).
2. Coefficients to calculate rainfall intensity for Hardin County.

Note: To calculate a flow rate at a point of interest, the rational method at a minimum the following requirements must be established:

- Total drainage area to the point of interest
- Total overland (sheet) flow and channelized lengths (channelized flow should be determined for each uniform section of the channels.)
- Rainfall intensity must be determined for a given geographic location.
- Run off coefficient

Note that the rational method lacks flow routing. Hydraulic properties of channels based on full flow conditions.

Method:

1. Outline a general procedure to calculate flow rates using the rational method.

2. Compare the peak flows generated using the rational method to peak flows calculated by the HEC-HMS analysis.

References:

1. State of Texas, Department of Transportation, Bridge Division, Hydraulic Manual, 3rd Edition, December 1985.
2. United States Department of Agriculture, Soil Conservation Service, Engineering Division, *TR-55 - Urban Hydrology for Small Watersheds*, 1986.
3. NOAA Hydro-35/TP-40

Summary of Symbols:

P_2 = rainfall depth for a 2-year, 24-hour storm (Ref 3).
L = drainage length for the specified flow type (ft)
S = slope of the drainage path (ft/ft)
n = Manning's roughness coefficient
V = flow velocity (ft/sec)
t_c = time of concentration
$t_{c,tot}$ = cumulative time of concentration for the drainage area
I = storm intensity (in/hr)

Procedure:

The peak flow can be calculated using the following calculation:

$Q = CIA$ where:

Q = peak flow (cfs)
C = runoff coefficient
I = intensity (in/hr)
A = area (ac)

The intensity of the design storm can be calculated using the following:

$$I = \frac{b}{(t_{c,tot} + d)^e}$$

where:

I = intensity (in/hr)
$t_{c,tot}$ = time of concentration (min)

b =	80	From Ref 1, for Hardin County
d =	7.5	25-year storm event
e =	0.720	

Under TCEQ guidelines, $t_{c,tot}$ is a minimum of 10 minutes.

The total time of concentration of a drainage area is the time it takes runoff from the hydraulically most distance part of the drainage area to a point of interest within the drainage area. The total time of concentration is calculated by summing the all the travel times for consecutive components of the drainage conveyance system. (Ref 2)

$$t_{c,tot} = t_{c,1} + t_{c,2} + \ldots t_{c,m} \qquad \text{(Ref 2)}$$

where:

$t_{c,tot}$ = cumulative time of concentration for the drainage area
m = number of flow segments

Typical conveyance within the drainage areas consist of either sheet flow, shallow concentrated flow or open channel flow (i.e., perimeter channels). Sheet flow usually occurs in the headwater of streams. After 300 feet, sheet flow becomes shallow concentrated flow.

The time of concentration for each type of conveyance can be calculated by the following:

sheet flow: $t_c = 0.007(nL)^{0.8} / P_2^{0.5}S^{0.4}$ (Ref 2)
shallow concentrated flow: $t_c = L / 3600V$ (Ref 2)
channel: $t_c = L / 3600V$ (Ref 2)

Example Calculation:

Determine the 25-year peak flow for area LDA4

Total area of this basin: 7.89 acres

The conveyance consists of two components:

First: Landfill Top Deck Area
100 feet of sheet flow on the landfill cover before it enters the longest swale.

Second: Swale as defined below:
500 feet of swale length for the longest swale for area LDA4 on Drawing IIIF.2.

1. Calculate the time of concentration for sheetflow

$$t_{c,1} = 0.007(nL)^{0.8} / P_2^{0.5}S^{0.4}$$

P_2 = 5 inches (Ref 3)
L = 100 ft
S = 0.04 ft/ft
n = 0.35

$t_{c,1}$ = 0.20 hr
11.70 min

2. Calculate the time of concentration for channel

List of Symbols

Q_d = design flow rate for channel, cfs
R = hydraulic radius, ft
n = Manning's roughness coefficient
S = channel slope, ft/ft
b = bottom width of channel, ft
z_r = z-ratio (ratio of run to rise for channel sideslope) for right side slope of swale
z_l = z-ratio (ratio of run to rise for channel sideslope) for left side slope of swale
A_f = flow area, sf
g = gravitational acceleration = 32.2 ft/s^2
T = top width of flow, ft
d = normal depth of swale, ft

2a. Estimating velocity in channel:

These calculations utilize an iterative process to calculate the velocity of the channel to satisfy Manning's Equation

$$Q = \frac{1.486}{n} A R^{0.67} S^{0.5}$$

Design Inputs:

Q_d = 5.0 cfs
S = 0.005 ft/ft
b = 0 ft
z_r = 25 (H) : 1 (V)
z_l = 2 (H) : 1 (V)
n = 0.03

Based on the geometry of the swale cross-section, solve for R and A_f

$$R = \frac{bd + 1/2d^2(z_r + z_l)}{b + d((z_l^2 + 1)^{0.5} + (z_r^2 + 1))}$$

$$A_f = bd + 1/2d^2(z_r + z_l)$$

assume: $d =$ 0.51 ft

$R =$ 0.255 ft

$A_f =$ 3.56 sf

solve for Q: Q = 5.0

if Q is not equal to Q_d, select a new d and repeat calculations

Q = VA => V = Q/A

V = 1.41 ft/s

2b. Calculate Time of Concentration for Channel

$t_{c,2} = L / V$

Length of Flow L = 500 ft

Flow Velocity V = 1.407 ft/s

$t_{c,2} =$ 0.10 hr

$t_{c,2} =$ 5.92 min

3. Total Time of Concentration for drainage basin:

Sheet Flow (hr)=	0.20	11.70 minutes
Channelized Flow (hr)=	0.10	5.92 minutes
Total channelized flow time (hr)=	0.29	17.62 minutes

4. Calculate Peak Flow for 25-year storm event.

Q = C*I*A

Where:

C= 0.85 (runoff coefficient, Ref 1.)

I = intensity, in/hr

A= 7.89 drainage area, ac

$$I = \frac{b}{(t_{c,tot} + d)^e}$$

From Ref 1, for Hardin County for a 25-year storm event

b = 80

d = 7.5

e = 0.720

I = 7.85 in/hr

Q = 53 cfs

5. Compare to Q_d used for channel velocity calculation

Q_d = 5 cfs

if Q is not equal to Q_d, select a new Q_d and repeat calculations

5. Compare to HEC-HMS Analysis

Using rational method:

Q_r = 53 cfs

From the post-development HEC-HMS analysis on page IIIF-A-23.

$Q_{HEC\text{-}HMS}$ = 56.7 cfs

$$Q_{HEC\text{-}HMS} > Q_r$$

APPENDIX IIIF-B

PERIMETER CHANNEL, DETENTION POND, AND POND OUTLET DESIGN

STATE OF TEXAS
NEVZAT TURAN
84059
LICENSED
PROFESSIONAL ENGINEER

12-5-2017

Includes pages IIIF-B-1 through IIIF-B-11

CONTENTS

NEVZAT TURAN
84059

12-5-2017

PERIMETER CHANNEL DESIGN

Perimeter channels have been designed to contain stormwater runoff from the 25-year storm frequency. A summary of the design information that is included in this Appendix is listed below.

- Flow rates used for the perimeter channel design were taken from the HEC-HMS analysis included in Appendix IIIF-A.
- Hydraulic calculations are summarized on pages IIIF-B-2.
- Perimeter channel design system information is summarized on Sheet IIIF-B-3.
- Perimeter Channel Erosion Control Design information is included on page IIIF-B-5.

Prep By: NT
Date: 03/31/17

Chkd By: JRP
Date: 03/31/17

HARDIN COUNTY LANDFILL
0120-758-11-02

PERIMETER CHANNEL HYDRAULIC ANALYSIS

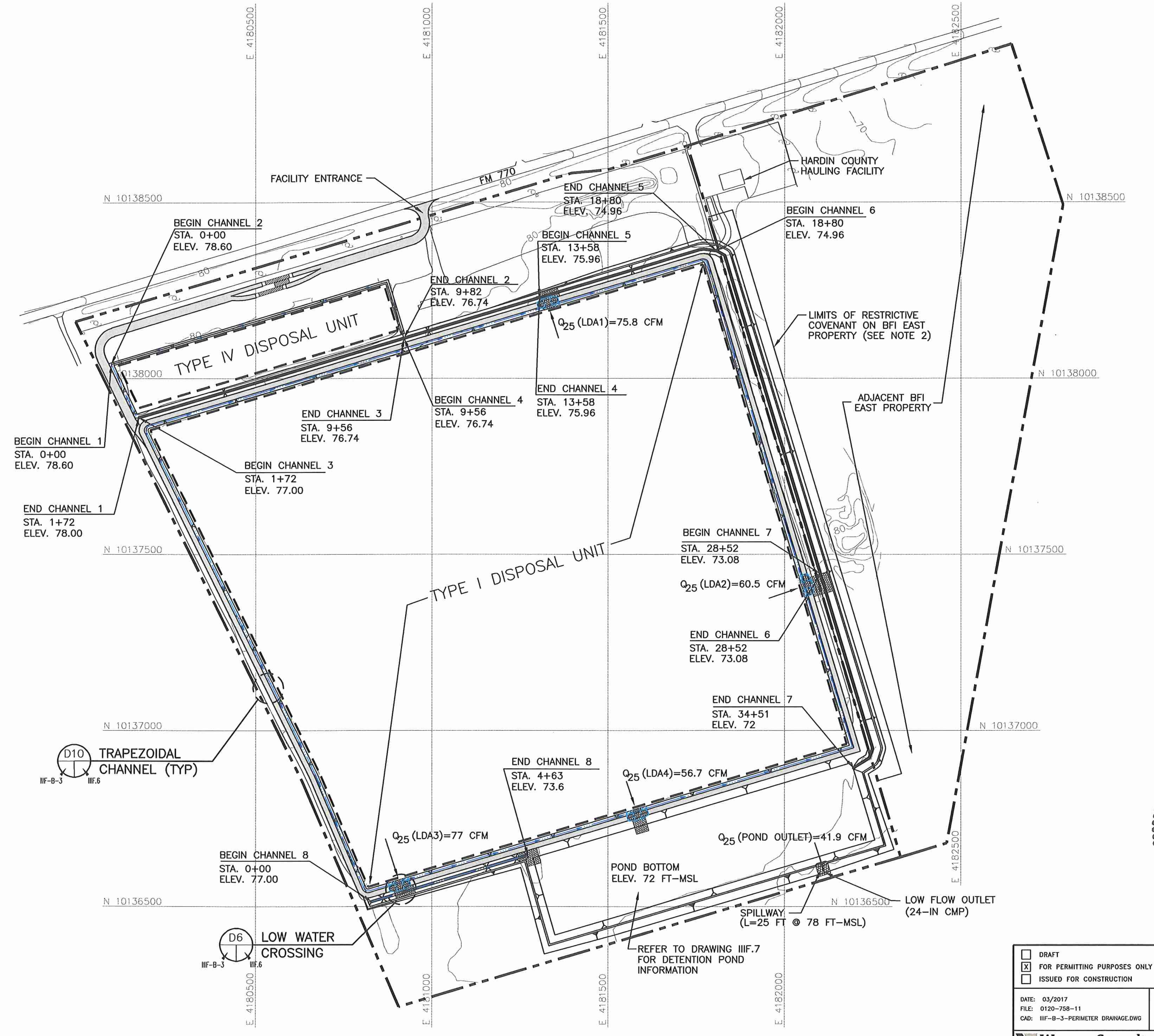
INPUT PARAMETERS AND VALUES

Channel^2	Station^2		Length	Elevations (ft-msl)		Flow Rate^3	Bottom	Bottom	Left Side	Right Side	Manning's
	From	To	(ft)	From	To	(cfs)	Slope (ft/ft)	Width (ft)	Slope (ft/ft)	Slope (ft/ft)	n-Value
1	0+00.00	1+72.00	172	78.60	78.00	0.7	0.0035	2	3	3	0.03
2	0+00	9+82.00	982	78.60	76.74	17.4	0.0019	4	3	3	0.03
3	1+72.00	9+56.00	784	78.00	76.74	27.1	0.0016	4	3	3	0.03
4	9+56.00	13+58.00	402	76.74	75.96	139.5	0.0019	6	3	3	0.03
5	13+58.00	18+80.00	522	75.96	74.96	162.6	0.0019	6	3	3	0.03
6	18+80.00	28+52.00	972	74.96	73.08	223.1	0.0019	6	3	3	0.03
7	28+52.00	34+51.00	599	73.08	72.00	234.8	0.0018	6	3	3	0.03
8	0+00.00	4+63.00	463	77.00	73.60	112.0	0.0073	6	3	3	0.03

RESULTS

Channel^2	Normal Depth (ft)	Flow Vel. (fps)	Froude No.	Vel. Head (ft)	Energy Head (ft)	Flow Area^1 (sq.ft.)	Top width of Flow^1 (ft)
1	0.26	0.99	0.390	0.02	0.27	0.71	3.53
2	1.22	1.86	0.360	0.05	1.28	9.37	11.33
3	1.58	1.96	0.342	0.06	1.64	13.81	13.48
4	2.96	3.17	0.411	0.16	3.11	43.98	23.74
5	3.17	3.30	0.415	0.17	3.34	49.27	25.04
6	3.67	3.58	0.423	0.2	3.87	62.32	28.00
7	3.80	3.55	0.414	0.2	3.99	66.05	28.79
8	1.93	4.92	0.761	0.38	2.31	22.79	17.59

Note:
1) Calculations were performed using the HYDROCALC HYDRAULIC FOR WINDOWS Computer Program developed by Dodson and Associates (Version 1.2a, 1996).
2) Refer to Drawing IIIF.3 for channel locations and Drawing IIIF-4 for Channel Profiles.
3) Flow rates shown are the peak flow rates obtained from the HEC-HMS model. See HEC-HMS Output-Postdevelopment Conditions in Appendix IIIF-A.



N

0 150 300
SCALE IN FEET

LEGEND

— · — · —	BFI EAST PROPERTY BOUNDARY
—— · · ——	PERMIT BOUNDARY
— — — —	CURRENTLY PERMITTED LIMITS OF WASTE
N 10137500	STATE PLANE COORDINATE GRID
—70—	EXISTING CONTOUR
STA.	CHANNEL STATION (FT)
ELEV.	ELEVATION (FT-MSL)
Q_{25}(LDA2)	25-YR FREQUENCY PEAK FLOW RATE (CUBIC FEET PER SECOND)

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS DEVELOPED BY WEAVER CONSULTANTS GROUP FROM AERIAL PHOTOGRAPHY FLOWN 05-17-2016. GRID SYSTEM IS TIED TO THE TEXAS STATE PLANE COORDINATE SYSTEM NAD 83, CENTRAL ZONE.
2. CHANNEL PROFILE INFORMATION IS PROVIDED ON DRAWING IIIF.4.

NEVZAT TURAN
84059

12-5-2017

☐ DRAFT
☒ FOR PERMITTING PURPOSES ONLY
☐ ISSUED FOR CONSTRUCTION

PREPARED FOR
BFI WASTE SYSTEMS OF NORTH AMERICA, LLC

MAJOR PERMIT AMENDMENT PERIMETER DRAINAGE PLAN

DATE: 03/2017
FILE: 0120-758-11
CAD: IIIF-B-3-PERIMETER DRAINAGE.DWG

DRAWN BY: SRF
DESIGN BY: AE
REVIEWED BY: NT

REVISIONS

NO.	DATE	DESCRIPTION
1	08/2017	FIRST NOD RESPONSE
2	11/2017	OWNERSHIP CHANGE

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS

WWW.WCGRP.COM

SHEET IIIF-B-3

O:\0120\758\2214B EXPANSION\IIIF\IIIF-B-3-PERIMETER DRAINAGE PLAN.dwg, 11/15/2017 11:04:15 AM, rsellers, 1:2

CHANNEL EROSION CONTROL DESIGN

Channel erosion controls have been designed for flow velocities resulted from the 25-year frequency flow rates. As shown on page IIIF-B-2 velocities in the perimeter channels range from 0.99 ft/s to 4.92 ft/s. The channel lining other than vegetation will not be required. Channel drainage letdown confluences are designed for erosion protection. Refer to Drawing IIIF.6 for drainage letdown confluence detail and typical trapezoidal channel section.

DETENTION POND DESIGN

Detention pond has been analyzed by using HEC-HMS, storage routing method. The input parameters for the model are presented in Appendix IIIF-A. A summary of HEC-HMS results are presented on page IIIF-B-6. As it is shown in the table, the detention pond does not have flow over the spillway for the design storm event.

Downstream sides of the low-level outlets will be designed with either rock riprap or gabions.

HARDIN COUNTY LANDFILL
0120-758-11-02
DETENTION POND DESIGN

Purpose: Demonstrate that the detention pond outlet structure designs are adequate to convey runoff from the various subbasins to their discharge points.

Method:

1. Use the 25-year, 24-hour flow rates and water surface elevations for the drainage areas that will discharge to each detention pond from the HEC-HMS analysis (see Appendix IIIF-A).
2. Use the Weir Equation to calculate the flow rate over the spillways as appropriate.

Solution:

	P1
Bottom ELEV, ft[1]	72.0
Spillway ELEV, ft	78.0
Spillway Length, ft	25
Top of Road/Berm, ft	80.0
Discharge Pipe Downstream Invert ELEV, ft	71.80
Peak Inflow Q_{25}, cfs	460.8
Peak Outflow Q_{25}, cfs	41.9
Peak Stage in Pond Q_{25}, ft	77.3
Est. Flow (Q_{25}) over Spillway, cfs	--
Velocity (Q_{25}) over Spillway, fps	--

Note: 1) Details of the pond outlet structures are presented on Drawings IIIF.7. As shown, erosion protection is provided downstream of the pond outlet structure.

2) No flow occurs over the spillway during 25-year frequency storm event.

POND OUTLET DESIGN

The pond out consists of a 24-inch-diameter corrugated metal pipe (cmp) low-flow outlet and a 25-foot-long spillway. The pond routing performed using HEC-HMS (refer to Appendix IIIF-A) indicates that no flow will occur during the 25-year, 24-hour design storm. The pond plan and details are shown on Drawing IIIF.7. The following pages include the pond outlet hydraulic design and erosion protection design.

Prep by: NT
Date: 03/31/17

Chkd By: JRP
Date: 03/31/17

HARDIN COUNTY LANDFILL
0120-758-11-02
CULVERT DESIGN

Require Design pond low flow outlet to convey the flow.

Method: Use HYDROCALC Hydraulics for Windows computer program to determine number and size of the culverts.
Use total 25-year frequency storm event flow estimated by HEC-HMS included in Appendix IIIF-A.

For proposed 24" CMP culvert at the bottom of Pond 1.

Total Flow=	41.9	cfs	calculated using HEC-HMS.
No. of Culverts=	1		
Culvert Diameter=	24	inches	

Culvert ID	FHWA Chart Number	FHWA Scale Number	Culvert Diameter (ft)	Manning's Coefficient	Entrance Loss Coefficient	Culvert Length (ft)	Downstream Invert Elevation (ft msl)	Upstream Invert Elevation (ft msl)	Flow Rate (cfs)	Tailwater Depth^2 (ft)	Normal Depth (ft)	Critical Depth (ft)	Depth at Outlet (ft)	Outlet Velocity (fps)
1	2	2	2	0.024	0.8	67.0	71.8	72.0	41.9	0.90	2.00	1.96	1.96	13.40

1. Calculations were performed using the HYDROCALC Hydraulics for Windows program developed by Dodson and Associates (Version 1.2a, 1996).
2. Initial tailwater depth is assumed to be zero at the culvert outlet to prevent underestimation of flow velocity.

Required: Determine the minimum length and median diameter of riprap required at the detention pond outlet structure to control erosion.

Reference:

1. Haan, Barfield, and Hayes, *Design Hydrology and Sedimentology for Small Catchments*, 1994.
2. Dodson's and Associates, Inc., *ProHec-1 Plus Program Documentation,* 1995.
3. Freeman, Gary E., J. Craig Fischenich, Gabion for Streambank Erosion Control, 2000. EMRRP Technical Notes Collection (ERDC TN-EMRRP-SR-22), U.S. Army Engineer Research and Development Center, Vicksburg, MS.

Solution: The riprap will be designed for the 25-year flow rate at the detention pond outlet structure.

2. Flow through the Low Water Outlet

The flow rate through the low water outlet (LWO) is summarized below.

Flow Structure	Pond Bottom Ele (ft-msl)	LWO Invert Elev. Upstream (ft-msl)	Downstream (ft-msl)	LWO Diamete (in)	25-Year Flow Rate[2] (cfs)	25-Year Outlet Velocity[1] (ft/s)
Pond 1	72.00	72.00	71.80	1-24	41.9	13.40

[1] Velocity through the low water outlet for P1 were calculated using the HYDROCALC HYDRAULICS FOR WINDOWS program developed by Dodson and Associates (Version 1.2a, 1996).

[2] The flowrates for all low water outlets are the peak discharges for the respective areas as calculated by HEC-HMS since the spillway is not reached in the 25-year event.

The flowrate through the low water outlet is used to design the riprap apron.

The nomograph used for design of the length of the riprap and the median diameter are shown on page IIIF-B-11 (Figure 5.25).

The minimum riprap length and diameter for each outlet is summarized below. The length of the riprap is increased by 20 percent to provide for a conservative design.

Pond	Riprap Design Flowrate (cfs)	Pipe Diameter (in)	Riprap Length (ft)	Adjusted Length L x 1.2 (ft)	Median Rock Diameter (ft)
Pond 1	41.9	24	48	58	2.25

Apron width required for the ponds (e.g., width of erosion protection in outlet channel) are:
W_{req}=LWO diameter + 0.4*(RipRap Length)

Pond	W_{req} (ft)	$W_{provided}$ (ft)
Pond 1	21.2	142.0

The riprap will be provided over the entire width of the spillway of ponds P2 and P3 and the outlet channel downstream of all the spillways/low water outlets.

The median diameter of riprap is intended to determine the minimum diameter of the riprap that will be used. As an alternative, 2-foot thick gabions with a d_{50} of 6-inches can be used.

APPENDIX IIIF-C

FINAL COVER EROSION CONTROL STRUCTURE DESIGN

STATE OF TEXAS
NEVZAT TURAN
84059
LICENSED
PROFESSIONAL ENGINEER

12-5-2017

Includes pages IIIF-C-1 through IIIF-C-12

CONTENTS

STATE OF TEXAS
NEVZAT TURAN
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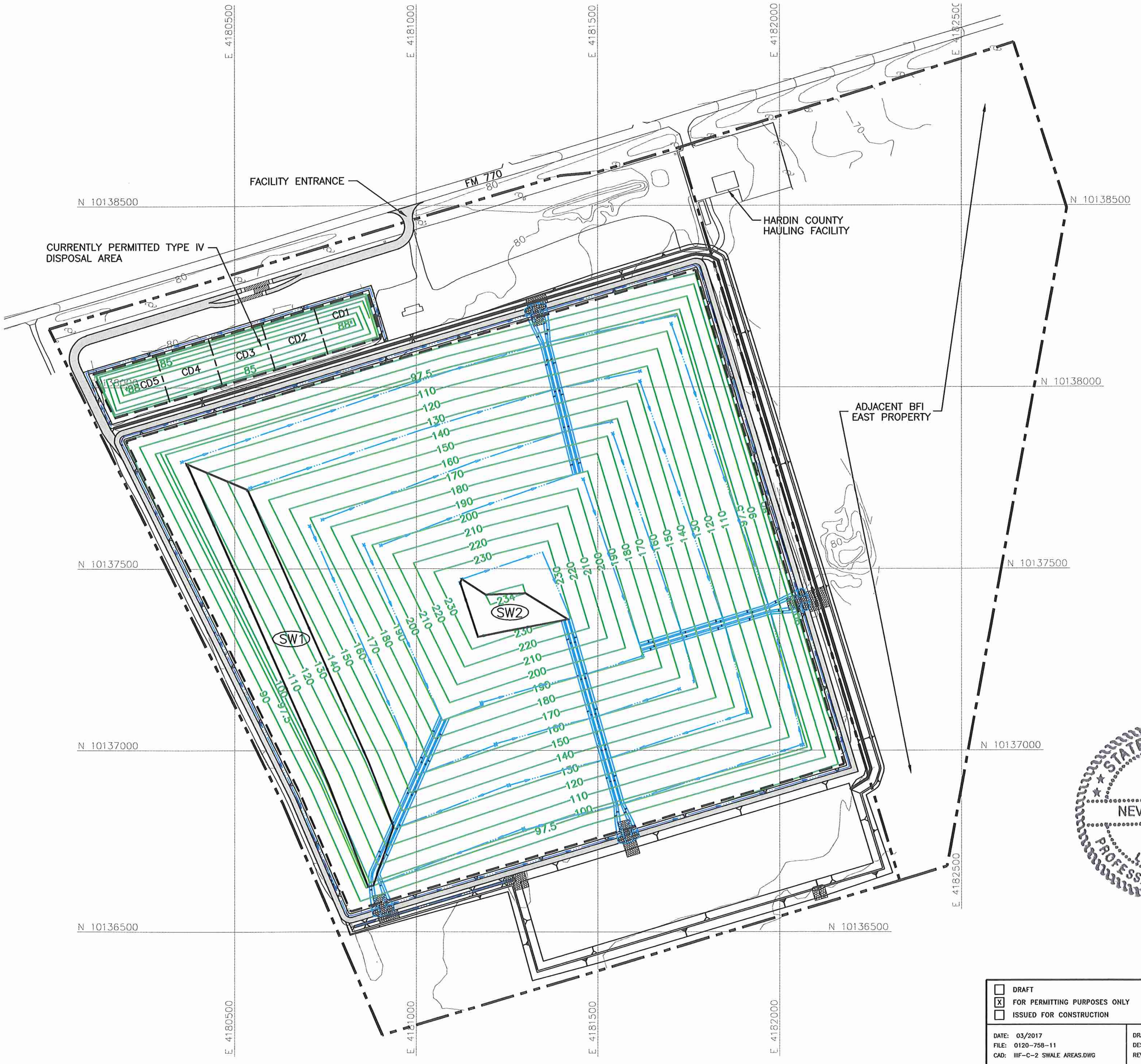
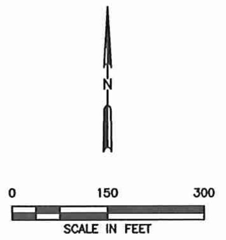
12-5-2017

DRAINAGE SWALE DESIGN

- The drainage swale layout is shown on Drawing IIIF-C-2 – Drainage Structure Plan. Typical details of the swale and swale/chute confluence are provided on Drawing IIIF.5.
- Typical Swale Design Summary:
  - Typical swale drainage areas analyzed are shown on sheet IIIF-C-2.
  - Hydraulic calculations are summarized on page IIIF-C-3.
  - Maximum normal depth is 1.62 feet (Drainage Area SW1).
  - Maximum flow velocity is 2.94 fps (Drainage Area SW1).
  - Vegetation will be established on the swales to protect against erosion.
  - Typical swale drainage areas were selected such that all slope conditions (3% and 25%) are included in this analysis. Additionally, swales with large individual drainage areas and short and long swale lengths are included in this analysis.





LEGEND

	EAST PROPERTY BOUNDARY
	PERMIT BOUNDARY
	CURRENTLY PERMITTED LIMITS OF WASTE
	CELL BOUNDARY
N 10137500	STATE PLANE COORDINATE GRID
70	EXISTING CONTOUR
234	PROPOSED FINAL CONTOUR (SEE NOTE 3)
	PROPOSED DRAINAGE SWALE
	PROPOSED DRAINAGE CHUTE
	SWALE DRAINAGE AREA BOUNDARY
SW1	SWALE AREA DESIGNATION

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS DEVELOPED BY WEAVER CONSULTANTS GROUP FROM AERIAL PHOTOGRAPHY FLOWN 05-17-2016. GRID SYSTEM IS TIED TO THE TEXAS STATE PLANE COORDINATE SYSTEM NAD 83, CENTRAL ZONE.
2. THE LARGEST SIDESLOPE SWALE IS 3.27 ACRES (SW1); THEREFORE THE SIDESLOPE SWALE DESIGN IS SHOWN FOR THIS SWALE AND FOR SW2 WHICH IS THE LARGEST TOPDECK SWALE AREA.

DRAINAGE AREA NO.	AREA (ACRES)
SW1	3.27
SW2	0.52

NEVZAT TURAN
84059

12-5-2017

☐ DRAFT
☒ FOR PERMITTING PURPOSES ONLY
☐ ISSUED FOR CONSTRUCTION

DATE: 03/2017
FILE: 0120-758-11
CAD: IIIF-C-2 SWALE AREAS.DWG

DRAWN BY: SRF
DESIGN BY: AE
REVIEWED BY: NT

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

PREPARED FOR
BFI WASTE SYSTEMS OF NORTH AMERICA, LLC

REVISIONS

NO.	DATE	DESCRIPTION
1	11/2017	OWNERSHIP CHANGE

MAJOR PERMIT AMENDMENT SWALE DRAINAGE AREAS

HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS

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DRAWING IIIF-C-2

O:\0120\758\2214B EXPANSION\IIIF\IIIF-C-2-SWALE AREAS-FINAL.dwg, 11/15/2017 11:06:47 AM, rsellers, 1:2

Required: Analyze swales to determine the adequacy of the swale design.

Method:

1. Determine the 25-year, 24-hour flow rates for the swale drainage areas by the Rational Method.

Reference:

1. State of Texas, Department of Transportation, Bridge Division, Hydraulic Manual, 3rd Edition, December 1985.

Solution:

1. Determine the 25-year intensity flow rates.

$$Q = CIA$$

Where:
C= 0.7 (runoff coefficient, Ref 1.)
I = intensity in/hr
A= drainage area, ac

$$I = \frac{b}{(t_c + d)^e}$$

b = 80 From Ref 1, for Hardin County
d = 7.5 25-year storm event
e = 0.720
t_c is assumed to be = 10 min

I = 10.2 in/hr

Swale	Area (ac)	Flow Rate (cfs)
SW1	3.27	23.3
SW2	0.52	3.7

Prep By: NT
Date: 3-31-17]

HARDIN COUNTY LANDFILL
0120-758-11-02
SWALE ANALYSIS

Chkd By: JRP
Date: 3-31-17

Swale	Flow Rate (cfs)	Bottom Slope (ft/ft)	n-value	Side Slope (left)	Side Slope (right)	Bottom Width (ft)	Normal Depth (ft)	Flow Vel. (fps)	Froude No.	Velocity Head (ft)	Energy Head (ft)	Flow Area (sq. ft.)	Top Width of Flow (ft)
SW1	23.3	0.005	0.03	2	4	0	1.62	2.94	0.574	0.13	1.76	7.93	9.76
SW2	3.7	0.005	0.03	2	33	0	0.41	1.23	0.475	0.02	0.44	3.01	14.52

Note: Calculations were performed using the HYDROCALC HYDRAULICS program developed by Dodson and Associates (Version 1.2a, 1996).
Minimum slope of 0.5% used for all swale calculations.

Maximum flow depth is 1.62 ft < 2.0 ft (swale height).

Design is okay.

Example Calculation: Calculate the normal depth for the swale for the maximum size side slope drainage area.

List of Symbols

Q_d = design flow rate for channel, cfs
R = hydraulic radius, ft
n = Manning's roughness coefficient
S = channel slope, ft/ft
b = bottom width of channel, ft
z_r = z-ratio (ratio of run to rise for channel sideslope) for right side slope of swale
z_l = z-ratio (ratio of run to rise for channel sideslope) for left side slope of swale
A_f = flow area, sf
g = gravitational acceleration = 32.2 ft/s^2
T = top width of flow, ft
d = normal depth of swale, ft

The program uses an iterative process to calculate the normal depth of the swale to satisfy Manning's Equation

$$Q = \frac{1.486}{n} A\,R^{0.67}\,S^{0.5}$$

Design Inputs:

Q_d = 23.3 cfs
S = 0.005 ft/ft
b = 0.01 ft
z_r = 4 (H) : 1 (V)
z_l = 2 (H) : 1 (V)
n = 0.03

Step 1 - Based on the geometry of the swale cross-section, solve for R and A_f

$$R = \frac{bd + 1/2d^2(z_r + z_l)}{b + d((z_l^2 + 1)^{0.5} + (z_r^2 + 1)^{0.5})}$$

$$A_f = bd + 1/2d^2(z_r + z_l)$$

assume: d = 1.62 ft

R = 0.77 ft

A_f = 7.94 sf

solve for Q: $Q = 23.3$

if Q is not equal to Q_d, select a new d and repeat calculations

Step 2 - solve for velocity, T, Froude number, velocity head, and energy head

$Q = VA \Rightarrow \quad V = Q/A$

$V = 2.94$ ft/s

$T = b + d(z_l + z_r)$

$T = 9.76$ ft

$$F_r = \frac{V}{(gA/T)^{0.5}}$$

$F_r = 0.574$

$$\text{Velocity Head} = \frac{V^2}{2g}$$

Velocity Head = 0.13 ft

Energy Head = water elevation + velocity head

Energy Head = 1.76 ft

Rainfall Intensity-Duration-Freq…cy Coefficients for Texas Counties

ENGLISH   UNITS: e is in milimeters, b is unitless, and d is in minutes

County name	2-year			5-year			10-year			25-year			50-year			100-year		
	e	b	d	e	b	d	e	b	d	e	b	d	e	b	d	e	b	d
Dawson	0.820	44	10.0	0.818	59	10.4	0.814	68	10.4	0.807	77	10.4	0.801	86	10.4	0.798	93	10.0
De Witt	0.810	65	8.9	0.785	75	8.7	0.758	78	8.7	0.756	90	8.7	0.747	96	8.7	0.739	102	8.9
Deaf Smith	0.847	48	10.7	0.844	60	9.3	0.794	61	9.3	0.831	82	9.3	0.837	94	9.3	0.830	107	10.7
Delta	0.788	54	8.2	0.783	68	9.1	0.775	77	9.1	0.770	89	9.1	0.759	94	9.1	0.753	100	8.2
Denton	0.789	51	8.0	0.777	65	8.5	0.779	77	8.5	0.781	90	8.5	0.780	102	8.5	0.769	107	8.0
Dickens	0.810	46	9.4	0.807	61	10.0	0.803	71	10.0	0.808	85	10.0	0.808	96	10.0	0.809	109	9.4
Dimmit	0.830	60	9.6	0.806	74	9.4	0.795	82	9.4	0.783	94	9.4	0.781	105	9.4	0.779	113	9.6
Donley	0.839	52	11.0	0.832	68	10.6	0.825	79	10.6	0.836	96	10.6	0.823	103	10.6	0.845	129	11.0
Duval	0.826	70	8.8	0.802	79	9.2	0.781	84	9.2	0.772	94	9.2	0.755	98	9.2	0.750	104	8.6
Eastland	0.780	45	8.0	0.772	58	7.8	0.771	69	7.8	0.772	81	7.8	0.775	92	7.8	0.770	101	8.0
Ector	0.812	39	9.5	0.821	56	10.5	0.816	65	10.5	0.789	68	10.5	0.802	82	10.5	0.800	89	9.5
Edwards	0.790	44	8.2	0.759	54	7.5	0.759	63	7.5	0.769	76	7.5	0.776	88	7.5	0.772	100	8.2
El Paso	0.797	24	9.5	0.802	34	12.0	0.795	42	12.0	0.843	60	12.0	0.900	90	12.0	0.825	65	9.5
Ellis	0.798	56	8.4	0.788	71	8.8	0.777	79	8.8	0.771	91	8.8	0.766	98	8.8	0.760	105	8.4
Erath	0.780	47	7.7	0.772	61	8.1	0.770	72	8.1	0.765	89	8.1	0.772	96	8.1	0.765	103	7.7
Falls	0.801	60	8.0	0.786	72	8.5	0.771	79	8.5	0.772	93	8.5	0.750	94	8.5	0.748	102	8.0
Fannin	0.790	54	8.3	0.778	66	9.1	0.782	79	9.1	0.762	93	9.1	0.779	99	9.1	0.762	103	8.3
Fayette	0.805	65	8.3	0.782	73	8.2	0.758	76	8.2	0.758	88	8.2	0.743	94	8.2	0.740	100	8.3
Fisher	0.786	41	8.6	0.779	55	9.5	0.793	70	9.5	0.790	83	9.5	0.798	96	9.5	0.788	102	8.6
Floyd	0.829	48	10.1	0.821	63	10.0	0.890	71	10.0	0.818	85	10.0	0.813	97	10.0	0.822	110	10.1
Foard	0.810	48	9.8	0.787	60	9.5	0.806	77	9.5	0.807	91	9.5	0.817	109	9.5	0.808	116	9.8
Fort Bend	0.804	70	7.9	0.760	71	8.1	0.751	80	8.1	0.729	84	8.1	0.726	91	8.1	0.710	92	7.9
Franklin	0.780	53	8.1	0.782	69	8.8	0.765	74	8.8	0.759	84	8.8	0.751	89	8.8	0.745	95	8.1
Freestone	0.803	62	8.5	0.795	77	9.0	0.769	80	9.0	0.775	95	9.0	0.749	94	9.0	0.745	100	8.5
Frio	0.814	61	9.3	0.789	71	9.1	0.769	82	9.1	0.772	89	9.1	0.765	96	9.1	0.769	110	9.3
Gaines	0.820	42	10.0	0.832	58	10.0	0.805	63	10.0	0.797	70	10.0	0.807	84	10.0	0.813	95	10.0
Galveston	0.787	67	7.8	0.739	66	7.6	0.742	78	7.6	0.727	85	7.6	0.704	88	7.6	0.690	85	7.8
Garza	0.812	44	9.4	0.811	60	10.2	0.800	67	10.2	0.810	83	10.2	0.799	88	10.2	0.800	100	9.4
Gillespie	0.787	49	8.5	0.766	60	8.1	0.767	71	8.1	0.765	82	8.1	0.764	94	8.1	0.765	104	8.5
Glasscock	0.801	40	9.1	0.796	55	10.0	0.803	66	10.0	0.789	74	10.0	0.789	83	10.0	0.784	92	9.1
Goliad	0.812	65	9.1	0.789	75	8.7	0.758	77	8.7	0.755	89	8.7	0.746	97	8.7	0.738	101	9.1
Gonzales	0.801	61	8.4	0.788	74	8.6	0.763	77	8.6	0.760	89	8.6	0.747	95	8.6	0.745	104	8.4
Gray	0.845	54	10.8	0.837	70	10.8	0.836	83	10.8	0.842	99	10.8	0.841	114	10.8	0.846	125	10.8
Grayson	0.790	52	8.3	0.778	65	8.9	0.779	78	8.9	0.790	95	8.9	0.781	104	8.9	0.769	108	8.3
Gregg	0.783	56	8.1	0.783	71	8.6	0.750	72	8.6	0.753	84	8.6	0.740	87	8.6	0.737	94	8.1
Grimes	0.808	68	8.0	0.784	75	8.3	0.760	81	8.3	0.744	87	8.3	0.742	95	8.3	0.721	94	8.0
Guadalupe	0.796	58	8.4	0.787	72	8.7	0.772	78	8.7	0.765	89	8.7	0.750	93	8.7	0.754	105	8.4
Hale	0.834	48	10.3	0.827	61	9.9	0.815	69	9.9	0.823	84	9.9	0.812	92	9.9	0.817	104	10.3
Hall	0.829	50	10.4	0.821	66	10.3	0.815	75	10.3	0.822	92	10.3	0.819	103	10.3	0.830	126	10.4
Hamilton	0.779	48	7.3	0.770	62	8.3	0.761	72	8.3	0.778	89	8.3	0.766	95	8.3	0.761	103	7.3
Hansford	0.865	57	10.4	0.846	73	11.3	0.842	84	11.3	0.862	104	11.3	0.867	124	11.3	0.839	116	10.4
Hardeman	0.815	49	10.0	0.794	61	9.5	0.810	78	9.5	0.816	95	9.5	0.817	110	9.5	0.810	120	10.0
Hardin	0.788	68	8.4	0.738	65	7.5	0.740	74	7.5	0.720	80	7.5	0.718	87	7.5	0.700	87	8.4
Harris	0.800	68	7.9	0.749	70	7.7	0.753	81	7.7	0.724	81	7.7	0.728	91	7.7	0.706	91	7.9
Harrison	0.771	63	8.0	0.773	69	8.4	0.750	70	8.4	0.747	80	8.4	0.730	82	8.4	0.732	90	8.0
Hartley	0.858	51	10.8	0.855	67	10.2	0.814	67	10.2	0.840	85	10.2	0.863	106	10.2	0.832	107	10.8
Haskell	0.790	45	8.9	0.779	57	9.2	0.799	74	9.2	0.787	85	9.2	0.805	103	9.2	0.788	106	8.9
Hays	0.796	56	8.2	0.783	69	8.6	0.776	78	8.6	0.765	87	8.6	0.747	90	8.6	0.755	104	8.2
Hemphill	0.845	56	10.6	0.851	76	10.7	0.842	87	10.7	0.843	103	10.7	0.840	115	10.7	0.847	132	10.6
Henderson	0.800	60	8.7	0.796	77	9.0	0.770	79	9.0	0.773	93	9.0	0.752	93	9.0	0.749	102	8.7
Hidalgo	0.831	74	9.6	0.795	80	9.2	0.778	87	9.2	0.771	88	9.2	0.749	99	9.2	0.740	103	9.6
Hill	0.799	57	8.2	0.780	72	8.8	0.777	78	8.8	0.773	91	8.8	0.764	98	8.8	0.759	105	8.2
Hockley	0.832	46	9.9	0.832	60	10.0	0.807	64	10.0	0.812	78	10.0	0.810	87	10.0	0.817	101	9.9
Hood	0.782	48	7.7	0.773	63	8.3	0.773	75	8.3	0.782	90	8.3	0.773	98	8.3	0.766	105	7.7
Hopkins	0.788	54	8.3	0.783	70	9.1	0.775	77	9.1	0.767	88	9.1	0.754	93	9.1	0.750	100	8.3
Houston	0.797	63	8.2	0.780	73	8.3	0.757	78	8.3	0.746	86	8.3	0.740	93	8.3	0.727	94	8.2
Howard	0.805	42	9.2	0.800	56	10.1	0.802	65	10.1	0.796	76	10.1	0.791	86	10.1	0.788	95	9.2
Hudspeth	0.800	27	9.4	0.840	44	11.4	0.827	50	11.4	0.819	60	11.4	0.856	78	11.4	0.833	77	9.4
Hunt	0.793	55	8.4	0.785	69	9.2	0.783	80	9.2	0.776	93	9.2	0.764	99	9.2	0.758	104	8.4
Hutchinson	0.860	56	10.4	0.844	70	11.0	0.837	80	11.0	0.851	100	11.0	0.863	121	11.0	0.837	115	10.4

DRAINAGE LETDOWN (OR CHUTE) DESIGN

Chute Design

The letdown structures are designed using gabions or flexible geomembrane as a liner. Bedding for the gabions will be prepared subgrade soil overlain by 8 oz/sy geotextile (refer to Drawing IIIF.6). The gabions are placed along the entire chute to protect the chute bottom and the final cover from erosion due to potential erosive velocities. Tumbling flow concrete energy dissipators will be placed at the bottom end of the letdown structure to dissipate excess energy present in the water as it travels down the two and three percent slopes in the low-water crossings over the perimeter road. Typical details of chutes, swale/chute confluence, and chute/perimeter channel confluence are provided on Drawings IIIF.5 and IIIF.6.

The following design information is included in this Appendix:

- Flow rates used in the chutes are presented in Appendix IIIF-A – HEC-HMS computer program output file.
- Hydraulic calculations are summarized on page IIIF-C-9.
- Chute layouts and drainage areas are shown on Sheet IIIF-C-10.
- Additional stormwater details are included on Drawings IIIF.5 and IIIF.6.
- Letdowns on the landfill sideslope (25%) will use 6-inch-thick gabions. Low water crossing and perimeter channel sideslopes portions of letdowns will use 18-inch-thick gabions due to vehicle traffic or higher flow velocities in these locations. The maximum 25-year flow velocity on 25 percent sideslopes is 14.32 fps, which is less the maximum recommended velocity for gabions.

HARDIN COUNTY LANDFILL
0120-758-11-02
CHUTE ANALYSIS
NORMAL DEPTH CALCULATIONS FOR
GABION-LINED CHUTES

Chute flow design for the gabion-lined chutes.

Drainage Area	Acreage	25-Yr Peak Flow Rate (cfs)	Bottom Slope (ft/ft)	Manning's n	Side Slope (lieft)	Side Slope (right)	Bottom Width (ft)	Normal Depth (ft)	Flow Vel. (fps)	Froude Number	Velocity Head (ft)	Energy Head (ft)	Flow Area (sf)	Flow Top Width (ft)
LDA1	10.70	75.8	0.25	0.033	2.0	2.0	8.0	0.58	14.25	3.499	3.15	3.74	5.32	10.32
LDA2	8.76	60.5	0.25	0.033	2.0	2.0	8.0	0.51	13.18	3.437	2.70	3.21	4.59	10.04
LDA3	11.51	77.0	0.25	0.033	2.0	2.0	8.0	0.59	14.32	3.503	3.19	3.77	5.38	10.34
LDA4	7.89	56.7	0.25	0.033	2.0	2.0	8.0	0.49	12.89	3.420	2.58	3.07	4.40	9.96

Note: Calculations were performed using the HYDROCALC HYDRAULICS for Windows program developed by Dodson and Associates (Version 1.2a, 1996).

1) The bottom width of the chutes are designed to provide depth with a minimum of 1 foot free board.

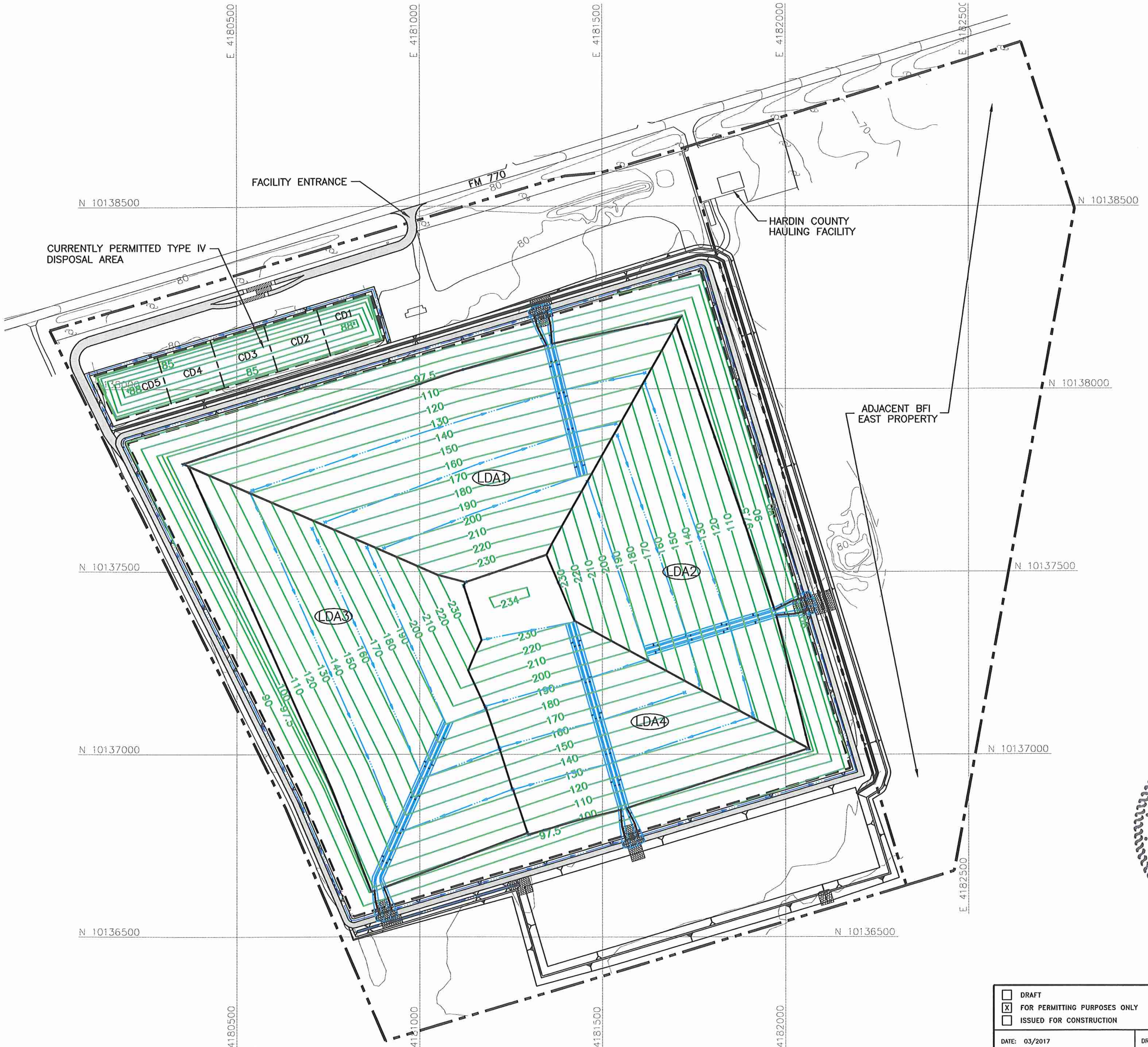
2) Manning's coefficient is obtained from TXDOT Drainage Criteria Manual for gabion mattress.

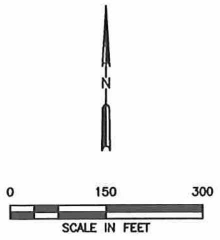
3) 25-yr peak flow rates are obtained from the HEC-HMS program results included in Appendix IIIF-A.

Chute flow apron design for the gabion-lined chutes.

Drainage Area	Acreage	25-Yr Peak Flow Rate (cfs)	Bottom Slope (ft/ft)	Manning's n	Side Slope (lieft)	Side Slope (right)	Bottom Width (ft)	Normal Depth (ft)	Flow Vel. (fps)	Froude Number	Velocity Head (ft)	Energy Head (ft)	Flow Area (sf)	Flow Top Width (ft)
LDA1	10.70	75.8	0.25	0.033	2.0	2.0	16.0	0.39	11.59	3.346	2.09	2.48	6.54	17.56
LDA2	8.76	60.5	0.25	0.033	2.0	2.0	16.0	0.34	10.64	3.278	1.76	2.10	5.69	17.36
LDA3	11.51	77.0	0.25	0.033	2.0	2.0	16.0	0.39	11.65	3.351	2.11	2.50	6.61	17.57
LDA4	7.89	56.7	0.25	0.033	2.0	2.0	16.0	0.33	10.38	3.258	1.67	2.00	5.46	17.31

Gabion-lined chutes are designed wider at the lower end to provide slower flow velocity at low water croswsing and the letdown perimeter channel confluence.





LEGEND

— — —— · —	BFI EAST PROPERTY BOUNDARY
—— · — · ——	PERMIT BOUNDARY
— — — — — —	CURRENTLY PERMITTED LIMITS OF WASTE
—— — — —— —	CELL BOUNDARY
N 10137500	STATE PLANE COORDINATE GRID
70	EXISTING CONTOUR
234	PROPOSED FINAL CONTOUR (SEE NOTE 3)
— · · — · · —	PROPOSED DRAINAGE SWALE
═══	PROPOSED DRAINAGE CHUTE
———	DRAINAGE AREA BOUNDARY
LDA1	DRAINAGE AREA DESIGNATION

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS DEVELOPED BY WEAVER CONSULTANTS GROUP FROM AERIAL PHOTOGRAPHY FLOWN 05-17-2016. GRID SYSTEM IS TIED TO THE TEXAS STATE PLANE COORDINATE SYSTEM NAD 83, CENTRAL ZONE.

DRAINAGE AREA NO.	AREA (ACRES)
LDA 1	10.69
LDA 2	8.74
LDA 3	11.51
LDA 4	7.89

NEVZAT TURAN
84059

12-5-2017

☐ DRAFT
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☐ ISSUED FOR CONSTRUCTION

DATE: 03/2017
FILE: 0120-758-11
CAD: IIIF-C-10 DRAINAGE AREAS.DWG

DRAWN BY: SRF
DESIGN BY: AE
REVIEWED BY: NT

PREPARED FOR
BFI WASTE SYSTEMS OF NORTH AMERICA, LLC

REVISIONS

NO.	DATE	DESCRIPTION
1	11/2017	OWNERSHIP CHANGE

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

MAJOR PERMIT AMENDMENT
LETDOWN STRUCTURE
DRAINAGE AREAS

HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS

WWW.WCGRP.COM

DRAWING IIIF-C-10

O:\0120\758\2214B EXPANSION\IIIF\IIIF-C-10-CHUTE-LETDOWN STRUCTURE AREAS.dwg, 11/15/2017 11:10:40 AM, rsellers, 1:2

Texas Department of Transportation

IH 635 Managed Lanes Project

Technical Provisions

Attachment 12-1A

Drainage Criteria Manual

Table 5.3.1 Manning's "n" Values

Channel Description	**"n" value**
Channel Roughness Coefficients:	
Well Defined Natural Channel	
Rock bottom	0.035
Dirt lined with light vegetation	0.040
Moderate vegetation on banks	0.060
Heavy vegetation on banks	0.070
Channel Description	**"n" value**
Irregular Channel with Meanders and Pools	
Rock bottom	0.047
Dirt lined with light vegetation	0.052
Moderate vegetation on banks	0.072
Heavy vegetation on banks	0.080
Lined Channel	
Concrete-lined channel	0.020
Grouted riprap	0.035
Ungrouted riprap	0.040
Gabion mattress	0.033 *
Geotextile fabric with established vegetation	0.043
Maintained grass-lined channel	0.035
Non-maintained grass-lined channel	0.060
Overbank Roughness Coefficients:	
Undeveloped Overbank	
Short grass, no brush	0.050
Tall grass, no brush	0.060
Grass with moderate tree cover	0.080
Grass with heavy tree cover	0.120
Developed Overbanks	
Residential	0.150
Developed commercial or industrial	0.100
Parks, manicured open space	0.035

APPENDIX IIIF-D

EROSION LAYER EVALUATION

STATE OF TEXAS
NEVZAT TURAN
84059
LICENSED PROFESSIONAL ENGINEER

12-5-2017

Includes pages IIIF-D-1 through IIIF-D-23

EROSION LAYER EVALUATION

This appendix presents the supporting documentation for evaluation of the thickness of the erosion layer for the final cover system at the Hardin County Landfill. The evaluation is based on the premise of adding excess soil to increase the time required before maintenance is needed as recommended in the EPA Solid Waste Disposal Facility Criteria Technical Manual (EPA 530-R-93-017, November 1993).

The design procedure is as follows:

1. Minimum thickness of the erosion layer at the end of the 30-year postclosure period is evaluated based on the depth of frost penetration or 6 inches, whichever is greater. For Hardin County, the approximate depth of frost penetration is approximately 1.0 inches (see IIIF-D-9). Therefore, the minimum erosion layer thickness is 6 inches.
2. Soil loss is calculated using the Universal Soil Loss Equation (USLE) by following SCS procedures. The soil loss is adjusted by a safety factor of 2 and is then converted to a thickness. The thickness of the soil loss over a 30-year postclosure period is added to the minimum thickness of the erosion layer (from Step 1) to yield an initial thickness to be placed at closure of the site. According to the USLE, the typical 4 percent topslope and 25 percent topslope require a minimum of 6.06 inches and 7.029 inches, respectively, for the erosion layer. These USLE requirements include the 6-inch minimum required by regulations. Conservatively, a 24-inch erosion layer is proposed over final cover. These calculations begin on page IIIF-D-3.
3. Stormwater flows over the final cover system by (1) sheet flow over the topslope and sideslopes and (2) channelized flow in the drainage berms (or swales). As discussed in Section 2.2 and Appendix IIIF-C, flow also occurs in the letdown structures. The letdown structures are lined with gabions to prevent erosion given that the velocities in the letdowns are over 5 ft/s.
4. Sheet flow velocities for the topslope and sideslope cases for a 25-year storm event are calculated to be less than permissible nonerosive velocities. A permissible nonerocive velocity is defined as 5.0 ft/s or less. Calculated sheet flow velocities range from 0.22 to 0.54 ft/s for topslope and sideslope cases. The supporting calculations are presented on pages IIIF-D-15 through IIIF-D-18.

5. Channelized flow for drainage swales is also calculated to be less than permissible nonerosive velocities. Channel velocity calculations demonstrating that the designed channels provide nonerosive velocities are provided in Appendix IIIF-C.
6. Vegetation for the site will be native and introduced grasses with root depths of 6 to 8 inches. The seeding is specified on page IIIF-D-23. The seeding is specified by TxDOT for temporary and permanent erosion control for Hardin County, Texas.
7. Native and introduced grasses will be hydroseeded with fertilizer on the disked (parallel to contours) erosion layer upon final grading. Temporary cold weather vegetation will be established if needed. Irrigation will be employed for 6 to 8 weeks or until vegetation is well established. Erosion control measures such as silt fences and straw bales will be used to minimize erosion until the vegetation is established. Areas that experience erosion or do not readily vegetate after hydroseeding will be reseeded until vegetation is established or the soil will be replaced with soil that will support the grasses.
8. Slope stability information is included in Appendix IIIE.

Required: Determine expected soil loss and minimum thickness for the erosion layer for the expansion area and for the existing landfill area.

Method: Expected soil loss is calculated using the Universal Soil Loss Equation. Minimum erosion layer thickness is determined by adding the minimum thickness allowed by TCEQ to the expected soil loss.

References:

1. SCS National Engineering Handbook, Chapter 3 - Erosion.
2. TNRCC, *Use of the USLE in Final Cover/Configuration Design*, 1993.
3. United States Department of Agriculture, Soil Conservation Service, *Soil Survey of Hill County, Texas*, January 2007
4. United States Environmental Protection Agency, *Solid Waste Disposal Facility Criteria Technical Manual*, 1993.

Solution: 1. Soil Loss Equation: $A=RKL_SCP$

Where:

$A=$ Soil loss (tons/ac/yr)
$R=$ Rainfall factor
$K=$ Soil erodibility factor
$L_S=$ Slope length/slope gradient factor
$C=$ Plant cover or cropping management factor
$P=$ Erosion practice factor

The rainfall factor, R, represents the average intensity for the maximum intensity, 30 minute storms over a 22 year period of record compiled by the SCS. Using Figure 1 (Ref 2), Average Annual Values of the R Factor, the R factor for Hardin County is:

$R =$ 440

The soil erodibility factor, K, factor represents the resistance of a soil surface to erosion as a function of the soil's physical and chemical properties. Assume an organic matter content of 2% to determine the K factor. The site top soil will consists of sandy clay with high organic content. Clean compost as a soil amendment maybe added to final cover top soil as necessary to protect against erosion. Therefore, the following is a K value for the site.

$K =$ 0.19

The slope length/slope gradient factor, L_s, represents the erosion of the soil due to both slope length and degree of slope. The slopes of interest are the typical sideslope and topslope conditions.
See attached drawing for the locations of the slopes analyzed.

Topslope Cases

Case 1. Typical topslope
slope = 0.04 ft/ft
length = 100 ft

Case 2. Longest topslope
slope = 0.04 ft/ft
length = 100 ft

Sideslope Cases

Case 3. Typical sideslope
slope = 0.25 ft/ft
length = 120 ft

Case 4. Longest sideslope
slope = 0.25 ft/ft
length = 230 ft

Using the above information and Figure 2 (Ref 2, p.9), the L_s factors are determined.

Case	Slope (%)	Slope Length (ft)	L_s
Topslope Cases			
1. Typical topslope	4	100	0.38
2. Longest topslope	4	100	0.38
Sideslope Cases			
3. Typical sideslope	25	120	6.50
4. Longest sideslope	25	230	9.00

The plant cover or cropping management factor, C, represents the percentage of soil loss that would occur if the surface were partially protected by some combination of cover and management practices. C Factor for Permanent Pasture, Range, and Idle Land with No Appreciable Canopy has the following relation with percent ground cover (GC) (from Ref 2, p.7) .

% GC	C Factor:
0	0.45
20	0.2
40	0.1
60	0.042
80	0.013
95	0.003

C Factor = **0.003** for **95%** ground cover.

The erosion control practice factor, P, measures the effect of control practices that reduce the erosion potential of the runoff by influencing drainage patterns, runoff concentration , and runoff velocity. Contouring for this site will be done only to establish vegetation.

P = 1.00

2. Soil loss calculations

Slope Condition	R	K	L_s	C	P	A (tons/ac/yr)
Topslope Cases 1. Typical Topslope 4% slope 100 ft length	440	0.19	0.380	~~0.0063~~ **0.003**	1.00	~~0.20~~ **0.10**
2. Longest Topslope 4% slope 108 ft length	440	0.19	0.380	~~0.0063~~ **0.003**	1.00	~~0.20~~ **0.10**
Sideslope Cases 3. Typical Sideslope 25% slope 120 ft length	440	0.19	6.500	~~0.0063~~ **0.003**	1.00	~~3.42~~ **1.63**
4. Longest Sideslope 25% slope 230 ft length	440	0.19	9.000	~~0.0063~~ **0.003**	1.00	~~4.74~~ **2.26**

Note: Erosion layer will be maintained to provide ~~90~~ **95%** ground cover.

3. Erosion layer thickness calculations:

$$T_{el} = 6in + \frac{AYF(2000lb/ton)(12in/ft)}{w(43,560sf/ac)}$$

Where: T_{el} = Erosion layer thickness
A = Soil loss (ton/ac/yr)
Y = Postclosure period (yr)
F = Factor of Safety
w = Specific weight of soil (pcf)

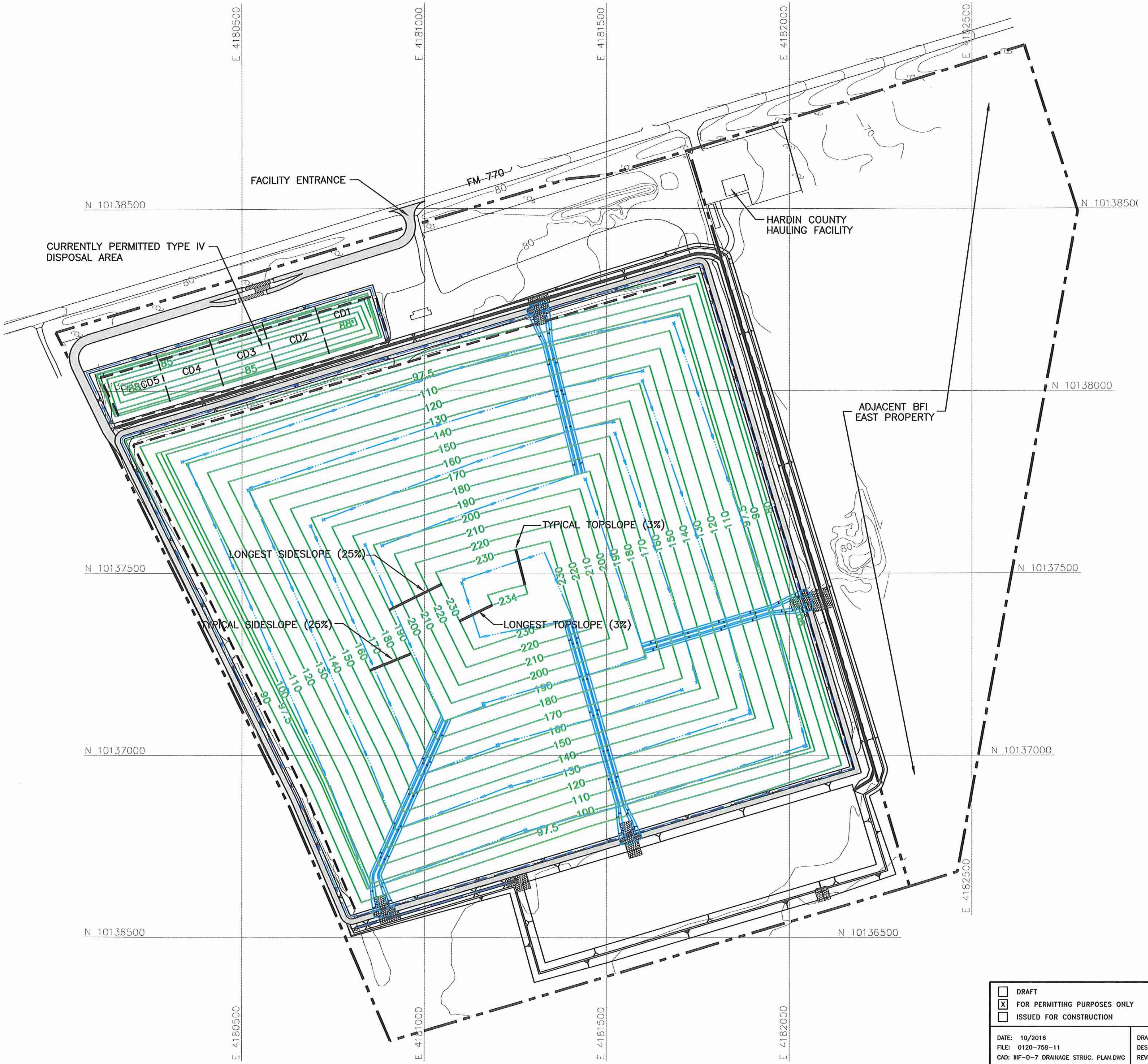
Y = 30 yr
F = 2
w = 110 pcf

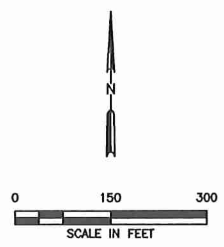
Topslope Cases		
1. Typical Topslope Thickness:		
T_{el}, Required thickness[1] =	**6.029**	in
Total estimated soil loss =	**0.029**	in
Specified thickness =	24.000	in
2. Longest Topslope Thickness:		
T_{el}, Required thickness[1] =	**6.029**	in
Total estimated soil loss =	**0.029**	in
Specified thickness =	24.000	in
Sideslope Cases		
3. Typical Sideslope Thickness:		
T_{el}, Required thickness[1] =	**6.490**	in
Total estimated soil loss =	**0.490**	in
Specified thickness =	24.000	in
4. Longest Sideslope Thickness:		
T_{el}, Required thickness[1] =	**6.678**	in
Total estimated soil loss =	**0.678**	in
Specified thickness =	24.000	in

Note: [1] Required thicknesses include 6 inch minimum required and estimated soil loss.

4. Summary:

Calculated erosion losses are shown Step 2 above. The erosion layer will be a minimum of 24 inches thick for the expansion area and existing landfill area. As shown above, this is a conservative design considering the maximum expected soil loss for a 30 year period is ~~1.33~~ **0.678** inches.





LEGEND

- BFI EAST PROPERTY BOUNDARY
- PERMIT BOUNDARY
- CURRENTLY PERMITTED LIMITS OF WASTE
- CELL BOUNDARY
- N 10137500 STATE PLANE COORDINATE GRID
- 70 EXISTING CONTOUR
- 234 PROPOSED FINAL CONTOUR (SEE NOTE 3)
- PROPOSED DRAINAGE SWALE
- PROPOSED DRAINAGE CHUTE
- DRAINAGE AREA BOUNDARY
- DRAINAGE AREA DESIGNATION

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS DEVELOPED BY WEAVER CONSULTANTS GROUP FROM AERIAL PHOTOGRAPHY FLOWN 05-17-2016. GRID SYSTEM IS TIED TO THE TEXAS STATE PLANE COORDINATE SYSTEM NAD 83, CENTRAL ZONE.

NEVZAT TURAN
84059

12-5-2017

☐ DRAFT
☒ FOR PERMITTING PURPOSES ONLY
☐ ISSUED FOR CONSTRUCTION

DATE: 10/2016
FILE: 0120-758-11
CAD: IIIF-D-7 DRAINAGE STRUC. PLAN.DWG

DRAWN BY: SRF
DESIGN BY: AE
REVIEWED BY: NT

PREPARED FOR
BFI WASTE SYSTEMS OF NORTH AMERICA, LLC

REVISIONS

NO.	DATE	DESCRIPTION
1	11/2017	OWNERSHIP CHANGE

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

MAJOR PERMIT AMENDMENT DRAINAGE STRUCTURE PLAN

HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS

WWW.WCGRP.COM

DRAWING IIIF-D-7

O:\0120\758\2214B EXPANSION\IIIF\IIIF-D-7-Drainage Structure Plan-erosion.dwg, 11/15/2017 11:13:56 AM, rsellers, 1:2

EPA

Solid Waste Disposal Facility Criteria

Technical Manual

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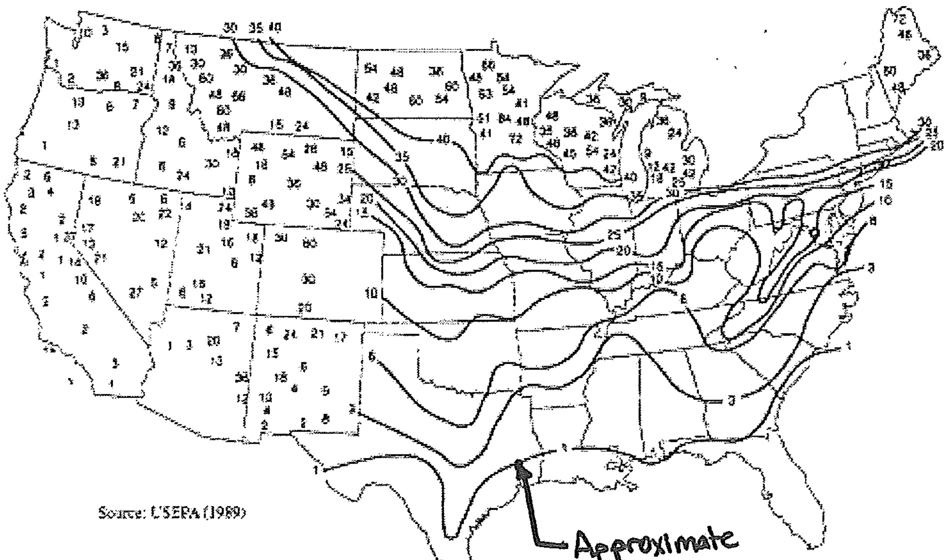


Source: USEPA (1989)

Approximate Site Location

Figure 6-4
Regional Depth of Frost Penetration in Inches

TEXAS NATURAL RESOURCE CONSERVATION COMMISSION

USE OF THE UNIVERSAL SOIL LOSS EQUATION IN FINAL COVER/CONFIGURATION DESIGN

PROCEDURAL HANDBOOK

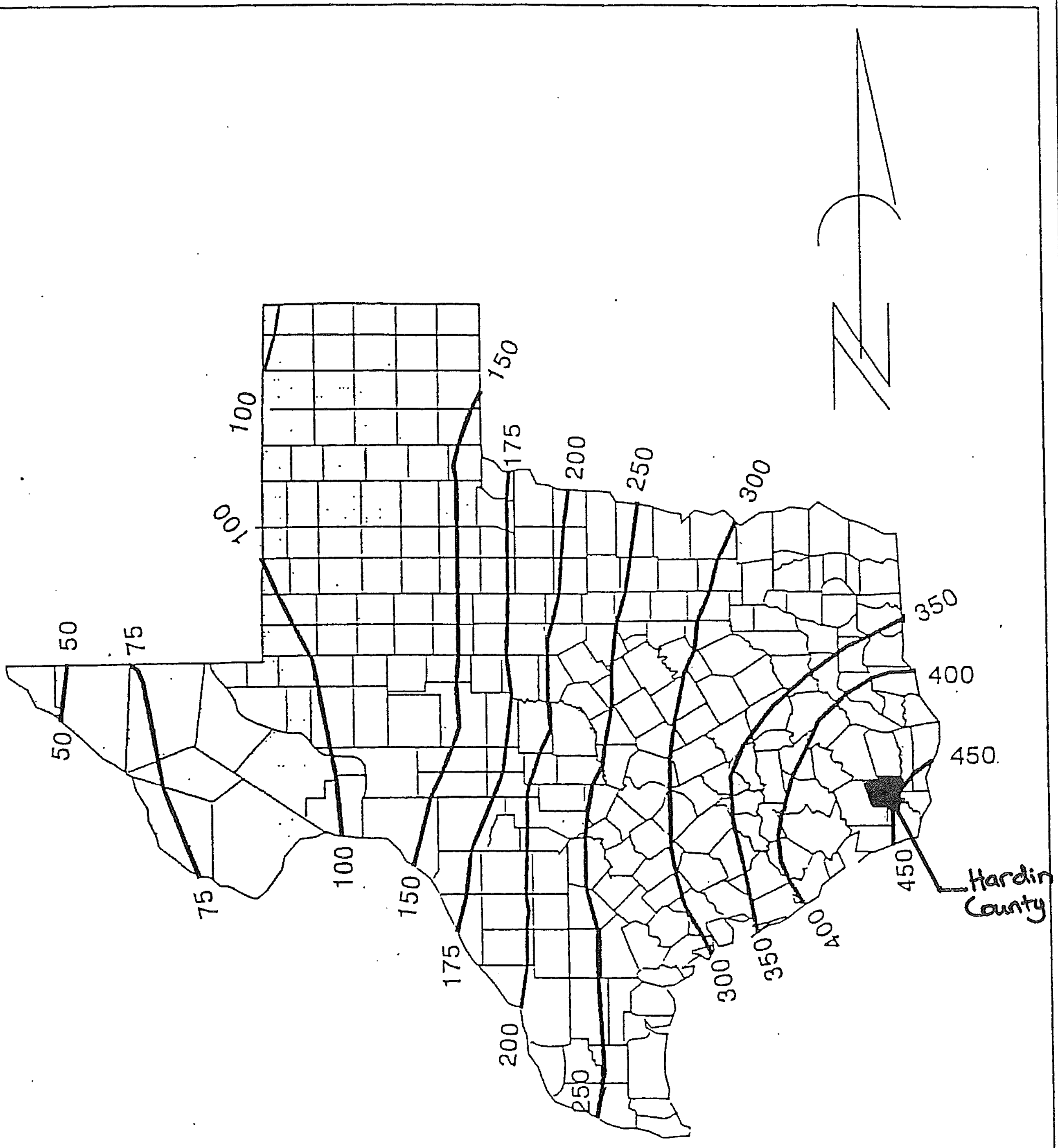
PERMITS SECTION
MUNICIPAL SOLID WASTE DIVISION

OCTOBER 1993

Table 1 Approximate Values of Factor K for USDA Textural Classes

Texture Class.	Organic Matter Content		
	<0.5%	2%	4%
	K	K	K
Sand	0.05	0.03	0.02
Fine Sand	0.16	0.14	0.10
Very Fine Sand	0.42	0.36	0.28
Loamy Sand	0.12	0.10	0.08
Loamy Fine Sand	0.24	0.20	0.16
Loamy Very Fine Sand	0.44	0.38	0.30
Sandy Loam	0.27	0.24	0.19
Fine Sandy Loam	0.35	0.30	0.24
Very Fine Sandy Loam	0.47	0.41	0.33
Loam	0.38	0.32	0.29
Silt Loam	0.48	0.42	0.33
Silt	0.60	0.52	0.42
Sandy Clay Loam	0.27	0.25	0.21
Clay Loam	0.28	0.25	0.21
Silty Clay Loam	0.37	0.32	0.26
Sandy Clay	0.14	0.13	0.12
Silty Clay	0.25	0.23	0.19
Clay	0.13 - 0.29 K=0.19		

The values shown are estimated averages of broad ranges of specific-soil values. When a texture is near the borderline of two texture classes, use the average of the two K values.



W.H. Wischmeier, SEA, 1976

FIGURE 1. - AVERAGE ANNUAL VALUES OF THE RAINFALL EROSION INDEX

Table 2 Factor C for permanent pasture, range, and idle land[1]

Vegetative Canopy		Cover that contacts the soil surface					
Type and height[2]	Percent cover[3]	Percent ground cover					
		0	20	40	60	80	95+
No Appreciable Canopy		0.45	0.20	0.10	0.042	0.013	0.003
Tall weeds or short brush with average drop fall height of 20 in.	25	0.36	0.17	0.09	0.038	0.013	0.011
	50	0.26	0.13	0.07	0.035	0.012	0.003
	75	0.17	0.10	0.06	0.032	0.011	0.003

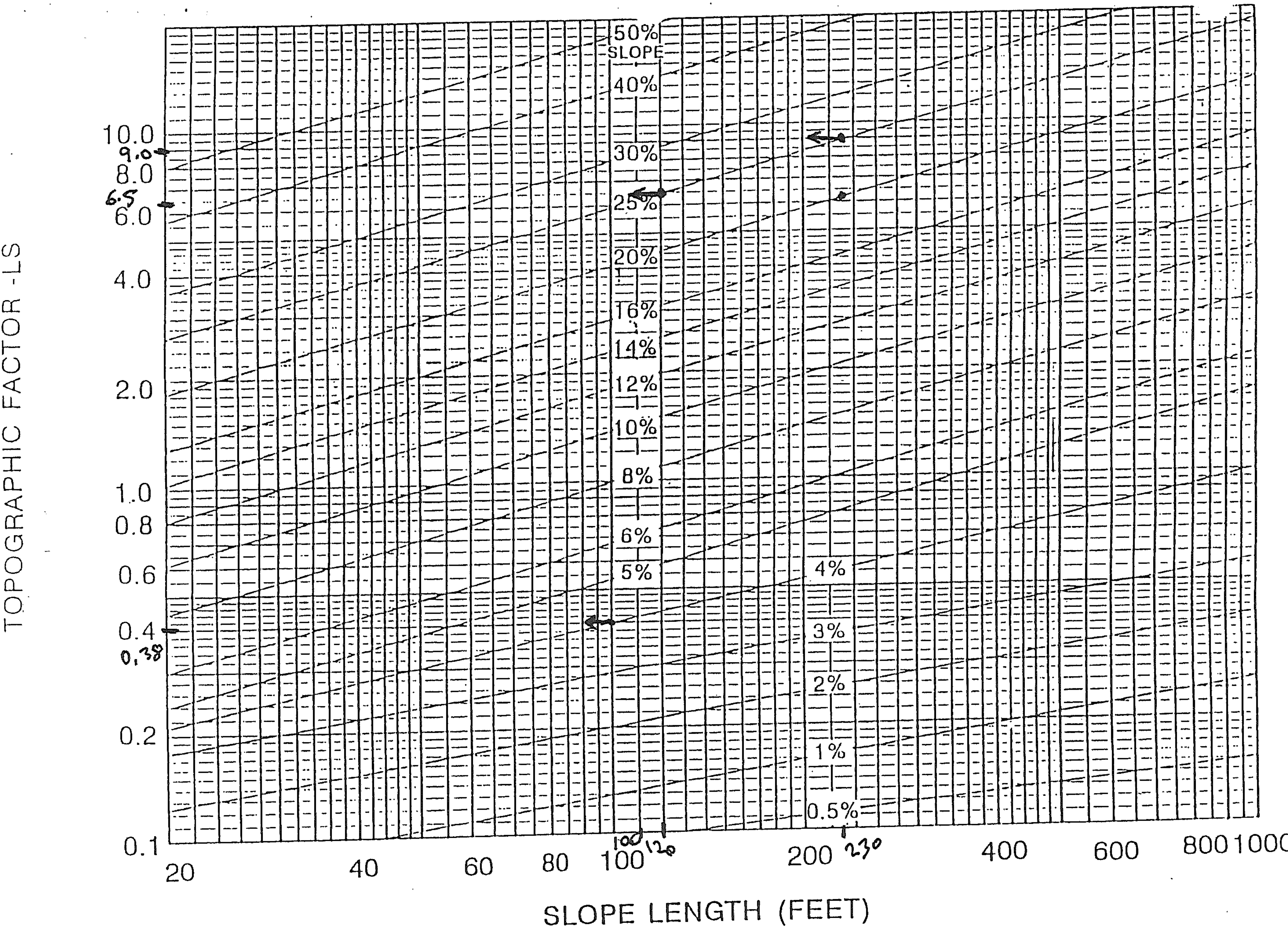
Extracted from:
United States Department of Agriculture, AGRICULTURE HANDBOOK NUMBER 537

---

[1] The listed C values assume that the vegetation and mulch are randomly distributed over the entire area.

[2] Canopy height is measured as the average fall height of water drops falling from the canopy to the ground. Canopy effect is inversely proportional to drop fall height and is negligible if fall height exceeds 33 ft.

[3] Portions of total-area surface that would be hidden from view by canopy in a vertical projection (a bird's-eye view).



Extracted from U.S.D.A. handbook No. 537 (Figure 4).

FIGURE 2.- Slope effect chart (topographic factor, LS). LS = $(\lambda/72.6)'''$ 65.41 $\sin^2$ ()=4.56 sin ()+ 0.065) whereλ= slope lenght in feet ;()=angle of slope; and m = 0.2 for gradients <1 percent, 0.3 for 1 to 3 slopes; 0.4 for 3.5 to 4.5 percent slopes, and 0.5 for slopes of 5 percent or steeper.

HARDIN COUNTY LANDFILL
0120-758-11-02
SHEET FLOW VELOCITY

Required: Determine the sheet flow velocity for the final cover system design and compare to the permissible non-erodible flow velocity.

Method:

1. Determine the flow using the Rational Method.
2. Calculate flow depth using Kinematic Wave procedures.
3. Compute flow velocity and compare to permissible non-erodibility velocity.

References:

1. Raudkivi, A.J., *Hydrology - An Advanced Introduction to Hydrological Processes and Modeling*, 1979.
2. Texas Department of Transportation, *Bridge Division Hydraulic Manual*, December 1985.
3. United States Soil Conservation Service, *TR-55 Hydrology for Small Watersheds*, December 1989.

Solution: Use the typical case scenarios from the USLE calculation to determine the expected sheet flow velocity.

Topslope Cases

Case 1. Typical topslope			Case 2. Longest topslope		
slope =	0.04	ft/ft	slope =	0.04	ft/ft
length =	100	ft	length =	100	ft

Sideslope Cases

Case 3. Typical sideslope			Case 4. Longest sideslope		
slope =	0.25	ft/ft	slope =	0.25	ft/ft
length =	120	ft	length =	230	ft

Time of Concentration:

$$t_c = \frac{0.007(nL)^{0.8}}{(P_{2,24})^{0.5}S^{0.4}}$$

Where:

t_c = time of concentration (hr)
n = Manning's roughness coefficient
L = slope length
$P_{2,24}$ = 2-year, 24-hour rainfall depth (in)
S = slope (ft/ft)

Determine $P_{2,24}$:

$$i = \frac{b}{(t_c + d)^e}$$

Where:

i = rainfall intensity (in/hr)
b = constant for Hardin County = 80
d = constant for Hardin County = 7.5
e = constant for Hardin County = 0.720
t_d = storm duration (min) = 1440

i = 0.42 in/hr

$P_{2,24}$ = 4.08 in (Ref 2.)

Calculate t_c:

Topslope Cases

Case 1:

n = 0.24
L = 100
$P_{2,24}$ = 4.1
S = 0.04
t_c = 0.16 hr
9.58 min

Case 2:

n = 0.24
L = 100
$P_{2,24}$ = 4.1
S = 0.04
t_c = 0.16 hr
9.58 min

Sideslope Cases

Case 3:

n = 0.24
L = 120
$P_{2,24}$ = 4.1
S = 0.25
t_c = 0.09 hr
5.32 min

Case 4:

n = 0.24
L = 230
$P_{2,24}$ = 4.1
S = 0.25
t_c = 0.15 hr
8.96 min

Calculate the design 25-year frequency for each condition:

$$Q = CiA$$

Where:

Q = flow rate (cfs)
C = runoff coefficient
i = rainfall intensity (in/hr)
A = drainage area (ac)

$i = b/(t_c+d)^e$

Where:

i = rainfall intensity (in/hr)
b = constant for Hardin County = 80
d = constant for Hardin County = 7.5
e = constant for Hardin County = 0.72

t_c = time of concentration (min)

For a unit width of final cover, the flow lengths shown on sheet IIIF-D-7 for each case is used.

A=[Length (ft) x Width (ft)] / 43560 = acres

Topslope Cases

Case 1:

C =	0.7	
t_c =	9.58	min
i =	10.37	in/hr
Length:	100.00	ft
A	0.0023	ac
Q =	0.017	cfs

Case 2:

C =	0.7	
t_c =	9.58	min
i =	10.37	in/hr
Length:	100.00	ft
A	0.0023	ac
Q =	0.017	cfs

Sideslope Cases

Case 3:

C =	0.7	
t_c =	5.32	min
i =	12.74	in/hr
Length:	120.00	ft
A	0.0028	ac
Q =	0.025	cfs

Case 4:

C =	0.7	
t_c =	8.96	min
i =	10.65	in/hr
Length:	230.00	ft
A	0.0053	ac
Q =	0.039	cfs

Approximate depth of flow:

Using Manning's Equation

$$V = 1.49/n\ y^{0.67}\ S^{0.5}$$

Q = VA => V = Q/A

A = y x 1 (assuming unit width of flow)

substituting for V

$$Q/y = 1.49/n\ y^{0.67}\ S^{0.5}$$

$$Q = 1.49/n\ y^{1.67}\ S^{0.5}$$

solve for y

$$y = (Qn/1.49\ S^{0.5})^{1/1.67}$$

$$y = (Qn/1.49S^{0.5})^{0.6}$$

Topslope Cases

Case 1:

Q =	0.017	cfs
n =	0.24	
S =	0.04	ft/ft
y =	0.075	ft

Case 2:

Q =	0.017	cfs
n =	0.24	
S =	0.04	ft/ft
y =	0.075	ft

Sideslope Cases

Case 3:			Case 4:		
Q =	0.025	cfs	Q =	0.039	cfs
n =	0.24		n =	0.24	
S =	0.25	ft/ft	S =	0.25	ft/ft
y =	0.055	ft	y =	0.073	ft

Determine sheet flow velocity:

V = Q/A (assume unit flow width for the flow area, A)

Topslope Cases

Case 1:			Case 2:		
Q =	0.017	cfs	Q =	0.017	cfs
A =	0.075	sf	A =	0.075	sf
V =	0.22	ft/s	V =	0.22	ft/s

Sideslope Cases

Case 3:			Case 4:		
Q =	0.025	cfs	Q =	0.039	cfs
A =	0.055	sf	A =	0.073	sf
V =	0.45	ft/s	V =	0.54	ft/s

Permissible non-erodible velocity is 5.0 ft/s. Therefore, expected sheet flow velocity is acceptable on the final cover system top and side slopes.

USDA

United States
Department of
Agriculture

**Natural
Resources
Conservation
Service**

**Conservation
Engineering
Division**

Technical
Release 55

June 1986

Urban Hydrology for Small Watersheds

TR-55

Chapter 3

Time of Concentration and Travel Time

Travel time (T_t) is the time it takes water to travel from one location to another in a watershed. T_t is a component of time of concentration (T_c), which is the time for runoff to travel from the hydraulically most distant point of the watershed to a point of interest within the watershed. T_c is computed by summing all the travel times for consecutive components of the drainage conveyance system.

T_c influences the shape and peak of the runoff hydrograph. Urbanization usually decreases T_c, thereby increasing the peak discharge. But T_c can be increased as a result of (a) ponding behind small or inadequate drainage systems, including storm drain inlets and road culverts, or (b) reduction of land slope through grading.

Factors affecting time of concentration and travel time

Surface roughness

One of the most significant effects of urban development on flow velocity is less retardance to flow. That is, undeveloped areas with very slow and shallow overland flow through vegetation become modified by urban development: the flow is then delivered to streets, gutters, and storm sewers that transport runoff downstream more rapidly. Travel time through the watershed is generally decreased.

Channel shape and flow patterns

In small non-urban watersheds, much of the travel time results from overland flow in upstream areas. Typically, urbanization reduces overland flow lengths by conveying storm runoff into a channel as soon as possible. Since channel designs have efficient hydraulic characteristics, runoff flow velocity increases and travel time decreases.

Slope

Slopes may be increased or decreased by urbanization, depending on the extent of site grading or the extent to which storm sewers and street ditches are used in the design of the water management system. Slope will tend to increase when channels are straightened and decrease when overland flow is directed through storm sewers, street gutters, and diversions.

Computation of travel time and time of concentration

Water moves through a watershed as sheet flow, shallow concentrated flow, open channel flow, or some combination of these. The type that occurs is a function of the conveyance system and is best determined by field inspection.

Travel time (T_t) is the ratio of flow length to flow velocity:

$$T_t = \frac{L}{3600V} \qquad \text{[eq. 3-1]}$$

where:

T_t = travel time (hr)
L = flow length (ft)
V = average velocity (ft/s)
3600 = conversion factor from seconds to hours.

Time of concentration (T_c) is the sum of T_t values for the various consecutive flow segments:

$$T_c = T_{t_1} + T_{t_2} + \ldots T_{t_m} \qquad \text{[eq. 3-2]}$$

where:

T_c = time of concentration (hr)
m = number of flow segments

IIIF-D-20

Sheet flow

Sheet flow is flow over plane surfaces. It usually occurs in the headwater of streams. With sheet flow, the friction value (Manning's n) is an effective roughness coefficient that includes the effect of raindrop impact; drag over the plane surface; obstacles such as litter, crop ridges, and rocks; and erosion and transportation of sediment. These n values are for very shallow flow depths of about 0.1 foot or so. Table 3-1 gives Manning's n values for sheet flow for various surface conditions.

Table 3-1 Roughness coefficients (Manning's n) for sheet flow

Surface description	n [1]
Smooth surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils:	
Residue cover ≤20%	0.06
Residue cover >20%	0.17
Grass:	
Short grass prairie	0.15
Dense grasses [2]	0.24
Bermudagrass	0.41
Range (natural)	0.13
Woods:[3]	
Light underbrush	0.40
Dense underbrush	0.80

[1] The n values are a composite of information compiled by Engman (1986).

[2] Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures.

[3] When selecting n , consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.

For sheet flow of less than 300 feet, use Manning's kinematic solution (Overtop and Meadows 1976) to compute T_t:

$$T_t = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5} s^{0.4}} \qquad \text{[eq. 3-3]}$$

where:

T_t = travel time (hr),
n = Manning's roughness coefficient (table 3-1)
L = flow length (ft)
P_2 = 2-year, 24-hour rainfall (in)
s = slope of hydraulic grade line (land slope, ft/ft)

This simplified form of the Manning's kinematic solution is based on the following: (1) shallow steady uniform flow, (2) constant intensity of rainfall excess (that part of a rain available for runoff), (3) rainfall duration of 24 hours, and (4) minor effect of infiltration on travel time. Rainfall depth can be obtained from appendix B.

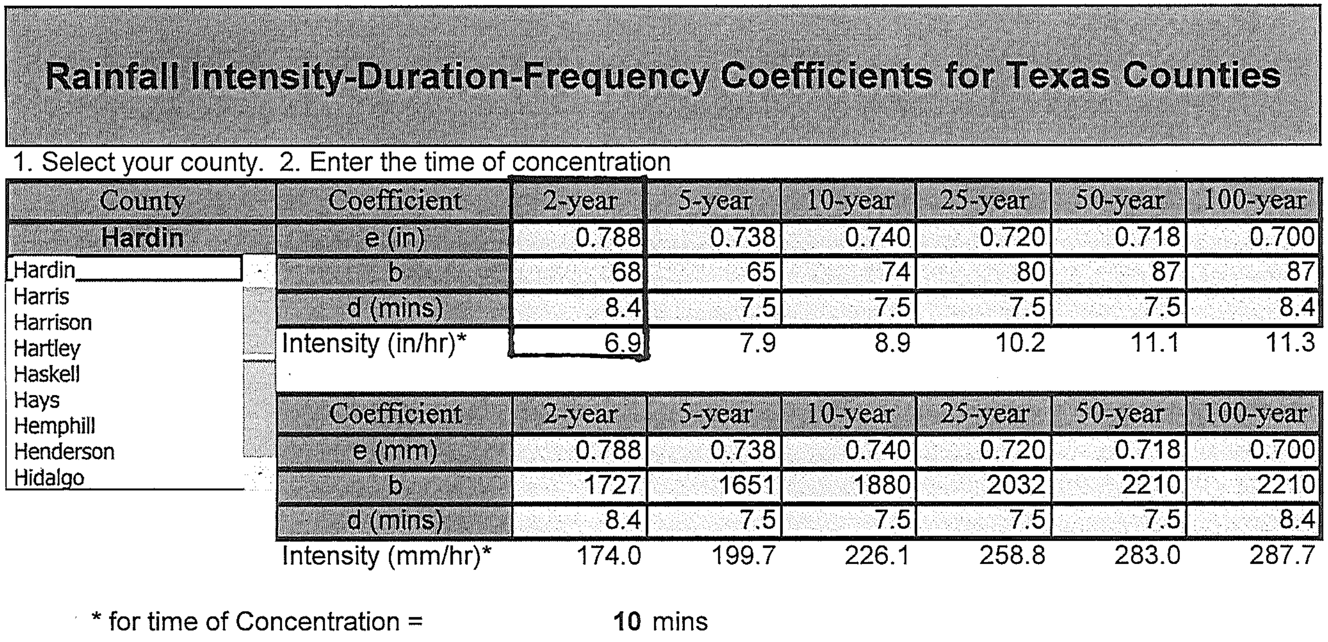
Shallow concentrated flow

After a maximum of 300 feet, sheet flow usually becomes shallow concentrated flow. The average velocity for this flow can be determined from figure 3-1, in which average velocity is a function of watercourse slope and type of channel. For slopes less than 0.005 ft/ft, use equations given in appendix F for figure 3-1. Tillage can affect the direction of shallow concentrated flow. Flow may not always be directly down the watershed slope if tillage runs across the slope.

After determining average velocity in figure 3-1, use equation 3-1 to estimate travel time for the shallow concentrated flow segment.

Open channels

Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, or where blue lines (indicating streams) appear on United States Geological Survey (USGS) quadrangle sheets. Manning's equation or water surface profile information can be used to estimate average flow velocity. Average flow velocity is usually determined for bankfull elevation.



Rainfall Intensity-Duration-Frequency Coefficients for Texas Counties

1. Select your county. 2. Enter the time of concentration

County	Coefficient	2-year	5-year	10-year	25-year	50-year	100-year
Hardin	e (in)	0.788	0.738	0.740	0.720	0.718	0.700
	b	68	65	74	80	87	87
	d (mins)	8.4	7.5	7.5	7.5	7.5	8.4
	Intensity (in/hr)*	6.9	7.9	8.9	10.2	11.1	11.3

Hardin
Harris
Harrison
Hartley
Haskell
Hays
Hemphill
Henderson
Hidalgo

Coefficient	2-year	5-year	10-year	25-year	50-year	100-year
e (mm)	0.788	0.738	0.740	0.720	0.718	0.700
b	1727	1651	1880	2032	2210	2210
d (mins)	8.4	7.5	7.5	7.5	7.5	8.4
Intensity (mm/hr)*	174.0	199.7	226.1	258.8	283.0	287.7

* for time of Concentration = **10** mins

Table 1 (continued)
Permanent Rural Seed Mix

District and Planting Dates	Clay Soils		Sandy Soils	
	Species and Rates (lb. PLS/ac.)		Species and Rates (lb. PLS/ac.)	
11 (Lufkin) Feb. 1 – May 15	Green Sprangletop	0.3	Green Sprangletop	0.3
	Bermudagrass	1.8	Bermudagrass	2.1
	Bahiagrass (Pensacola)	9.0	Bahiagrass (Pensacola)	9.0
	Sideoats Grama (Haskell)	2.7	Sand Lovegrass	0.5
	Illinois Bundleflower	1.0	Lance-Leaf Coreopsis	1.0
12 (Houston) Jan. 15 – May 15	Green Sprangletop	0.3	Green Sprangletop	0.3
	Bermudagrass	2.1	Bermudagrass	2.4
	Sideoats Grama (Haskell)	3.2	Bahiagrass (Pensacola)	10.5
	Little Bluestem (Native)	1.4	Weeping Lovegrass (Ermelo)	0.5
	Illinois Bundleflower	1.0	Lance-Leaf Coreopsis	1.0
13 (Yoakum) Jan. 15 – May 15	Green Sprangletop	0.3	Green Sprangletop	0.3
	Sideoats Grama (Haskell)	3.6	Bermudagrass	1.8
	Bermudagrass	1.8	Bahiagrass (Pensacola)	6.0
	Little Bluestem (Native)	1.4	Sand Lovegrass	0.6
	Illinois Bundleflower	1.0	Weeping Lovegrass (Ermelo)	0.6
			Partridge Pea	1.0
14 (Austin) Feb. 1 – May 15	Green Sprangletop	0.3	Green Sprangletop	0.3
	Bermudagrass	0.9	Bermudagrass	2.4
	Sideoats Grama (Haskell)	2.7	Weeping Lovegrass (Ermelo)	0.8
	Little Bluestem (Native)	1.0	Sand Lovegrass	0.8
	Blue Grama (Hachita)	0.9	Partridge Pea	1.0
	Illinois Bundleflower	1.0		
15 (San Antonio) Feb. 1 – May 1	Green Sprangletop	0.3	Green Sprangletop	0.3
	Bermudagrass	1.2	Bermudagrass	1.8
	Sideoats Grama (Haskell)	2.7	Lehmanns Lovegrass	0.6
	Little Bluestem (Native)	1.4	Sand Lovegrass	0.6
	Plains Bristlegrass	1.2	Buffelgrass (Common)	0.4
	Illinois Bundleflower	1.0	Partridge Pea	1.0
16 (Corpus Christi) Jan. 1 – May 1	Green Sprangletop	0.3	Green Sprangletop	0.3
	Sideoats Grama (Haskell)	2.7	Bermudagrass	1.8
	Bermudagrass	1.8	Buffelgrass (Common)	0.4
	Buffalograss (Texoka)	1.6	Sand Lovegrass	0.6
	Plains Bristlegrass	1.2	Lehmanns Lovegrass	0.6
	Illinois Bundleflower	1.0	Purple Prairieclover	0.5

APPENDIX IIIF-E

PERMITTED LANDFILL CONDITION DRAINAGE ANALYSIS

IESI HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS
TCEQ PERMIT NO. MSW 2214A

PART III SITE DEVELOPMENT PLAN

ATTACHMENT 6
GROUNDWATER AND SURFACE WATER PROTECTION AND DRAINAGE PLANS

Prepared for

IESI TX Landfill LP

August 1994
Revised August 1996
Revised March 2007

Revised June 2007

STATE OF TEXAS
KENNETH J. WELCH
50773
REGISTERED PROFESSIONAL ENGINEER

6/22/07

Prepared by

BIGGS & MATHEWS ENVIRONMENTAL
1700 Robert Road ◆ Mansfield, Texas 76063 ◆ 817-563-1144

CONTENTS

Attachment 6 – Groundwater and Surface Water Protection and Drainage Plans

The Attachment 6 – Groundwater and Surface Water Protection and Drainage Plans was prepared by KSA Engineers, Inc., dated August 16, 1996. There are no revisions to this attachment.

Attachment 6A – Erosion and Sediment Control Plan

The Attachment 6A –Erosion and Sediment Control Plan is a new attachment that addresses the requirements of Subchapter G: Surface Water Drainage as related to the provisions for erosion and sediment controls during the intermediate cover phase of landfill development per §330.305(d) and §330.305(e)(2). Attachment 6A – Erosion and Sediment Control Plan has been prepared by Biggs and Mathews Environmental, Inc., dated June 2007.

TABLE OF CONTENTS

ATTACHMENT #6

R. L. THURBER
44228

CERTIFIED STATEMENT
CHANGE IN DRAINAGE CONTROL PLAN

This certified statement is offered in accordance with the request of Mr. Mark Dollins, P.E., of the Texas Natural Resources Conservation Commission (TNRCC). Mr. Dollins' request, included in a letter dated December 9, 1996 to the Honorable Tom Mayfield, County Judge, Hardin County, regarded the proposed modifications to the Hardin County Landfill Permit (MSW Permit #2214) and was phrased as follows:

> Please provide a statement certified by a registered professional engineer that the proposed, modified drainage plan does not significantly change the drainage flows from the landfill in its final condition as compared with the developed conditions which were approved in the original permit. This is required to demonstrate that the proposed modifications complies with 30 TAC §305.70(i) and/or 30 TAC §305.70(g)(20).

Total runoff flow from the site has increased by 37.96 cfs due to changes in the landfill footprint and calculation variable assumptions. This amounts to an approximate increase of 0.006 cfs per foot of property line from the original permit.

The proposed, modified drainage plan, as outlined in Part III, Attachment 6, of the Hardin County Landfill Permit (MSW Permit #2214) modification does not significantly change the drainage flows from the landfill in its final condition as compared with the developed conditions which were approved in the original permit.

This statement is offered in support of 30 TAC §305.70(i) and/or 30 TAC §305.70(g)(20) and certified on this, the 15th day of January, 1997, by the undersigned:

1/15/97

Robert L. Thurber, P. E.

1 DRAINAGE AND RUNOFF CONTROL ANALYSIS

The groundwater and surface water protection and drainage plan contained within this attachment provides for the safe passage of any internal or externally adjacent floodwaters for the Hardin County Landfill.

The rational method was utilized in calculating the runoff for the existing site prior to any construction and for the final site design which includes the finished contours of the proposed cover for the site. The rational method for determining the discharge is recommended for sites less than 200 acres and is represented by the formula Q=CIA, where:

Q = Discharge in cubic feet per second
C = Runoff Coefficient
I = Rainfall intensity in inches per hour
A = Drainage area in acres

The runoff coefficients for the existing and proposed site improvements were selected as 0.35 and 0.85 respectively. These assumptions are based on Table 5 of the Texas Department of Highways and Public Transportation's (TxDOT) "Bridge Division Hydraulic Manual" (Manual) third addition. Said coefficients are suitable for the type of drainage area encountered on this proposed site and in accordance with sound engineering practices.

Rainfall intensity was determined utilizing the referenced manual above. The rainfall intensity is represented by the formula:

$$I = b / (t_c + d)^e$$

Where b, d, and e are constants listed by Texas counties in Table 6 of the Manual.

The time of concentration was determined utilizing the various slopes and the type of topography in conjunction with Figure 5 of the Manual. The slope distance was then divided by the velocity to determine the amount of time in minutes for water to traverse the drainage area under consideration.

1.1 MANNING EQUATION

The Manning Equation was utilized in the sizing of all ditches and culverts for the site. The Manning Equation for open channel flow is an often used engineering formula and is not being reproduced herein, however, certain assumptions are required in the utilization of said formula.

The coefficient of roughness used for this project site is 0.013 and 0.025 for concrete pipe and channel flow respectively.

Three (3) to one (1) side slopes were used on the triangular channel section.

The calculations for the culverts and ditches are made a part of this attachment and the results are shown on Part III, Attachments #6, Final Drainage Detail, and as derived from Part III, Attachments #6, and Attachment #3.

1.2 EXISTING DRAINAGE PATTERN

The existing drainage pattern for the site in general in Northwest to Southeast and East with approximately 58 cfs exiting the Southeasterly portion of the tract and 55 cfs exiting the East side of the tract. Minor flows exit the West and North sides of the tract as indicated on Part III, Attachment #3. These existing flows are also shown on the existing drainage drawing included with this Attachment.

Final flows are shown on the drawing included with this Attachment. Approximately **172** cfs will be discharging in the Southwest corner and routing through the wetlands to the Southeast corner, **78**cfs on the South side, **95cfs at the Southeast corner, 107** cfs on the East side, and **22** cfs on the Northeast corner **as ditched from the C & D area.** Runoff exiting the southern portion of the site will generally be diffused and sheet flow in nature. Sediment in runoff which could be intercepted by the wetlands will have been removed through the passing of runoff through erosion control measures discussed in this attachment as well as through the passing of runoff through a shallow basin included at the Southwest corner of the landfill. Inundation of the wetlands will be gradual and clean no harm to this area.

At the suggestion of TNRCC personnel, a storm hydrograph was routed through the wetlands to determine if they provide any storage for the protection of surrounding properties. **Originally** based on an estimated peak inflow of approximately145 cfs, this hydrograph approximated the 25 year, 24 hour storm slow which might be routed through the wetlands when the landfill is total closed (peak runoff conditions). **As the new estimated peak inflow is approximately 172 cfs, this analysis was not repeated**. Enough storage was found to be available to provide a peak outflow of 76 cfs. This compares to am original outflow at the site of approximately 58 cfs. Although modeled areas, there is enough in common between these flows to state that there is an appreciable amount of storage in the wetlands for some runoff control. A table illustrating the hydrograph routing follows this page of text.

The storm hydrograph used in this analysis was developed using known data and a computer program called HYDRO. HYDRO was programmed by the Untied States Geologic Survey for use with a series of other related programs used by the Federal Highway Administration. Inputs provided for the development of the hydrograph were: a peak flow of 148 cfs; latitude of 30 degrees, 20 minutes; longitude of 94 degrees, 21 minutes; and a time lag of 0.335 hours. A copy of the Hydro output is included with this attachment.

Routing was accomplished by use of a program called WSPRO. WSPRO is a command line step backwater program for natural channels with an orientation to bridge construction. The Water Surface Profile Computation Model Microcomputer Program (WSPRO) has been designed to provide a water surface profile for six major types of open channel flow situations: unconstricted flow, single opening bridge, bridge opening(s) with spur dikes, single opening embankment overflow, multiple alternatives for a single job, multiple openings. WSPRO was originally developed by the United States Geologic Survey (USGS) for the Federal Highway Administration. The model was a batch mode mainframe program, written in FORTRAN. The members of the Pooled Fund Project decided to use WSPRO as the bridge waterways analysis element of the Integrated Computerized Drainage Design System. WSPRO was downloaded to the microcomputer by the USGS and FHWA. The microcomputer version of WSPRO, is dated August 1987.

The command input file forms a logical description of the physical characteristics of a waterway. Once the user is comfortable with this method of data setup, the program provides a step backwater method for determining water surface profiles. The scheme is similar to the Corps of Engineers HEC-2 program. Both WSPRO and HEC-2 are acceptable to the Federal Emergency Management Agency. WSPRO has the advantage that is utilizes more recent approximation techniques for the backwater effects associated with bridge constrictions.

Since all of the surrounding properties are not developed and are used for timber production, the impact to the area is deemed minor.

BILLY D. SIMS
83016

HARDIN COUNTY LANDFILL PERMIT APPLICATION
HYDROGRAPH ROUTNING THROUGH WETLANDS
25 YEAR STORM RUNOFF FROM FINAL COVER OF LANDFILL

For pages 6 & 7

TIME ELAPSED (HOURS)	TIME ELAPSED (SECONDS)	FLOW IN (CFS)	VOLUME IN (CF)	VOLUME STORED (CF)	VOLUME OUT (CF)	FLOW OUT (CFS)	CALC'D STORAGE (CF)
0.1004	361.4	20.72	7,488	5,681	1,807	5	5389
0.1339	481.9	31.08	11,232	8,461	2,771	8	8622
0.1673	602.3	54.76	17,828	13,492	4,337	13	14011
0.2008	722.8	82.88	27,815	21,068	6,746	20	21555
0.2342	843.3	112.48	41,364	31,124	10,240	29	31255
0.2677	963.7	136.16	57,765	42,827	14,937	39	42032
0.3012	1084.2	148.00	75,598	54,515	21,083	51	54965
0.3346	1204.7	145.04	93,069	64,759	28,310	60	64665
0.3681	1325.1	133.20	109,113	72,733	36,380	67	72209
0.4016	1445.6	115.44	123,019	77,965	45,053	72	77598
0.4350	1566.1	96.20	134,610	80,520	54,090	75	80831
0.4685	1686.5	79.92	144,237	80,992	63,245	76	81909
0.5019	1807.0	65.12	152,081	79,922	72,159	74	79753
0.5354	1927.4	53.28	158,499	77,667	80,831	72	77598
0.5689	2047.9	44.40	163,849	74,703	89,145	69	74365
0.6023	2168.4	37.00	168,306	71,210	97,095	66	71131
0.6358	2288.8	31.08	172,049	67,245	104,805	64	68976
0.6693	2409.3	25.16	175,081	63,167	111,914	59	63587
0.7027	2529.8	19.24	177,399	58,980	118,418	54	58198
0.7362	2650.2	14.80	179,181	54,620	124,562	51	54965
0.7696	2770.7	8.88	180,251	50,028	130,223	47	50654
0.8031	2891.2	4.44	180,786	45,502	135,284	42	45265
0.8366	3011.7	0.00	180,786	40,925	139,861	38	40954
0.8866	3191.7	0.00	180,786	34,985	145,801	33	35566
0.9366	3371.7	0.00	180,786	29,945	150,841	28	30177
0.9866	3551.7	0.00	180,786	25,625	155,161	24	25866
1.0366	3731.7	0.00	180,786	22,025	158,761	20	21555
1.0866	3911.7	0.00	180,786	18,965	161,821	17	18322
1.1366	4091.7	0.00	180,786	16,265	164,521	15	16166
1.1866	4271.7	0.00	180,786	13,925	166,861	13	14011
1.2366	4451.7	0.00	180,786	11,945	168,841	11	11855
1.2866	4631.7	0.00	180,786	10,145	170,641	10	10777
1.3366	4811.7	0.00	180,786	8,705	172,081	8	8622
1.3866	4991.7	0.00	180,786	7,445	173,341	7	7544
1.4366	5171.7	0.00	180,786	6,365	174,421	6	6466

HARDIN COUNTY LANDFILL PERMIT APPLICATION
HYDROGRAPH ROUTNING THROUGH WETLANDS
25 YEAR STORM RUNOFF FROM FINAL COVER OF LANDFILL

TIME ELAPSED (HOURS)	TIME ELAPSED (SECONDS)	FLOW IN (CFS)	VOLUME IN (CF)	VOLUME STORED (CF)	VOLUME OUT (CF)	FLOW OUT (CFS)	CALC'D STORAGE (CF)
1.4866	5351.7	0.00	180,786	5,465	175,321	5	5389
1.5366	5531.7	0.00	180,786	4,745	176,041	4	4311
1.5866	5711.7	0.00	180,786	4,025	176,761	4	4311
1.6366	5891.7	0.00	180,786	3,485	177,301	3	3233
1.6866	6071.7	0.00	180,786	2,945	177,841	3	3233
1.7366	6251.7	0.00	180,786	2,585	178,201	2	2155
1.7866	6431.7	0.00	180,786	2,225	178,561	2	2155
1.8366	6611.7	0.00	180,786	1,865	178,921	2	2155
1.8866	6791.7	0.00	180,786	1,505	179,281	2	2155
1.9366	6971.7	0.00	180,786	1,325	179,461	1	1078
1.9866	7151.7	0.00	180,786	1,145	179,641	1	1078
2.0366	7331.7	0.00	180,786	965	179,821	1	1078
2.0866	7511.7	0.00	180,786	785	180,001	1	1078
2.1366	7691.7	0.00	180,786	605	180,181	1	1078
2.1866	7871.7	0.00	180,786	425	180,361	1	1078

```
                 **************** HYDRO - Version 3.2 ****************
                 *  HEC19 / Design Event vs Return Period Program  *
                 *            Date of Run: 08-25-94                *
                                                        PAGE NO   1

     HARDIN COUNTY LANDFILL DETENTION POND

   --- Input File: hardlfdp.hdo

     JOBHARDIN COUNTY LANDFILL DETENTION POND
     FLW4

*** A HYDROGRAPH WILL BE CREATED USING
*** A USER-SUPPLIED PEAK FLOW.

      LOC30 20 94 21

*** THE LATITUDE IS  30 DEGREES, 20 MINUTES
*** THE LONGITUDE IS  94 DEGREES, 21 MINUTES

      QPK 148

 *** THE USER-SUPPLIED PEAK FLOW IS    148.0 CFS.

       DHY
       TLG10  7.8  .27

  *** THE TIME LAG IS   .335 HOURS

        END

     END OF COMMAND FILE

   *** THE USER-SUPPLIED PEAK FLOW, RATHER THAN A COMPUTED VALUE, WILL BE USED
   *** TO DERIVE THE HYDROGRAPH.
                                                         DATE 08-25-94
             ***** HYDRO ***** (Version 3.2) *****       PAGE NO   2

          HARDIN COUNTY LANDFILL DETENTION POND


             GRAPH OF HYDROGRAPH
                      FLOW (CFS) vs.  TIME (HOURS)
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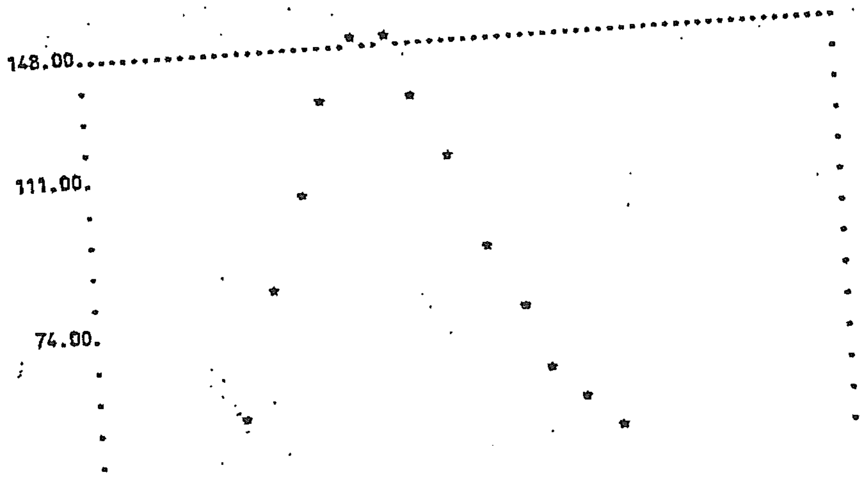

BILLY D. SIMS
53016
8/25/94

For pages 8 & 9

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37.00.                                                   *                      .
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     .        *                                                *                .
     .     *                                                      *  *          .
     .  *                                                               * *     .
.00*.............................................................................*
   .00                  .21                   .42                  .63           .84

      -------------------------------------------------------
      POINT              TIME (HRS)           FLOW (CFS)
      -------------------------------------------------------
         1                 .00000                .00000
         2                 .03346               5.92000
         3                 .06693              11.84000
         4                 .10039              20.72000
         5                 .13385              31.08000
         6                 .16731              54.76000
         7                 .20078              82.88000
         8                 .23424             112.48000
         9                 .26770             136.16000
        10                 .30117             148.00000
        11                 .33463             145.04000
        12                 .36809             133.20000
        13                 .40155             115.44000
        14                 .43502              96.20000
        15                 .46848              79.92001
        16                 .50194              65.12000
        17                 .53540              53.28000
        18                 .56887              44.40000
        19                 .60233              37.00000
        20                 .63579              31.08000
        21                 .66926              25.16000
        22                 .70272              19.24000
        23                 .73618              14.80000
        24                 .76964               8.88000
        25                 .80311               4.44000
        26                 .83657                .00000
      -------------------------------------------------------
      ***** HYDRO ***** (Version 3.2) *****          DATE 08-25-94
                                                     PAGE NO   3

   HARDIN COUNTY LANDFILL DETENTION POND

*** END OF RUN
```

R. L. THURBER
44228
10/17/96

FOR PAGES 10 TO 19A

HARDIN COUNTY LANDFILL
SITE DEVELOPMENT PLAN - ATTATCHMENT 6
GROUNDWATER AND SURFACE WATER PROTECTION AND DRAINAGE PLANS

DRAINAGE CALCULATIONS

POINT OF CONC.	RUN-OFF AREA	$d_1^{1,2}$ (feet)	d_2 (feet)	d_3 (feet)	V_1^4 (ft/sec)	V_2 (ft/sec)	V_3 (ft/sec)	t_{c1}^2 (min)	t_{c2} (min)	t_{c3} (min)	$t_c^{2,3}$ (min)	b^5	d^5	e^5	I^5 (in/hr)	C^5	$AREA^1$ (Ac)	Q= C*I*A (cfs)	DITCH FLOW (cfs)
	2	400.00	721.00		3.75	3.20		1.78	3.76		5.53	80	7.5	0.72	12.5967	0.85	3.62	38.76	
		"V" Ditch: 22' wide, 1.65' deep, 0.54% slope Ditch flow: 1.35' deep, 3.20 fps																	
	3	697.75	23.25		2.60	7.00		4.47	0.06		4.53	80	7.5	0.72	13.3459	0.85	3.00	34.03	68.14
	2,3	400.00	721.00	400.00	3.75	4.19	4.11	1.78	2.87	1.62	6.27	80	7.5	0.72	12.1090	0.85	6.62		
		"V" Ditch: 22' wide, 2.62' deep, 0.54% slope Ditch flow: 1.99' deep, 4.11 fps																	
	4	697.75	23.25	225.00	2.60	7.00	4.59	4.47	0.06	0.82	5.35	80	7.5	0.72	12.7291	0.85	3.72	40.25	
A		Flow conveyed east, under the road, through three (3) 24" RCP's @ 1.5% grade													12.1090	0.85	10.34		106.43

1 Drainage paths (d_x) and areas (AREA) developed from drawings included in this and other attachments to the Site Development Plan

2 Drainage paths (d_x) and times of concentration (t_{cx}) divided and calculated separately for varying slopes through drainage areas

3 Time of concentration (t_c) for each drainage area equivilant to the summation of the times of concentration (t_{cx}) along each drainage path.

4 Velocities obtained from TxDOT Bridge Division Hydraulic Manual and open channel flow calculations

5 Intensity calculated with the formula, $I = b / (t + d)^e$, and cooeficients (b,d,e,C) obtained from TxDOT Bridge Division Hydraulic Manual.

HARDIN COUNTY LANDFILL
SITE DEVELOPMENT PLAN - ATTATCHMENT 6
GROUNDWATER AND SURFACE WATER PROTECTION AND DRAINAGE PLANS

DRAINAGE CALCULATIONS

POINT OF CONC.	RUN-OFF AREA	$d_1^{1,2}$ (feet)	d_2 (feet)	d_3 (feet)	V_1^4 (ft/sec)	V_2 (ft/sec)	V_3 (ft/sec)	t_{c1}^2 (min)	t_{c2} (min)	t_{c3} (min)	$t_c^{2,3}$ (min)	b^5	d^5	e^5	I^5 (in/hr)	C^5	$AREA^1$ (Ac)	Q= C*I*A (cfs)	DITCH FLOW (cfs)
	5	697.75	23.25	438.68	2.60	7.00	3.88	4.47	0.06	1.88	6.41	80	7.5	0.72	12.0182	0.85	7.26	74.16	94.61
	6	697.75	23.25	339.03	2.60	7.00	2.15	4.47	0.06	2.63	7.16	80	7.5	0.72	11.5759	0.85	2.77	27.26	
	5,6	697.75	23.25	777.71	2.60	7.00	3.69	4.47	0.06	3.51	8.04	80	7.5	0.72	11.0976	0.85	10.03		
		"V" Ditch: 22' wide, 2.81' deep, 0.31% average slope Ditch flow: 2.56' deep, 3.69 fps																	
B		Flow discharges out the end of the ditch to the south/southeast of the property																	94.61
	7	336.33			3.75			1.49			1.49	80	7.5	0.72	16.4519	0.85	2.84	39.71	
	8	336.33			3.75			1.49			1.49	80	7.5	0.72	16.4519	0.85	2.73	38.18	
C		Flow discharges to the south of the landfill with limited ditching east and west																77.89	

1 Drainage paths (d_x) and areas (AREA) developed from drawings included in this and other attachments to the Site Development Plan

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3 Time of concentration (t_c) for each drainage area equivilant to the summation of the times of concentration (t_{cx}) along each drainage path

4 Velocities obtained from TxDOT Bridge Division Hydraulic Manual and open channel flow calculations

5 Intensity calculated with the formula, $I = b / (t + d)^e$, and cooeficients (b,d,e,C) obtained from TxDOT Bridge Division Hydraulic Manual

HARDIN COUNTY LANDFILL
SITE DEVELOPMENT PLAN - ATTATCHMENT 6
GROUNDWATER AND SURFACE WATER PROTECTION AND DRAINAGE PLANS

DRAINAGE CALCULATIONS

POINT OF CONC.	RUN-OFF AREA	$d_1^{1,2}$ (feet)	d_2 (feet)	d_3 (feet)	V_1^4 (ft/sec)	V_2 (ft/sec)	V_3 (ft/sec)	t_{c1}^2 (min)	t_{c2} (min)	t_{c3} (min)	$t_c^{2,3}$ (min)	b^5	d^5	e^5	I^5 (in/hr)	C^5	AREA[1] (Ac)	Q= C*I*A (cfs)	DITCH FLOW (cfs)
	1	400.00	909.75		3.75	2.57		1.78	5.90		7.68	80	7.5	0.72	11.2882	0.85	4.55	43.66	
		"V" Ditch: 22' wide, 2.97' deep, 0.20% slope Ditch flow: 2.05' deep, 2.54 fps																	
	12	819.08	23.25		2.30	7.00		5.94	0.06		5.99	80	7.5	0.72	12.2875	0.85	3.49	36.45	72.29
	1,12	400.00	909.75	405.64	3.75	2.99	2.99	1.78	5.07	2.26	9.11	80	7.5	0.72	10.5785	0.85	8.04		
		"V" Ditch: 22' wide, 3.03' deep, 0.20% slope Ditch flow: 2.63' deep, 2.99 fps																	
	11	819.08	23.25	228.13	2.30	7.00	2.60	5.94	0.06	1.46	7.45	80	7.5	0.72	11.4099	0.85	4.25	41.22	108.45
	1,11,12	400.00	909.75	633.77	3.75	3.31	3.31	1.78	4.58	3.19	9.55	80	7.5	0.72	10.3813	0.85	12.29		
		"V" Ditch: 22' wide, 3.14' deep, 0.20% slope Ditch flow: 3.06' deep, 3.31 fps																	
	10	781.44	23.25	444.91	2.30	7.00	3.09	5.66	0.06	2.40	8.12	80	7.5	0.72	11.0582	0.85	7.73	72.66	158.16
	1,10-12	400.00	909.75	1078.68	3.75	2.61	3.75	1.78	5.81	4.79	12.38	80	7.5	0.72	9.2942	0.85	20.02		
		"V" Ditch: 22' wide, 3.75' deep, 0.20% slope Ditch flow: 3.75' deep, 3.75 fps																	
	9	707.21	23.25	341.21	2.30	7.00	2.41	5.12	0.06	2.36	7.54	80	7.5	0.72	11.3626	0.85	2.81	27.14	171.82
	1,9-12	400.00	909.75	1419.89	3.75	2.61	3.83	1.78	5.81	6.18	13.77	80	7.5	0.72	8.8543	0.85	22.83		
		"V" Ditch: 22' wide, 3.93' deep to crown in road, 0.20% slope Ditch flow: 3.87' deep, 3.83 fps																	
D		Flow discharges out the ditch, into the sedimentation basin, thence south and east																	171.82

BILLY D. SIMS

HARDIN COUNTY LANDFILL
SITE DEVELOPMENT PLAN - ATTACHMENT 6
GROUNDWATER AND SURFACE WATER PROTECTION AND DRAINAGE PLANS

DRAINAGE CALCULATIONS

POINT OF CONC.	RUN-OFF AREA	$d_1^{1,2}$ (feet)	d_2 (feet)	d_3 (feet)	V_1^4 (ft/sec)	V_2 (ft/sec)	V_3 (ft/sec)	t_{c1}^2 (min)	t_{c2} (min)	t_{c3} (min)	$t_c^{2,3}$ (min)	t_c^6 (min)	b^5	d^6	e^5	I^5 (in/hr)	C^5	AREA[1] (Ac)	Q= C*I*A (cfs)	DITCH FLOW (cfs)
	2	400.00	721.00		3.75	3.04		1.78	3.95		5.73	10.00	80	7.5	0.72	10.19	0.85	3.62	31.35	31.35
		"V" Ditch: 22' wide, 1.65' deep, 0.54% slope Ditch flow: 1.24' deep, 3.04 fps																		
	3	697.75	23.25		2.60	7.00		4.47	0.06		4.53	10.00	80	7.5	0.72	10.19	0.85	3.00	25.98	57.33
	2,3			400.00			3.93			1.70	7.43	10.00	80	7.5	0.72	10.19	0.85	6.62		
		"V" Ditch: 22' wide, 2.62' deep, 0.54% slope Ditch flow: 1.86' deep, 3.93 fps																		
	4	697.75	23.25	225.00	2.60	7.00	3.95	4.47	0.06	0.95	5.48	10.00	80	7.5	0.72	10.19	0.85	3.72	32.22	32.22
		"V" Ditch: 22' wide, 2.62' deep, 0.80% slope Ditch flow: 1.39' deep, 3.95 fps																		
A	2,3,4	Flow conveyed east, under the road, through three (3) 27" RCP's @ 1.5% grade														10.19	0.85	10.34		89.55

1 Drainage paths (d_x) and areas (AREA) developed from drawings included in this and other attachments to the Site Development Plan

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4 Velocities obtained from TxDOT Bridge Division Hydraulic Manual and open channel flow calculations

5 Intensity calculated with the formula, $I = b / (t + d)^e$, and cooeficients (b,d,e,C) obtained from TxDOT Bridge Division Hydraulic Manual

6 Time of concentration (t_c)assumed to be a minimum time of 10 minutes.

HARDIN COUNTY LANDFILL
SITE DEVELOPMENT PLAN - ATTACHMENT 6
GROUNDWATER AND SURFACE WATER PROTECTION AND DRAINAGE PLANS

DRAINAGE CALCULATIONS

POINT OF CONC.	RUN-OFF AREA	$d_1^{1,2}$ (feet)	d_2 (feet)	d_3 (feet)	V_1^4 (ft/sec)	V_2 (ft/sec)	V_3 (ft/sec)	t_{c1}^2 (min)	t_{c2} (min)	t_{c3} (min)	$t_c^{2,3}$ (min)	t_c^6 (min)	b^5	d^5	e^5	I^5 (in/hr)	C^5	AREA^1 (Ac)	Q= C*I*A (cfs)	DITCH FLOW (cfs)
	1	400.00	909.75		3.75	2.55		1.78	5.95		7.72	10.00	80	7.5	0.72	10.19	0.85	4.55	39.40	39.40
	"V" Ditch: 22' wide, 2.97' deep, 0.20% slope Ditch flow: 2.06' deep, 2.55 fps																			
	12	819.08	23.25		2.30	7.00		5.94	0.06		5.99	10.00	80	7.5	0.72	10.19	0.85	3.49	30.22	69.56
	1,12			405.64			2.94			2.30	10.02	10.02	80	7.5	0.72	10.18	0.85	8.04		
	"V" Ditch: 22' wide, 3.03' deep, 0.20% slope Ditch flow: 2.55' deep, 2.94 fps																			
	11	819.08	23.25	228.13	2.30	7.00	2.60	5.94	0.06	1.46	7.45	10.00	80	7.5	0.72	10.19	0.85	4.25	36.81	101.50
	1,11,12			228.13			3.25			1.17	11.19	11.19	80	7.5	0.72	9.72	0.85	12.29		
	"V" Ditch: 22' wide, 3.14' deep, 0.20% slope Ditch flow: 2.99' deep, 3.25 fps																			
	10	781.44	23.25	444.91	2.30	7.00	3.09	5.66	0.06	2.40	8.12	10.00	80	7.5	0.72	10.19	0.85	7.73	66.94	153.70
	1,10-12			444.91			3.72			1.99	13.19	13.19	80	7.5	0.72	9.03	0.85	20.02		
	"V" Ditch: 22' wide, 3.75' deep, 0.20% slope Ditch flow: 3.71' deep, 3.72 fps																			
	9	707.21	23.25	341.21	2.30	7.00	2.41	5.12	0.06	2.36	7.54	10.00	80	7.5	0.72	10.19	0.85	2.81	24.33	166.68
	1,9-12			341.21			3.80			1.50	14.68	14.68	80	7.5	0.72	8.59	0.85	22.83		
	"V" Ditch: 22' wide, 3.93' deep to crown in road, 0.20% slope Ditch flow: 3.82' deep, 3.80 fps																			
D	Flow discharges out the ditch, into the sedimentation basin, thence south and east																			166.68

HARDIN COUNTY LANDFILL
SITE DEVELOPMENT PLAN - ATTACHMENT 6
GROUNDWATER AND SURFACE WATER PROTECTION AND DRAINAGE PLANS

DRAINAGE CALCULATIONS

POINT OF CONC.	RUN-OFF AREA	d_1[1,2] (feet)	d_2 (feet)	d_3 (feet)	V_1[4] (ft/sec)	V_2 (ft/sec)	V_3 (ft/sec)	t_{c1}[2] (min)	t_{c2} (min)	t_{c3} (min)	t_c[2,3] (min)	t_c[6] (min)	b[5]	d[5]	e[5]	I[5] (in/hr)	C[5]	AREA[1] (Ac)	Q= C*I*A (cfs)	DITCH FLOW (cfs)
	13	58.00	8.00	760.00	3.75	7.00	1.93	0.26	0.02	6.56	6.84	10.00	80	7.5	0.72	10.19	0.85	1.05	9.09	9.09
		"V" Ditch: 22' wide, 2.26' deep, 0.30% slope Ditch flow: 0.98' deep, 1.93 fps																		
	15	58.00	8.00	780.00	4.20	7.00	1.59	0.23	0.02	8.18	8.43	10.00	80	7.5	0.72	10.19	0.85	1.07	9.27	9.27
		"V" Ditch: 11' wide, 1.12' deep, 0.20% slope Ditch flow: 1.09' deep, 1.59 fps																		
	16	58.00	8.00	66.00	3.90	7.00	1.20	0.25	0.02	0.92	1.18	10.00	80	7.5	0.72	10.19	0.85	0.10	0.87	
	15,16			132.00			1.96				1.12	9.55	10.00	80	7.5	0.72	10.19	0.85	1.17	10.13	10.13
		"V" Ditch: 22' wide, 2.26' deep, 0.30% slope Ditch flow: 1.46' deep, 1.96 fps																		
	13,15,16	Flow conveyed east, under the road, through three (3) 18" RCP's @ 0.30% grade "V" Ditch: 20' wide, 1.70' deep, 0.30% slope Ditch flow: 1.26' deep, 2.28 fps														10.19	0.85	2.22		19.23
E		Ditch flows continue east to property line																		19.23
	14	58.00	8.00	66.00	3.90	7.00	1.16	0.25	0.02	0.95	1.22	10.00	80	7.5	0.72	10.19	0.85	0.10	0.87	
F		Sheet flow allowed to drain freely north and west of the property																		0.87

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4 Velocities obtained from TxDOT Bridge Division Hydraulic Manual and open channel flow calculations
5 Intensity calculated with the formula, $I = b / (t + d)^e$, and cooeficients (b,d,e,C) obtained from TxDOT Bridge Division Hydraulic Manual
6 Time of concentration (t_c)assumed to be a minimum time of 10 minutes.

BILLY D. SIMS
9/30/16

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              Triangular Channel Analysis & Design
                 Open Channel - Uniform flow

Worksheet Name: A1

Comment: DITCH ALONG NORTH SIDE OF AREA 2

Solve For Depth

Given Input Data:
        Left Side Slope..      6.67:1 (H:V)
        Right Side Slope.      6.67:1 (H:V)
        Manning's n......      0.026
        Channel Slope....      0.0054 ft/ft
        Discharge........     31.35 cfs

Computed Results:
        Depth............      1.24 ft
        Velocity.........      3.04 fps
        Flow Area........     10.32 sf
        Flow Top Width...     16.59 ft
        Wetted Perimeter.     16.78 ft
        Critical Depth...      1.07 ft
        Critical Slope...      0.0123 ft/ft
        Froude Number....      0.68 (flow is Subcritical)




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           Triangular Channel Analysis & Design
              Open Channel - Uniform flow


Worksheet Name: A2

Comment: DITCH ALONG EAST SIDE OF AREA 3

Solve For Depth

Given Input Data:
         Left Side Slope..     4.20:1 (H:V)
         Right Side Slope.     4.20:1 (H:V)
         Manning's n......     0.026
         Channel Slope....     0.0054 ft/ft
         Discharge........    57.33 cfs

Computed Results:
         Depth............     1.86 ft
         Velocity.........     3.93 fps
         Flow Area........    14.58 sf
         Flow Top Width...    15.65 ft
         Wetted Perimeter.    16.09 ft
         Critical Depth...     1.63 ft
         Critical Slope...     0.0109 ft/ft
         Froude Number....     0.72 (flow is Subcritical)



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              Triangular Channel Analysis & Design
                 Open Channel - Uniform flow

Worksheet Name: A3

Comment: DITCH ALONG EAST SIDE OF AREA 4

Solve For Depth

Given Input Data:
        Left Side Slope..      4.20:1 (H:V)
        Right Side Slope.      4.20:1 (H:V)
        Manning's n......      0.026
        Channel Slope....      0.0080 ft/ft
        Discharge........     32.22 cfs

Computed Results:
        Depth............      1.39 ft
        Velocity.........      3.95 fps
        Flow Area........      8.17 sf
        Flow Top Width...     11.71 ft
        Wetted Perimeter.     12.04 ft
        Critical Depth...      1.30 ft
        Critical Slope...      0.0118 ft/ft
        Froude Number....      0.83 (flow is Subcritical)




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        Triangular Channel Analysis & Design
           Open Channel - Uniform flow


Worksheet Name: B1

Comment: DITCH ALONG EAST SIDE OF AREA 5

Solve For Depth

Given Input Data:
        Left Side Slope..     4.38:1 (H:V)
        Right Side Slope.     4.38:1 (H:V)
        Manning's n......     0.026
        Channel Slope....     0.0045 ft/ft
        Discharge........    62.87 cfs

Computed Results:
        Depth............     1.96 ft
        Velocity.........     3.72 fps
        Flow Area........    16.88 sf
        Flow Top Width...    17.20 ft
        Wetted Perimeter.    17.64 ft
        Critical Depth...     1.67 ft
        Critical Slope...     0.0108 ft/ft
        Froude Number....     0.66 (flow is Subcritical)




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            Triangular Channel Analysis & Design
               Open Channel - Uniform flow

Worksheet Name: B2

Comment: DITCH ALONG EAST SIDE OF AREA 6

Solve For Depth

Given Input Data:
          Left Side Slope..      3.91:1 (H:V)
          Right Side Slope.      3.91:1 (H:V)
          Manning's n......      0.026
          Channel Slope....      0.0031 ft/ft
          Discharge........     86.86 cfs

Computed Results:
          Depth............      2.48 ft
          Velocity.........      3.60 fps
          Flow Area........     24.13 sf
          Flow Top Width...     19.43 ft
          Wetted Perimeter.     20.05 ft
          Critical Depth...      1.98 ft
          Critical Slope...      0.0103 ft/ft
          Froude Number....      0.57 (flow is Subcritical)




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             Triangular Channel Analysis & Design
                Open Channel - Uniform flow

Worksheet Name: D1-ROUTED

Comment: DITCH FROM AREA 1 TO MEET AREA 12

Solve For Depth

Given Input Data:
          Left Side Slope..      3.63:1 (H:V)
          Right Side Slope.      3.63:1 (H:V)
          Manning's n......      0.026
          Channel Slope....      0.0020 ft/ft
          Discharge........     39.40 cfs

Computed Results:
          Depth............      2.06 ft
          Velocity.........      2.55 fps
          Flow Area........     15.47 sf
          Flow Top Width...     14.99 ft
          Wetted Perimeter.     15.54 ft
          Critical Depth...      1.49 ft
          Critical Slope...      0.0114 ft/ft
          Froude Number....      0.44 (flow is Subcritical)




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              Triangular Channel Analysis & Design
                 Open Channel - Uniform flow

Worksheet Name: D1A-ROUTED

Comment: DITCH TRU AREA 12 SERVING 1 & 12

Solve For Depth

Given Input Data:
          Left Side Slope..      3.63:1 (H:V)
          Right Side Slope.      3.63:1 (H:V)
          Manning's n......      0.026
          Channel Slope....      0.0020 ft/ft
          Discharge........     69.56 cfs

Computed Results:
          Depth............      2.55 ft
          Velocity.........      2.94 fps
          Flow Area........     23.69 sf
          Flow Top Width...     18.55 ft
          Wetted Perimeter.     19.24 ft
          Critical Depth...      1.87 ft
          Critical Slope...      0.0106 ft/ft
          Froude Number....      0.46 (flow is Subcritical)




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                Triangular Channel Analysis & Design
                   Open Channel - Uniform flow

Worksheet Name: D2-ROUTED

Comment: ROUTED FLOW FROM AREAS 1, 11, AND 12

Solve For Depth

Given Input Data:
        Left Side Slope..      3.50:1 (H:V)
        Right Side Slope.      3.50:1 (H:V)
        Manning's n......      0.026
        Channel Slope....      0.0020 ft/ft
        Discharge........    101.50 cfs

Computed Results:
        Depth............      2.99 ft
        Velocity.........      3.25 fps
        Flow Area........     31.21 sf
        Flow Top Width...     20.90 ft
        Wetted Perimeter.     21.74 ft
        Critical Depth...      2.21 ft
        Critical Slope...      0.0101 ft/ft
        Froude Number....      0.47 (flow is Subcritical)




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                Triangular Channel Analysis & Design
                   Open Channel - Uniform flow

Worksheet Name: D3-ROUTED

Comment: ROUTED FLOW FROM AREAS 1, 12, 11, AND 10

Solve For Depth

Given Input Data:
        Left Side Slope..      3.00:1 (H:V)
        Right Side Slope.      3.00:1 (H:V)
        Manning's n......      0.026
        Channel Slope....      0.0020 ft/ft
        Discharge........    153.70 cfs

Computed Results:
        Depth............      3.71 ft
        Velocity.........      3.72 fps
        Flow Area........     41.26 sf
        Flow Top Width...     22.25 ft
        Wetted Perimeter.     23.46 ft
        Critical Depth...      2.77 ft
        Critical Slope...      0.0095 ft/ft
        Froude Number....      0.48 (flow is Subcritical)




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              Triangular Channel Analysis & Design
                 Open Channel - Uniform flow

Worksheet Name: D4-ROUTED

Comment: FLOW ROUTED FROM AREAS 1,12,11,10,9

Solve For Depth

Given Input Data:
         Left Side Slope..      3.00:1 (H:V)
         Right Side Slope.      3.00:1 (H:V)
         Manning's n......      0.026
         Channel Slope....      0.0020 ft/ft
         Discharge........    166.68 cfs

Computed Results:
         Depth............      3.82 ft
         Velocity.........      3.80 fps
         Flow Area........     43.85 sf
         Flow Top Width...     22.94 ft
         Wetted Perimeter.     24.18 ft
         Critical Depth...      2.86 ft
         Critical Slope...      0.0094 ft/ft
         Froude Number....      0.48 (flow is Subcritical)




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                Triangular Channel Analysis & Design
                   Open Channel - Uniform flow

Worksheet Name: 13

Comment: RUNOFF FROM AREA 13

Solve For Depth

Given Input Data:
        Left Side Slope..     4.87:1 (H:V)
        Right Side Slope.     4.87:1 (H:V)
        Manning's n......     0.026
        Channel Slope....     0.0030 ft/ft
        Discharge........     9.09 cfs

Computed Results:
        Depth............     0.98 ft
        Velocity.........     1.93 fps
        Flow Area........     4.72 sf
        Flow Top Width...     9.59 ft
        Wetted Perimeter.     9.79 ft
        Critical Depth...     0.74 ft
        Critical Slope...     0.0141 ft/ft
        Froude Number....     0.48 (flow is Subcritical)




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            Triangular Channel Analysis & Design
               Open Channel - Uniform flow

Worksheet Name: 15

Comment: RUNOFF FROM AREA 15

Solve For Depth

Given Input Data:
         Left Side Slope..      4.92:1 (H:V)
         Right Side Slope.      4.92:1 (H:V)
         Manning's n......      0.026
         Channel Slope....      0.0018 ft/ft
         Discharge........      9.27 cfs

Computed Results:
         Depth............      1.09 ft
         Velocity.........      1.59 fps
         Flow Area........      5.82 sf
         Flow Top Width...     10.70 ft
         Wetted Perimeter.     10.92 ft
         Critical Depth...      0.74 ft
         Critical Slope...      0.0141 ft/ft
         Froude Number....      0.38 (flow is Subcritical)




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          Triangular Channel Analysis & Design
             Open Channel - Uniform flow

Worksheet Name: 15 & 16 ROUTED

Comment: DITCH IN FRONT OF AREA 16 FLOW FROM 15 & 16

Solve For Depth

Given Input Data:
          Left Side Slope..      2.43:1 (H:V)
          Right Side Slope.      2.43:1 (H:V)
          Manning's n......      0.026
          Channel Slope....      0.0020 ft/ft
          Discharge........     10.13 cfs

Computed Results:
          Depth............      1.46 ft
          Velocity.........      1.96 fps
          Flow Area........      5.16 sf
          Flow Top Width...      7.08 ft
          Wetted Perimeter.      7.66 ft
          Critical Depth...      1.02 ft
          Critical Slope...      0.0137 ft/ft
          Froude Number....      0.41 (flow is Subcritical)




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                 Triangular Channel Analysis & Design
                    Open Channel - Uniform flow

Worksheet Name: 13,15,16

Comment: FLOW FROM 13,15,16 TO DITCH FLOWING EAST

Solve For Depth

Given Input Data:
         Left Side Slope..     5.30:1 (H:V)
         Right Side Slope.     5.30:1 (H:V)
         Manning's n......     0.026
         Channel Slope....     0.0030 ft/ft
         Discharge........    19.23 cfs

Computed Results:
         Depth............     1.26 ft
         Velocity.........     2.28 fps
         Flow Area........     8.45 sf
         Flow Top Width...    13.38 ft
         Wetted Perimeter.    13.62 ft
         Critical Depth...     0.96 ft
         Critical Slope...     0.0129 ft/ft
         Froude Number....     0.51 (flow is Subcritical)





   Open Channel Flow Module, Version 3.41 (c) 1991
   Haestad Methods, Inc. * 37 Brookside Rd * Waterbury, Ct 06708
```

BILLY D. SIMS
63016

```
CURRENT DATE: 09-10-1997                                          FILE DATE: 09-02-1997
CURRENT TIME: 13:40:53                                            FILE NAME: HARDINA

                                   CULVERT AREA A
                               FHWA CULVERT ANALYSIS
                                 HY-8, VERSION 3.2

C              SITE DATA                          CULVERT SHAPE, MATERIAL, INLET
U
L    INLET    OUTLET   CULVERT   BARRELS
V    ELEV.    ELEV.    LENGTH    SHAPE          SPAN    RISE   MANNING    INLET
     (FT)     (FT)     (FT)      MATERIAL       (FT)    (FT)      n       TYPE
1    74.67    74.16     27.98    3 RCP          2.00    2.00    .012    IMPR SLT
2
3
4
5
6

SUMMARY OF CULVERT FLOWS (CFS)          FILE: HARDINA            DATE: 09-02-1997

 ELEV (FT)    TOTAL      1      2      3      4      5      6   ROADWAY  ITR
   74.67         0       0      0      0      0      0      0        0    1
   75.59        10      10      0      0      0      0      0        0    1
   75.79        20      20      0      0      0      0      0        0    1
   76.02        30      30      0      0      0      0      0        0    1
   76.27        40      40      0      0      0      0      0        0    1
   76.55        50      50      0      0      0      0      0        0    1
   76.87        60      60      0      0      0      0      0        0    1
   77.21        70      70      0      0      0      0      0        0    1
   77.60        80      80      0      0      0      0      0        0    1
   78.01        90      90      0      0      0      0      0        0    1
   78.18       100      93      0      0      0      0      0        6   21
   78.14        92      92      0      0      0      0      0   OVERTOPPING

SUMMARY OF ITERATIVE SOLUTION ERRORS     FILE: HARDINA          DATE: 09-02-1997

     HEAD          HEAD          TOTAL          FLOW          % FLOW
   ELEV(FT)      ERROR(FT)     FLOW(CFS)     ERROR(CFS)       ERROR
    74.67          0.00            0             0            0.00
    75.59          0.00           10             0            0.00
    75.79          0.00           20             0            0.00
    76.02          0.00           30             0            0.00
    76.27          0.00           40             0            0.00
    76.55          0.00           50             0            0.00
    76.87          0.00           60             0            0.00
    77.21          0.00           70             0            0.00
    77.60          0.00           80             0            0.00
    78.01          0.00           90             0            0.00
    78.18         -0.00          100             1            0.87
<1> TOLERANCE (FT) = 0.010                       <2> TOLERANCE (%) = 1.000
```

```
                                                    FILE DATE: 09-02-1997
CURRENT DATE: 09-10-1997                            FILE NAME: HARDINA
CURRENT TIME: 13:40:53

                              CULVERT #  1
 ___________________________________________________________________________
                  PERFORMANCE CURVE FOR  3  BARREL(S)

                                                                          VO
                                      OCH   FLOW    CCE     FCE     TCE  (fps)
  Q       HWE      TWE      ICH      (ft)   TYPE    (ft)    (ft)    (ft)
 (cfs)    (ft)     (ft)     (ft)
   0     74.67    74.06     0.00    -0.51   0-NF   74.67   74.67   74.57   0.00
  10     75.59    74.36     0.58     0.92   6-FF   75.11   75.21   75.25   3.91
  20     75.79    74.45     0.94     1.12   6-FF   75.37   75.52   75.61   4.81
  30     76.02    74.51     1.27     1.35   6-FF   75.59   75.79   75.94   5.49
  40     76.27    74.56     1.56     1.60   6-FF   75.79   76.02   76.23   6.12
  50     76.55    74.60     1.83     1.88   6-FF   75.96   76.24   76.50   6.77
  60     76.87    74.64     2.09     2.20   6-FF   76.13   76.44   76.76   7.40
  70     77.21    74.68     2.33     2.54   6-FF   76.29   76.63   77.00   8.21
  80     77.60    74.71     2.57     2.93   6-FF   76.44   76.83   77.24   8.95
  90     78.01    74.74     2.81     3.34   6-FF   76.58   77.05   77.48   9.77
  93     78.17    74.77     2.89     3.50   6-FF   76.63   77.14   77.56  10.05

     El. inlet face invert       74.67 ft   El. outlet invert      74.16 ft
     El. inlet throat invert     74.57 ft   El. inlet crest        74.67 ft
 ___________________________________________________________________________

***** SITE DATA ***** EMBANKMENT TOE ***************
      UPSTREAM STATION (FT)                    140.00
      UPSTREAM ELEVATION (FT)                   74.67
      UPSTREAM EMBANKMENT SLOPE (X:1)            3.00
      DOWMSTREAM STATION (FT)                  100.00
      DOWNSTREAM ELEVATION (FT)                 74.07
      DOWNSTREAM EMBANKMENT SLOPE (X:1)          3.00

***** CULVERT DATA SUMMARY ************************
      BARREL SHAPE           CIRCULAR
      BARREL DIAMETER          2.00 FT
      BARREL MATERIAL        CONCRETE
      BARREL MANNING'S N     0.012
      INLET TYPE             IMPR SLT
      INLET EDGE AND WALL    SQUARE EDGE TOP (26-90 DEG WINGWALL)
      INLET DEPRESSION       FALL INCLUDED

***** SLOPE-TAPERED IMPROVED INLET ***************
      FACE WIDTH                                3.00 FT
      SIDE TAPER (4:1 TO 6:1)  (X:1)            1.01
      FALL SLOPE (2:1 TO 3:1)  (X:1)            2.00
      FALL                                      0.10 FT
      MITERED FACE              (Y/N)            Y
      FACE-CREST LENGTH IF MITERED              4.00 FT
 ___________________________________________________________________________
```

```
CURRENT DATE: 09-10-1997                             FILE DATE: 09-02-1997
CURRENT TIME: 13:40:53                               FILE NAME: HARDINA

                                 TAILWATER
          TAILWATER RATING CURVE

        FLOW(CFS)        W.S.E.(FT)
               0           74.06
              10           74.36
              20           74.45
              30           74.51
              40           74.56
              50           74.60
              60           74.64
              70           74.68
              80           74.71
              90           74.74
             100           74.77

                           ROADWAY OVERTOPPING DATA

    ROADWAY SURFACE                               GRAVEL
    EMBANKMENT TOP WIDTH (FT)                      20.00
    CREST LENGTH (FT)                             350.00
    OVERTOPPING CREST ELEVATION (FT)               78.14
```

```
              Triangular Channel Analysis & Design
                 Open Channel - Uniform flow

    Worksheet Name: area a tailwater

    Description: tailwater curve for area a 24' culverts

    Solve For Discharge

    Given Constant Data;

      Z-Left............ 200.00
      Z-Right........... 400.00
      Mannings 'n'.......   0.050
      Channel Slope......   0.0020

ariable Input Data        Minimum      Maximum     Increment By
==================        =======      =======     ============
Channel Depth              0.00         1.00            0.10
```

```
     Open Channel Flow Module, Version 3.41 (c)
     Haestad Methods, Inc. * 37 Brookside Rd * Waterbury, Ct 06708
```

```
                                              VARIABLE COMPUTED COMPUTED
                                              ===========================
 Z-Left    Z-Right   Mannings Channel  Channel   Channel Velocity
  (H:V)     (H:V)      'n'     Slope    Depth   Discharge (fps)
                                ft/ft     ft       cfs
=======================================================================
  Unable to compute this instance.
200.00    400.00      0.050   0.0020     0.10        0.54      0.18
200.00    400.00      0.050   0.0020     0.20        3.44      0.29
200.00    400.00      0.050   0.0020     0.30       10.13      0.38
200.00    400.00      0.050   0.0020     0.40       21.82      0.45
200.00    400.00      0.050   0.0020     0.50       39.56      0.53
200.00    400.00      0.050   0.0020     0.60       64.33      0.60
200.00    400.00      0.050   0.0020     0.70       97.03      0.66
200.00    400.00      0.050   0.0020     0.80      138.54      0.72
200.00    400.00      0.050   0.0020     0.90      189.66      0.78
200.00    400.00      0.050   0.0020     1.00      251.19      0.84
200.00    400.00      0.050   0.0020     1.10      323.87      0.89
```

```
      Open Channel Flow Module, Version 3.41 (c)
      Haestad Methods, Inc. * 37 Brookside Rd * Waterbury, Ct 06708
```

```
CURRENT DATE: 09-09-1997                                  FILE DATE: 09-02-1997
CURRENT TIME: 15:26:28                                    FILE NAME: HARDINB

                              CULVERT AREAS C,& D
                            FHWA CULVERT ANALYSIS
                             HY-8, VERSION 3.2

 C             SITE DATA                         CULVERT SHAPE, MATERIAL, INLET
 U
 L   INLET    OUTLET   CULVERT    BARRELS
 V   ELEV.     ELEV.   LENGTH     SHAPE          SPAN    RISE   MANNING    INLET
     (FT)      (FT)     (FT)      MATERIAL       (FT)    (FT)     n        TYPE
 1   79.50     79.30    47.00     3 RCP          1.50    1.50    .012    IMPR SLT
 2
 3
 4
 5
 6

SUMMARY OF CULVERT FLOWS (CFS)          FILE: HARDINB          DATE: 09-02-1997

 ELEV (FT)   TOTAL      1       2       3       4       5       6  ROADWAY ITR
   79.50       0        0       0       0       0       0       0        0  1
   80.01       3        3       0       0       0       0       0        0  1
   80.23       6        6       0       0       0       0       0        0  1
   80.41       9        9       0       0       0       0       0        0  1
   80.57      12       12       0       0       0       0       0        0  1
   80.72      15       15       0       0       0       0       0        0  1
   80.87      18       18       0       0       0       0       0        0  1
   80.95      20       20       0       0       0       0       0        0  1
   81.13      24       24       0       0       0       0       0        0  1
   81.30      27       27       0       0       0       0       0        0  1
   81.54      30       30       0       0       0       0       0        0  1
   82.15      38       38       0       0       0       0       0 OVERTOPPING

SUMMARY OF ITERATIVE SOLUTION ERRORS   FILE: HARDINB          DATE: 09-02-1997

       HEAD            HEAD           TOTAL          FLOW          % FLOW
     ELEV(FT)        ERROR(FT)      FLOW(CFS)      ERROR(CFS)       ERROR
      79.50            0.00             0              0             0.00
      80.01            0.00             3              0             0.00
      80.23            0.00             6              0             0.00
      80.41            0.00             9              0             0.00
      80.57            0.00            12              0             0.00
      80.72            0.00            15              0             0.00
      80.87            0.00            18              0             0.00
      80.95            0.00            20              0             0.00
      81.13            0.00            24              0             0.00
      81.30            0.00            27              0             0.00
      81.54            0.00            30              0             0.00
 <1> TOLERANCE (FT) = 0.010                         <2> TOLERANCE (%) = 1.000
```

```
CURRENT DATE: 09-09-1997                                   FILE DATE: 09-02-1997
CURRENT TIME: 15:26:28                                     FILE NAME: HARDINB

                                       CULVERT # 1
------------------------------------------------------------------------------------
                           PERFORMANCE CURVE FOR  3  BARREL(S)

   Q       HWE      TWE      ICH      OCH   FLOW     CCE      FCE      TCE      VO
 (cfs)    (ft)     (ft)     (ft)     (ft)   TYPE     (ft)     (ft)     (ft)    (fps)
    0    79.50    79.30     0.00    -0.20   0-NF    79.50    79.50    79.40     0.00
    3    80.01    79.43     0.29     0.50   2-M2    79.74    79.74    79.80     2.97
    6    80.23    79.50     0.47     0.73   2-M2    79.88    79.88    79.98     3.59
    9    80.41    79.56     0.64     0.91   2-M2    80.00    80.00    80.15     4.06
   12    80.57    79.61     0.80     1.07   2-M2    80.11    80.11    80.30     4.45
   15    80.72    79.65     0.95     1.21   2-M2    80.21    80.21    80.45     4.80
   18    80.87    79.69     1.08     1.37   2-M2    80.30    80.30    80.59     5.12
   20    80.95    79.71     1.16     1.45   2-M2    80.35    80.35    80.66     5.34
   24    81.13    79.76     1.34     1.63   2-M2    80.46    80.46    80.84     5.79
   27    81.30    79.80     1.46     1.80   2-M2    80.54    80.54    80.96     6.17
   30    81.54    79.83     1.57     2.04   7-FF    80.62    80.62    81.07     6.51

        El. inlet face invert      79.50 ft   El. outlet invert      79.30 ft
        El. inlet throat invert    79.40 ft   El. inlet crest        79.50 ft
------------------------------------------------------------------------------------

***** SITE DATA ***** EMBANKMENT TOE **************
      UPSTREAM STATION (FT)                        160.50
      UPSTREAM ELEVATION (FT)                       79.50
      UPSTREAM EMBANKMENT SLOPE (X:1)                3.00
      DOWMSTREAM STATION (FT)                      104.50
      DOWNSTREAM ELEVATION (FT)                     79.29
      DOWNSTREAM EMBANKMENT SLOPE (X:1)              3.00

***** CULVERT DATA SUMMARY ************************
      BARREL SHAPE         CIRCULAR
      BARREL DIAMETER        1.50 FT
      BARREL MATERIAL      CONCRETE
      BARREL MANNING'S N   0.012
      INLET TYPE           IMPR SLT
      INLET EDGE AND WALL  SQUARE EDGE TOP (26-90 DEG WINGWALL)
      INLET DEPRESSION     FALL INCLUDED

***** SLOPE-TAPERED IMPROVED INLET ***************
      FACE WIDTH                                 3.00 FT
      SIDE TAPER (4:1 TO 6:1) (X:1)              1.68
      FALL SLOPE (2:1 TO 3:1) (X:1)              2.00
      FALL                                       0.10 FT
      MITERED FACE               (Y/N)           Y
      FACE-CREST LENGTH IF MITERED               3.00 FT
------------------------------------------------------------------------------------
```

```
URRENT DATE: 09-09-1997                               FILE DATE: 09-02-1997
CURRENT TIME: 15:26:28                                FILE NAME: HARDINB

                                  TAILWATER

******* REGULAR CHANNEL CROSS SECTION ****************
        BOTTOM WIDTH (FT)                              31.00
        SIDE SLOPE  H/V (X:1)                           6.0
        CHANNEL SLOPE V/H (FT/FT)                       0.003
        MANNING'S N (.01-0.1)                           0.030
        CHANNEL INVERT ELEVATION (FT)                  79.30
        CULVERT NO.1 OUTLET INVERT ELEVATION           79.30 FT

******* UNIFORM FLOW RATING CURVE FOR DOWNSTREAM CHANNEL

            FLOW     W.S.E.  FROUDE   VEL.    SHEAR
            (CFS)    (FT)    NUMBER   (FPS)   (PSF)
             0.00    79.30   0.000    0.00    0.00
             3.00    79.43   0.336    0.70    0.03
             6.00    79.50   0.357    0.91    0.04
             9.00    79.56   0.370    1.07    0.05
            12.00    79.61   0.378    1.19    0.06
            15.00    79.65   0.385    1.29    0.07
            18.00    79.69   0.390    1.38    0.07
            19.80    79.71   0.393    1.43    0.08
            24.00    79.76   0.398    1.54    0.09
            27.00    79.80   0.401    1.60    0.09
            30.00    79.83   0.404    1.67    0.10

                           ROADWAY OVERTOPPING DATA

       ROADWAY SURFACE                               PAVED
       EMBANKMENT TOP WIDTH (FT)                     40.00
       CREST LENGTH (FT)                             30.00
       OVERTOPPING CREST ELEVATION (FT)              82.15
```

```
CURRENT DATE: 09-09-1997                              FILE DATE: 09-02-1997
CURRENT TIME: 15:38:43                                FILE NAME: HARDINB1

                              CULVERT AREAS C & D
                            FHWA CULVERT ANALYSIS
                              HY-8, VERSION 3.2

 C              SITE DATA                    CULVERT SHAPE, MATERIAL, INLET
 U
 L    INLET    OUTLET   CULVERT   BARRELS
 V    ELEV.    ELEV.    LENGTH    SHAPE         SPAN    RISE   MANNING    INLET
      (FT)     (FT)     (FT)      MATERIAL      (FT)    (FT)      n       TYPE
 1    79.79    79.72    15.00     3 RCP         1.50    1.50    .012    IMPR SLT
 2
 3
 4
 5
 6

SUMMARY OF CULVERT FLOWS (CFS)          FILE: HARDINB1          DATE: 09-02-1997

ELEV (FT)    TOTAL       1       2       3       4       5       6    ROADWAY ITR
   79.79         0       0       0       0       0       0       0          0   1
   80.29         3       3       0       0       0       0       0          0   1
   80.52         6       6       0       0       0       0       0          0   1
   80.70         9       9       0       0       0       0       0          0   1
   80.86        12      12       0       0       0       0       0          0   1
   81.00        15      15       0       0       0       0       0          0   1
   81.14        18      18       0       0       0       0       0          0   1
   81.24        20      20       0       0       0       0       0          0   1
   81.42        24      24       0       0       0       0       0          0   1
   81.55        27      27       0       0       0       0       0          0   1
   81.68        30      30       0       0       0       0       0          0   1
   82.15        38      38       0       0       0       0       0     OVERTOPPING

SUMMARY OF ITERATIVE SOLUTION ERRORS     FILE:HARDINB1          DATE: 09-02-1997

        HEAD            HEAD            TOTAL          FLOW           % FLOW
      ELEV(FT)       ERROR(FT)       FLOW(CFS)      ERROR(CFS)        ERROR
       79.79            0.00              0             0              0.00
       80.29            0.00              3             0              0.00
       80.52            0.00              6             0              0.00
       80.70            0.00              9             0              0.00
       80.86            0.00             12             0              0.00
       81.00            0.00             15             0              0.00
       81.14            0.00             18             0              0.00
       81.24            0.00             20             0              0.00
       81.42            0.00             24             0              0.00
       81.55            0.00             27             0              0.00
       81.68            0.00             30             0              0.00
<1> TOLERANCE (FT) = 0.010                          <2> TOLERANCE (%) = 1.000
```

```
CURRENT DATE: 09-09-1997                            FILE DATE: 09-02-1997
CURRENT TIME: 15:38:43                              FILE NAME: HARDINB1

                              CULVERT #  1

______________________________________________________________________________
PERFORMANCE CURVE FOR  3  BARREL(S)

  Q       HWE      TWE      ICH      OCH   FLOW     CCE      FCE      TCE      VO
(cfs)     (ft)     (ft)     (ft)     (ft)  TYPE     (ft)     (ft)     (ft)    (fps)
  0      79.79    79.72     0.00    -0.07  0-NF    79.79    79.79    79.69    0.00
  3      80.29    79.85     0.29     0.50  2-M2    80.03    80.03    80.08    2.97
  6      80.52    79.92     0.47     0.73  2-M2    80.17    80.17    80.26    3.59
  9      80.70    79.98     0.64     0.91  2-M2    80.29    80.29    80.43    4.06
 12      80.86    80.03     0.80     1.07  2-M2    80.40    80.40    80.59    4.45
 15      81.00    80.07     0.95     1.21  2-M2    80.49    80.49    80.74    4.80
 18      81.14    80.11     1.08     1.35  2-M2    80.58    80.58    80.87    5.12
 20      81.24    80.13     1.16     1.45  2-M2    80.64    80.64    80.95    5.34
 24      81.42    80.18     1.34     1.63  2-M2    80.75    80.75    81.13    5.79
 27      81.55    80.22     1.46     1.76  2-M2    80.83    80.83    81.25    6.17
 30      81.68    80.25     1.57     1.89  2-M2    80.91    80.91    81.36    6.51

        El. inlet face invert      79.79 ft   El. outlet invert    79.72 ft
        El. inlet throat invert    79.69 ft   El. inlet crest      79.79 ft
______________________________________________________________________________

______________________________________________________________________________
*****  SITE DATA ***** EMBANKMENT TOE **************
       UPSTREAM STATION (FT)                     124.00
       UPSTREAM ELEVATION (FT)                    79.79
       UPSTREAM EMBANKMENT SLOPE (X:1)             3.00
       DOWMSTREAM STATION (FT)                   100.00
       DOWNSTREAM ELEVATION (FT)                  79.70
       DOWNSTREAM EMBANKMENT SLOPE (X:1)           3.00

*****  CULVERT DATA SUMMARY ************************
       BARREL SHAPE          CIRCULAR
       BARREL DIAMETER         1.50 FT
       BARREL MATERIAL       CONCRETE
       BARREL MANNING'S N    0.012
       INLET TYPE            IMPR SLT
       INLET EDGE AND WALL   SQUARE EDGE TOP (26-90 DEG WINGWALL)
       INLET DEPRESSION      FALL INCLUDED

*****  SLOPE-TAPERED IMPROVED INLET ****************
       FACE WIDTH                               3.00 FT
       SIDE TAPER (4:1 TO 6:1) (X:1)            1.68
       FALL SLOPE (2:1 TO 3:1) (X:1)            2.00
       FALL                                     0.10 FT
       MITERED FACE               (Y/N)          Y
       FACE-CREST LENGTH IF MITERED             3.00 FT
______________________________________________________________________________
```

```
CURRENT DATE: 09-09-1997                                  FILE DATE: 09-02-1997
CURRENT TIME: 15:38:43                                    FILE NAME: HARDINB1

                                     TAILWATER

*******  REGULAR CHANNEL CROSS SECTION ****************
         BOTTOM WIDTH (FT)                         31.00
         SIDE SLOPE  H/V (X:1)                      6.0
         CHANNEL SLOPE V/H (FT/FT)                  0.003
         MANNING'S N (.01-0.1)                      0.030
         CHANNEL INVERT ELEVATION (FT)             79.72
         CULVERT NO.1 OUTLET INVERT ELEVATION      79.72 FT

*******  UNIFORM FLOW RATING CURVE FOR DOWNSTREAM CHANNEL

              FLOW     W.S.E.   FROUDE    VEL.    SHEAR
             (CFS)      (FT)    NUMBER   (FPS)    (PSF)
              0.00     79.72    0.000     0.00     0.00
              3.00     79.85    0.336     0.70     0.03
              6.00     79.92    0.357     0.91     0.04
              9.00     79.98    0.370     1.07     0.05
             12.00     80.03    0.378     1.19     0.06
             15.00     80.07    0.385     1.29     0.07
             18.00     80.11    0.390     1.38     0.07
             19.80     80.13    0.393     1.43     0.08
             24.00     80.18    0.398     1.54     0.09
             27.00     80.22    0.401     1.60     0.09
             30.00     80.25    0.404     1.67     0.10

                              ROADWAY OVERTOPPING DATA

ROADWAY SURFACE                                   PAVED
      EMBANKMENT TOP WIDTH (FT)                     40.00
      CREST LENGTH (FT)                             30.00
      OVERTOPPING CREST ELEVATION (FT)              82.15
```

1.3 EROSION AND SEDIMENTATION CONTROL

1.3.1 General

Erosion and sedimentation control in conjunction with a maintenance plan is developed herein as a part of this attachment.

Soil loss computations were made for the final grading plan. The formula utilized for these losses is the universal soil loss equation (USLE) as developed by the Soil Conservation Service. A maximum soil loss of 2 Tons/Acre/Year is recommended for most landfill sites. Several critical areas were selected for detailed calculations (See calculations in this section). Each of the areas selected were found to have a soil loss of less than the maximum recommended, therefore, negating the requirements to utilize berms or other devices to impede the loss of soil.

1.3.2 Proposed Controls

General erosion control is proposed to be accomplished through the combined use of erosion control matting and vegetative cover. Upon completion of ditch work or placement of final cover, portions of the completed earthwork will be covered with jute erosion control matting as manufactured by Erosion Control Systems, Inc. or equal. Matting will be placed according to the manufacturers specifications, covering the entire ditch and the side slope of the landfill cover from the toe of the cover to 40' up the slope. The entire matted area will be seeded and fertilized to facilitate the growth of a stable vegetative cover.

Sack or "pipe" gabions as manufactured by Maccaferri Gabions, Inc. or equal are proposed for placement in various locations for the purpose of sediment control as shown in the drawings included with this attachment. It is proposed that the 9' long, 2' diameter sack gabions be placed across the flow line of the ditch, perpendicular to the flow of water. These gabions will tend to slow velocities in the ditch and settle out sedement which may be transported by stormwater runoff. Sack gabions are also proposed at culvert inlets and outlets as shown in the drawings.

At the Southwest corner of the landfill, the western landfill ditch will empty into a shallow pond. This pond will serve as a means of slowing flows coming out of the ditch, reducing sedement loading in the runoff and diffusing the flow. Some limited storage is also available in this pond, extending the runoff time.

1.3.3 Maintenance

Inspections of all constructed earthwork shall be undertaken once a week and within 24 hours of any major rainfall. Where sedement has collected in ditches or ponds, this will be removed or graded as necessary to maintain the constructed grades. Sack gabions

CONSTRUCTION NOTES

Fig. 1

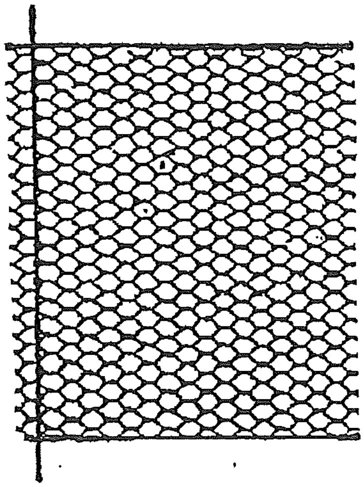
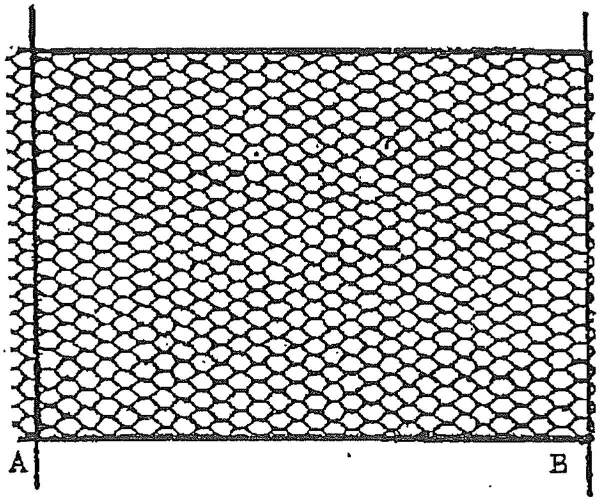


Fig. 2

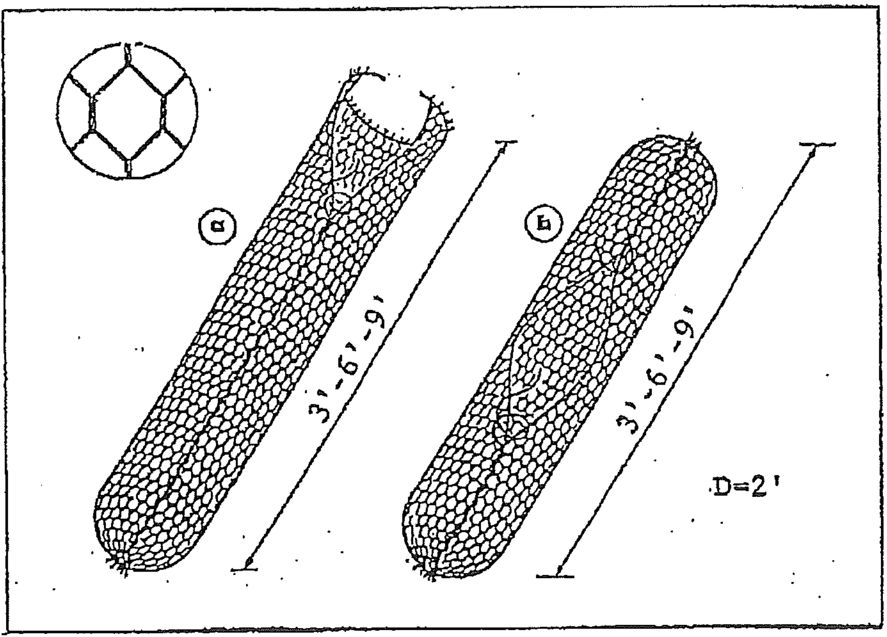


ASSEMBLING:

Sack Gabions are supplied folded flat, packed in bundles. Single sacks shall be removed from the bundle, (Fig. 1) unfolded flat on the ground, and all kinks and bends stepped out. (Fit. 2).

For vertical filling (Fig. 3a), connect the two sides edge wires by using the supplied lacing wire in a single loop-double loop pattern on a 4" to 5" spacing (Fasteners supplied by Spenax Corp. may be used on a 4" spacing). END A: the "end lacing rod" must be pulled tight and wrapped around end and twisted 4 times. END B: pull rod tight, cut and leave about 6" length and twist 4 times.

Fig. 3



For horizontal filling (Fig. 3b), place flat in a filling trough, fill with stone, then connect sides (as mentioned above), secure both ends (as mentioned above).

LIFTING AND PLACING:

Place a #6 rebar (or equal) 5' long in mesh, perpendicularly to the longitudinal axis close to the knot of end A and lift from central point. Sack Gabions should conform to existing contours.

MACCAFERRI GABIONS INC.
10303 GOVERNOR LANE BLVD., WILLIAMSPORT, MARYLAND 21795

FAX: (301) 223-6134 TELEPHONE: (301) 223-6910

SPECIFICATON FOR STANDARD SACK GABIONS MADE OF ZINC COATED DOUBLE TWIST 6 X 8 TYPE WIRE MESH WITH PVC SLEEVE

1) GENERAL DESCRIPTION

The PVC coated Sack Gabion shall be a flexible zinc coated (galvanized) sack of the type and sizes stated below. It is made of wire mesh of the type and size and selvedged as specified in the following paragraphs. The Sack Gabions are made of a single sheet of wire mesh.

2) MESH

The mesh shall be a double twisted hexagonal woven mesh. The size of the mesh conforms to the specification issued by the plant and shall be of 6 X 8 type wire mesh. Nominal mesh size is 2 1/2" X 3 1/4".

3) WIRE

All wire used in the fabrication of the Sack Gabions and in the wiring operations during construction shall conform to or exceed requirements of ASTM A 853-91 Class 3 Finish 5 soft, i.e. wire having an average tensile strength in accordance with ASTM A641-89 measured before fabrication of the netting. The diameter of the wire used in the fabrication of the netting shall be 0.0866 inches with a PVC coating extruded onto the wire core, having a nominal thickness of 0.02165 inches (minimum thickness 0.015 inches). An overall nominal diameter of 0.1299 inches is obtained.

4) ELONGATION OF WIRE

Test shall be made on the wire before coating with PVC and fabrication of the sacks, on a sample ten inches long. Elongation shall not be less than 12%.

5) ZINC COATING (GALVANIZING)

All wire used in the fabrication of the gabions and in the wiring operations during construction shall be galvanized to ASTM A641-89. Zinc coated (Galvanized) carbon steel wire, that is to say, the minimum weight of the zinc coating shall be according to the figures shown in the table below when tested in accordance with ASTM A90-81.

Nominal Diameter of Wire	Minimm weight of coating
0.0866 inches	0.70 ozs./sq.ft.
0.1063 inches	0.80 ozs./sq.ft.

6) EDGE WIRES AND RODS

The edge wires and the "end lacing rods" shall have a diameter greater than that of the wire used to form the mesh, namely:
For the 6 X 8 type wire mesh, made of wire having a core diameter of 0.0866 inches, the edge wire and the end lacing rod shall be of wire having a core diameter of 0.1063 inches of greater.

7) STANDARD DIMENSIONS OF PVC COATED SACK GABIONS

Nominal Length = 3 feet, 6 feet or 9 feet.
Nominal Diameter = 2 feet.

8) LACING WIRE

Lacing PVC coated wire is supplied with the Sack Gabions for all wiring operations carried out in the construction of the work.

The diameter of lacing wire shall be 0.0866 inches and shall comply to the same specification as the wire used in the mesh.

8A) FASTENERS

Fasteners of stainless steel may be used with PVC coated sacks for wiring operations in alternative to PVC lacing wire. The use of fasteners shall be approved by the owner/engineer. Fasteners should be Spenax Corporation reference #11SS40 or similar and should be fabricated from 0.120 inch stainless steel wire having high tensile strength.

9) TOLERANCES

Wire.

Tolerances on the diameter of all wire in the above clauses shall be permitted in accordance with ASTM A641-89 Table 3.

Sack Gabion.

A tolerance of + or - 5% on the width and on the length of the Sack Gabions shall be permitted.

10) P.V.C. COATING

All wire used in the fabrication of the Sack Gabions and in the wiring operations during construction shall, after zinc coating, have extruded onto it a coating of poly vinyl chloride, otherwise referred to as "P.V.C.". The coating shall be gray in color, of nominal thickness 0.02165 inches and shall nowhere be less than 0.015 inches in thickness. It shall be capable of resisting deleterious effects of natural weather exposure, immersion in salt water and shall not show any material difference in its initial characteristics.

NOTES

Maccaferri Gabions, Inc. reserves the right to amend these specifications without notice and specifiers are requested to check as to the validity of the specification they are using. The date of this issue is May 1992.

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2 NOTICE OF INTENT

The Contractor and Owner shall submit a Notice of Intent (NOI) to comply with the NPDES General Permit for the State of Texas no later than two days prior to the commencement of Construction Activities (e.g. the initial disturbance of soils associated with clearing, grading, excavation activities, or other construction activities). A sample NOI is appended to this attachment.

Appendix C — NOI Form Instructions

See Reverse for Instructions

Form Approved. OMB No. 2040-0086
Approval expires: 8-31-95

NPDES
FORM

EPA

United States Environmental Protection Agency
Washington, DC 20460

Notice of Intent (NOI) for Storm Water Discharges Associated with Industrial Activity Under the NPDES General Permit

Submission of this Notice of Intent constitutes notice that the party identified in Section I of this form intends to be authorized by a NPDES permit issued for storm water discharges associated with industrial activity in the State identified in Section II of this form. Becoming a permittee obligates such discharger to comply with the terms and conditions of the permit. ALL NECESSARY INFORMATION MUST BE PROVIDED ON THIS FORM.

I. Facility Operator Information

Name:
Phone:
Address:
Status of Owner/Operator: ☐
City:
State:
ZIP Code:

II. Facility/Site Location Information

Is the Facility Located on Indian Lands? (Y or N) ☐

Name:
Address:
City:
State:
ZIP Code:
Latitude:
Longitude:
Quarter:
Section:
Township:
Range:

III. Site Activity Information

MS4 Operator Name:
Receiving Water Body:
If You are Filing as a Co-permittee, Enter Storm Water General Permit Number:
Are There Existing Quantitative Data? (Y or N) ☐
Is the Facility Required to Submit Monitoring Data? (1, 2, or 3) ☐
SIC or Designated Activity Code: Primary: 2nd: 3rd: 4th:
If This Facility is a Member of a Group Application, Enter Group Application Number:
If You Have Other Existing NPDES Permits, Enter Permit Numbers:

IV. Additional Information Required for Construction Activities Only

Project Start Date:
Completion Date:
Estimated Area to be Disturbed (in Acres):
Is the Storm Water Pollution Prevention Plan in Compliance with State and/or Local Sediment and Erosion Plans? (Y or N) ☐

V. Certification: I certify under penalty of law that this document and all attachments were prepared under my direction or supervision in accordance with a system designed to assure that qualified personnel properly gather and evaluate the information submitted. Based on my inquiry of the person or persons who manage the system, or those persons directly responsible for gathering the information, the information submitted is, to the best of my knowledge and belief, true, accurate, and complete. I am aware that there are significant penalties for submitting false information, including the possibility of fine and imprisonment for knowing violations.

Print Name:
Date:

Signature: ______________________________

EPA Form 3510-6 (8-92)

3 SOIL LOSS CALCULATIONS

Soil loss calculations were made in accordance with the Universal Soil Loss Equation as provided by TNRCC Procedural Handbook. The recommended allowable soil loss is 2.0 tons/yr. Calculations for critical areas does not exceed the allowable soil loss. See calculations at the end of this attachment.

will be shaken clear of silt as frequently as necessary to maintain adequate flow through the gabion, but no less frequently than once per quarter.

Eroded earthwork will be repaired to constructed grades and conditions as shown in this permit application. Grading and placement of soil materials shall be done within 48 hours of the damage. Erosion matting along with seeding and fertilization shall be placed as soon as possible but within no more than 7 days of the damage.

Areas where erosion proves to be a regular occurrence shall be noted and actions will be taken to provide more permanent solutions to the problem in these areas. Such actions include, but are not limited to, Reno mattresses, getextiles, terracing, and other physical modifications which may be warranted.

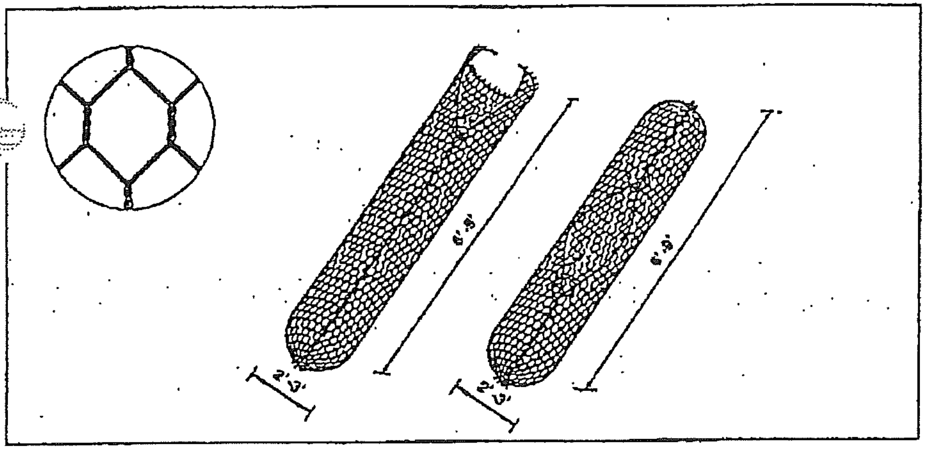
1.4 FLOODPLAINS

No portion of this site is located in any flood plain which would adversely affect the integrity or operation of the liner or landfill. The nearest streams are Cypress Creek and Langston Branch. Cypress Creek is located 1,500 feet to the north of the site while Langston Branch is 1,000 feet east of the site. Neither creek is close enough to the site to be a threat to the site nor be threatened by contamination from the site. Additionally, research has shown that the site is well outside of the 100 year flood plain, removing any concern for surface water contamination at the site.

FEMA records the site as being outside of the 100 year flood plain as shown on FEMA's Flood Plain Maps. In the spring of 1979, the highest water levels for the area as remembered by the highway department were observed. Highway 326 flooded at the Cypress Creek Crossing. As this bridge is designed for a 50 year flood, it is assumed that the observed flooding approximated the 100 year flood. The Highway 770 crossing of Langston Branch, just to the east of the site, was fordable by pickup trucks, but not by cars. This being the case, the flood elevation is assumed to have been 18" higher than the top of the box culverts at FM 770 and Langston Branch. Based on these assumptions, the 100 year flood elevation can be determined to be 62.6 feet. Conservatively, the 100 year flood elevation is estimated to be 63.0 feet. The lowest elevation on the proposed site is 71 feet, placing the lowest part of the landfill approximately 8 feet above the 100 year flood elevation. The proposed landfill site is well outside any flood hazard.

Description of the Maccaferri sack gabion

32



The Maccaferri sack gabions are mainly used for river and stream training works. They are made up of a single sheet of mesh and are supplied with top and bottom steel wire bars inserted during the production phase to facilitate closing during installation procedures.

The characteristics of mesh, wire, zinc and PVC coating are indentical to those of box gabions. Filling operations of these units could be carried out either from the side or from one of the open ends of the sack.

Applications

33

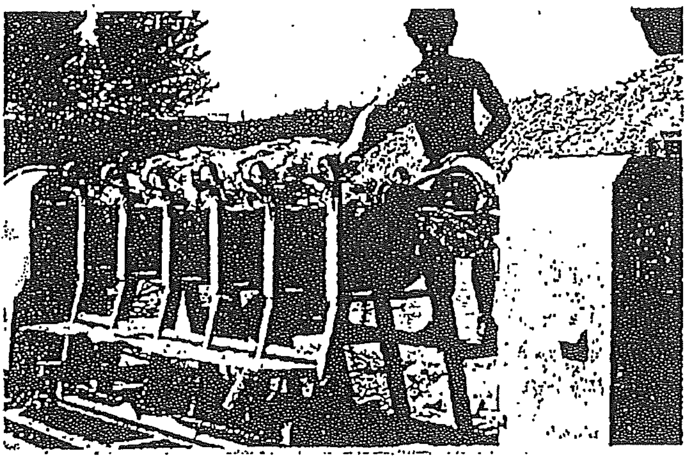


Fig. 33 - ITALY - Lombardy
Assembly of a sack gabion using a special machine.

34

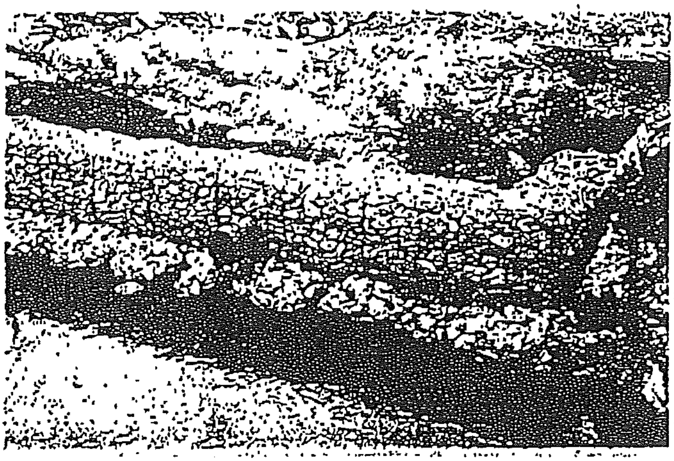
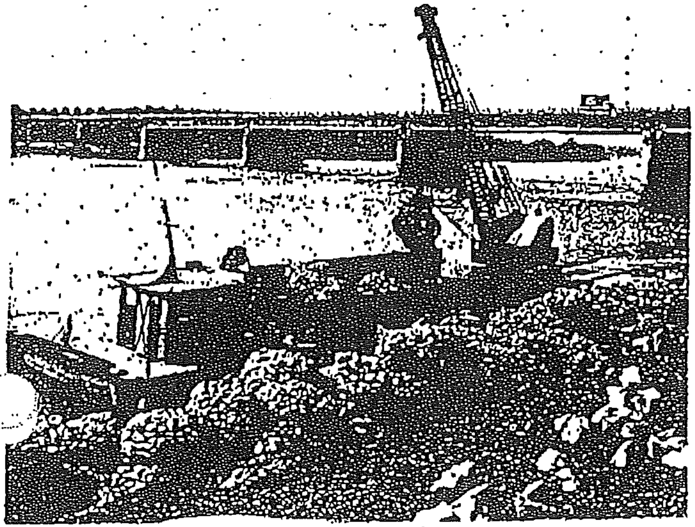
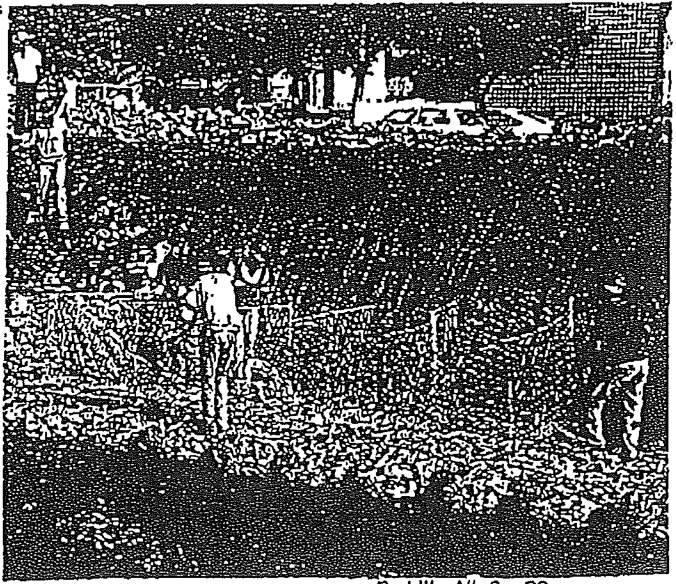


Fig. 34 - BRAZIL - S. Paulo
Sack gabion foundation for a bank protection wall on the Tamanduatéi river.

35



36



HARDIN COUNTY LANDFILL
SITE DEVELOPMENT PLAN - ATTATCHMENT 6
GROUNDWATER AND SURFACE WATER PROTECTION AND DRAINAGE PLANS

SOIL LOSS CALCULATIONS

R. L. THURBER
44228

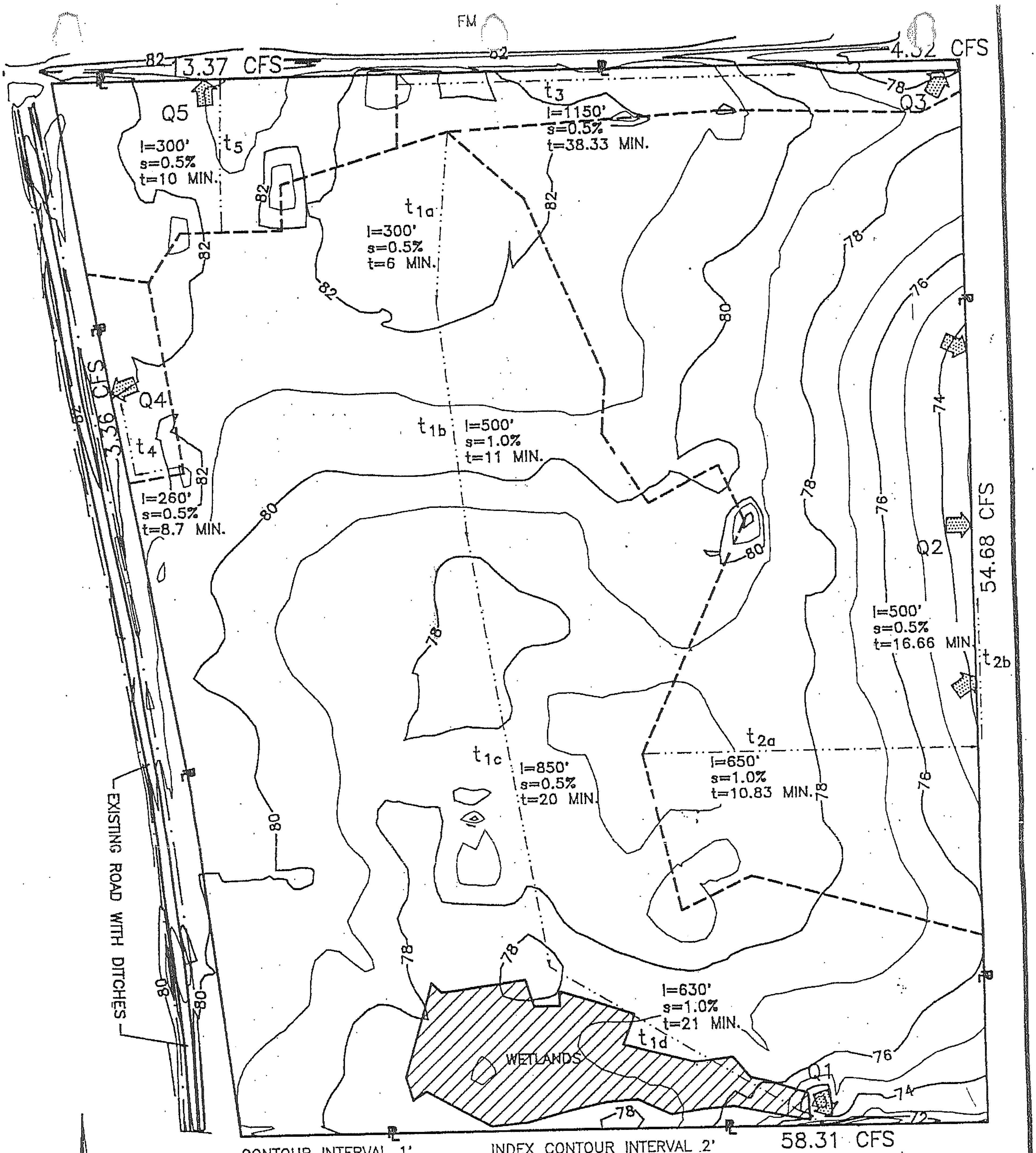
8/19/96

Universal Soil Loss Equation: $A = R*K*LS*C*P$

A = soil loss in tons / acre / year
R = rainfall and runoff factor
K = soil erodibility factor
LS = topographic factor/slope steepness factor
C = cover and management factor
P = support practice factor

R	K	Slope (%)	Length (feet)	LS	C	P	A (ton/Ac/yr)	< 2 t.p.y.?
FOR AREAS 1 & 2								
450.00	0.30	6.00	400.00	1.32	0.013	0.50	1.16	Yes
FOR AREAS 7 & 8								
450.00	0.30	6.00	336.33	1.25	0.013	0.50	1.10	Yes
FOR AREAS 3, 4, 5, & 6								
450.00	0.30	3.20	721.00	0.56	0.013	0.50	0.49	Yes
FOR AREAS 9, 10, 11, 12								
450.00	0.30	2.20	842.33	0.39	0.013	0.50	0.34	Yes
FOR AREAS 13, 14, 15, & 16								
450.00	0.30	6.90	66.00	0.68	0.013	0.50	0.60	Yes

Calculations revised to reflect change in MSW landfill footprint and Inclusion of C & D Waste Area



CONTOUR INTERVAL 1' INDEX CONTOUR INTERVAL 2'

- ⇨ INDICATES DIRECTION OF RUNOFF
- — — BOUNDARY OF RUNOFF AREA
- — - — FLOW LINE OF DITCH
- — · · — TIME OF CONCENTRATION LINE
- —P— PROPERTY LINE

l= LENGTH
s= SLOPE
t= TIME OF CONCENTRATION

NOTE:
NO PORTION OF SITE WITHIN 100–YEAR FLOODPLAN.

FOR 25 – YEAR STORM

$Q = CIA \quad I = \frac{b}{(t + d)^e}$ HARDIN COUNTY b = 80 d = 7.5 e = 0.720

	C	I (in/hour)	A (Ac)	FLOW (CFS)
Q1	0.35	3.625	45.96	58.31
Q2	0.35	6.185	25.26	54.68
Q3	0.35	5.096	2.42	4.32
Q4	0.35	10.19	1.03	3.36
Q5	0.35	10.19	3.75	13.37

SCALE IN FEET
200 0 200

SCALE: 1"=200'

FOR PERMITTING PURPOSES ONLY

STATE OF TEXAS
BILLY D. SIMS
53016
PROFESSIONAL ENGINEER

KSA ENGINEERS, INC. 1313 S. JOHN REDDITT DR.–P.O. BOX 151308 LUFKIN, TEXAS 75915–1508 (409) 637–6061
ENGINEERS – SURVEYORS DALLAS LONGVIEW LUFKIN TYLER

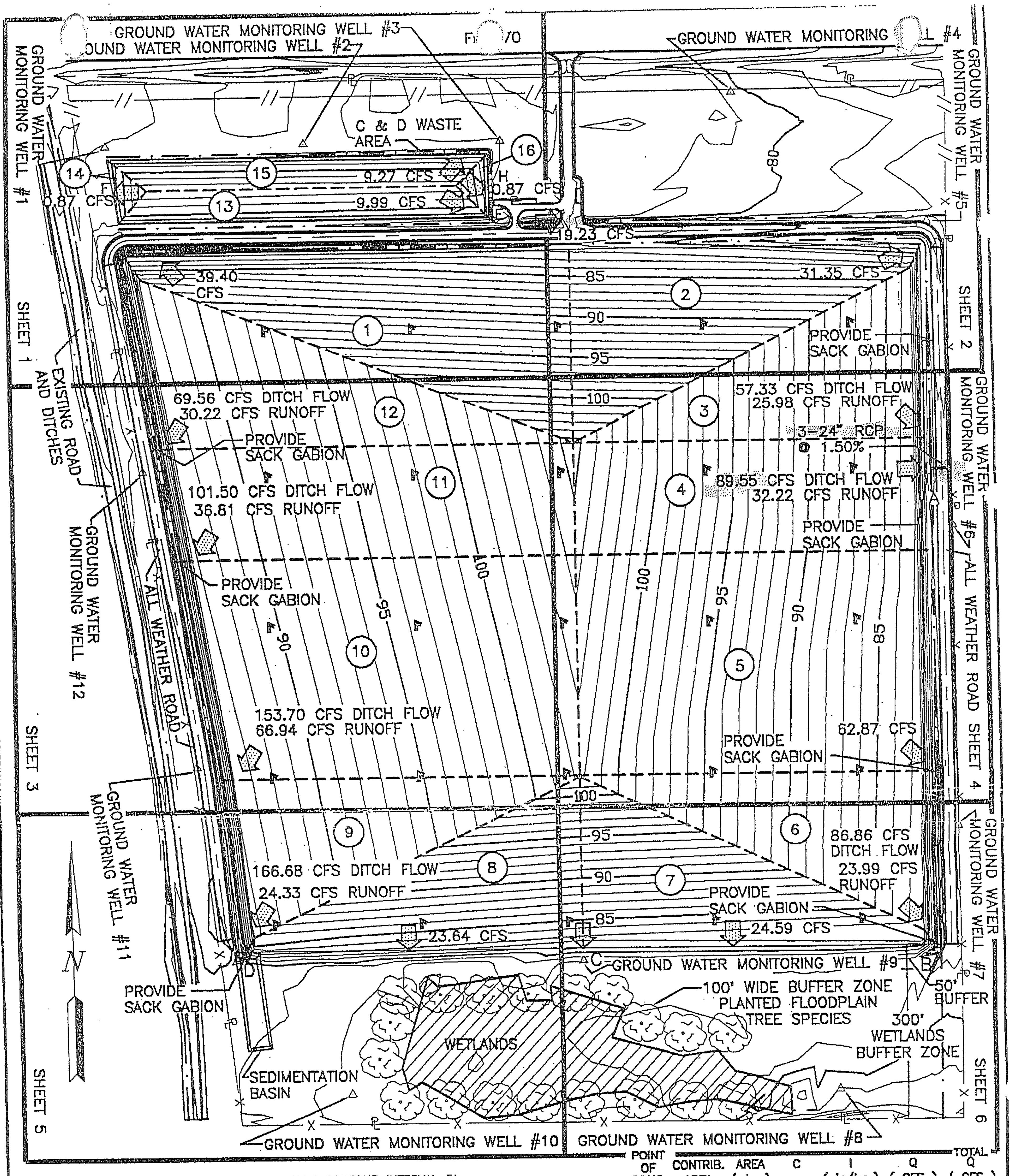
SURVEYED	DESIGNED	DRAWN	APPROVED	JOB NO.	DATE
N.A.	ALSII	DLM	BDS	HN-001	AUG. 1994

HARDIN COUNTY
LANDFILL PERMIT
APPLICATION–PART III
PROJECT NAME

SITE DEVELOPMENT PLAN
ATTACHMENT #6
EXISTING DRAINAGE
SHEET NAME

DRAWING SCALE:
VERT: NONE
HORIZ: 1"=200'
PLOT: AUG. 94
FN: EXDRAIN

SHEET NO.
33
OF



CONTOUR INTERVAL 1' INDEX CONTOUR INTERVAL 5'

- - - BOUNDARY OF RUNOFF AREA
⇨ INDICATES DIRECTION OF RUNOFF
(#) RUNOFF AREA (SEE ATTACHMENT #6 & #13)
—·— FLOW LINE OF DITCH (SEE ATTACHMENT #6)
△ MONITORING WELL (SEE ATTACHMENT #11 & #13)
—℄— PROPERTY LINE
—X— BARBED WIRE FENCE(SEE ATTACHMENT #1)
—//— CHAIN LINK FENCE(SEE ATTACHMENT #1)
GAS VENT(SEE ATTACHMENT #14)

$Q = CIA$

$$I = \frac{b}{(t + d)^e}$$

HARDIN COUNTY
25-YEAR STORM

$b = 80$
$d = 7.5$
$e = 0.720$

NOTE:
SIDE SLOPES 4:1 HORIZONTAL TO VERTICAL
TOP SLOPES 2% MINIMUM AND 25% MAXIMUM
FOR CROSS SECTIONS SEE ATTACHMENT #2

POINT OF CONC.	CONTRIB. AREA	AREA (Ac)	C	I (In/hr)	Q (CFS)	TOTAL Q (CFS)
	2	3.62	0.85	10.19	31.35	31.35
	3	3.00	0.85	10.19	25.98	57.33
	4	3.72	0.85	10.19	32.22	32.22
A		10.34	0.85	10.19	89.55	89.55
	5	7.26	0.85	10.19	62.87	62.87
	6	2.77	0.85	10.19	23.99	86.86
B		10.03	0.85	10.19	86.86	86.86
	7	2.84	0.85	10.19	24.59	24.59
	8	2.73	0.85	10.19	23.64	23.64
C		5.57	0.85	10.19	48.24	48.24
	1	4.55	0.85	10.19	39.40	39.40
	12	3.49	0.85	10.19	30.22	69.56
	11	4.25	0.85	10.19	36.81	101.50
	10	7.73	0.85	10.19	66.94	153.70
	9	2.81	0.85	10.19	24.33	166.68
D		22.83	0.85	8.59	166.68	166.68
	13	1.05	0.85	10.19	9.09	9.09
	15	1.07	0.85	10.19	9.27	9.27
	16	0.10	0.85	10.19	0.87	10.13
E		2.22	0.85	10.19	19.23	19.23
F	14	0.10	0.85	10.19	0.87	0.87

FOR PERMITTING PURPOSES ONLY

BILLY D. SIMS
53016

SCALE IN FEET
200 0 200
SCALE: 1"=200'

REVISED SEPT. 9, 1997
REVISED AUG. 16, 1996

KSA ENGINEERS, INC. 1313 S. JOHN REDDITT DR.–P.O. BOX 151508
LUFKIN, TEXAS 75915-1508 (409) 637-6061
ENGINEERS - SURVEYORS
DALLAS LONGVIEW LUFKIN TYLER

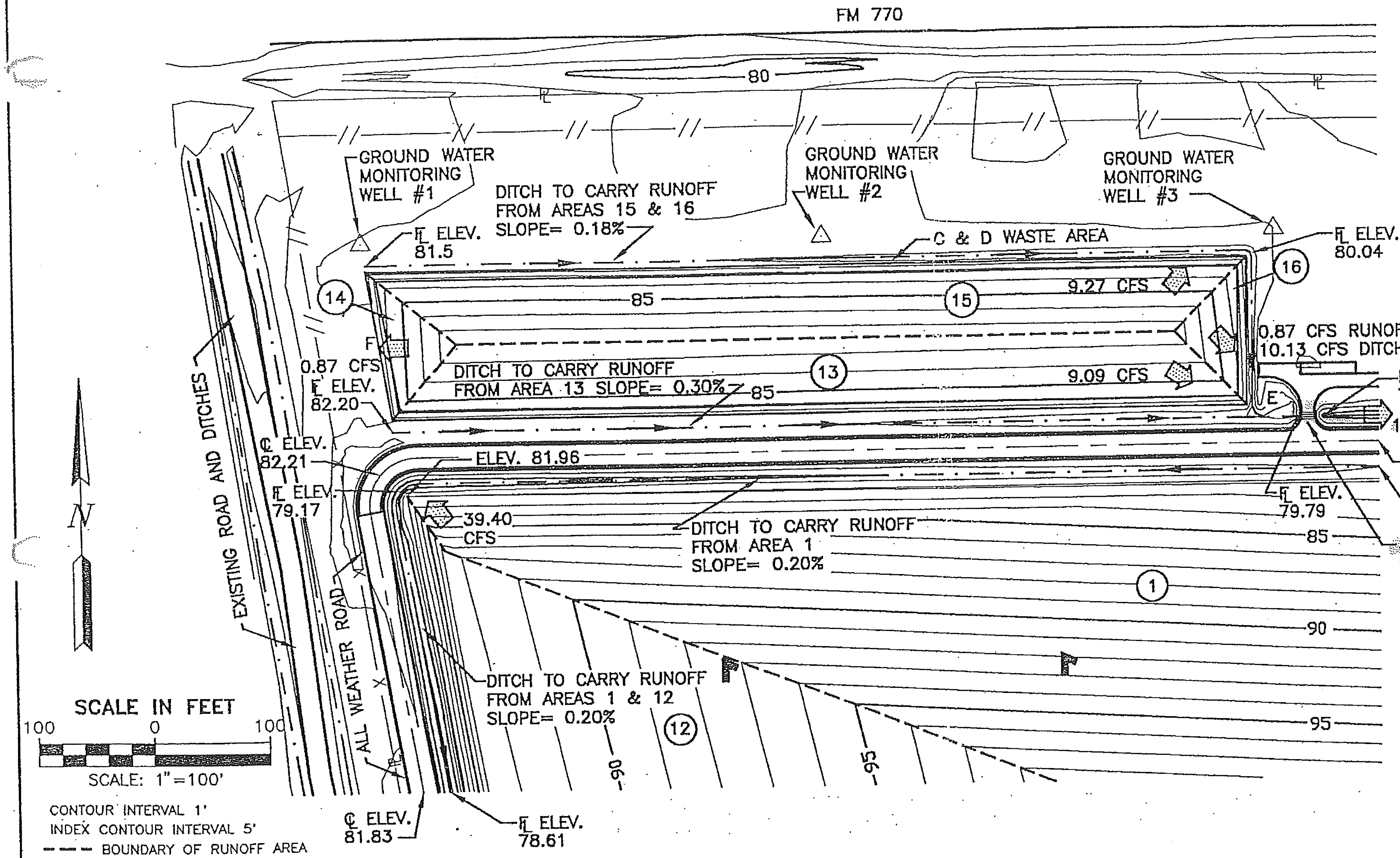
SURVEYED	DESIGNED	DRAWN	APPROVED	JOB NO.	DATE
N.A.	ALSII	DLM	BDS	HN-001	AUG. 1994

HARDIN COUNTY
LANDFILL PERMIT
APPLICATION–PARTIII
PROJECT NAME:

SITE DEVELOPMENT PLAN
ATTACHMENT # 6
FINAL CONTOUR MAP
SHEET NAME:

DRAWING SCALE:
VERT: NONE
HORIZ: 1"=200'
PLOT: SEPT. 97
FN: FINDRAIN

SHEET NO.
34
OF



MAXIMUM DITCH SIDE SLOPE 3:1 HORIZONTAL:VERTICAL

ALL DITCHES SIZED TO CARRY FLOWS FROM THE 25 – YEAR STORM.

⟶ DIRECTION OF FLOW

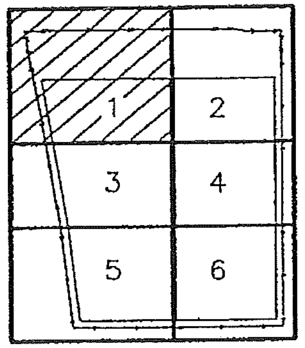
SIDE SLOPES 4:1 HORIZONTAL:VERTICAL

TOP SLOPES 2% MINIMUM AND 25% MAXIMUM.

FOR MORE INFORMATION ON FENCING AND ALL-WEATHER ROAD SEE ATTACHMENT #1

FOR MORE INFORMATION ON GAS VENTS SEE ATTACHMENT #14

FOR MORE INFORMATION ON MONITORING WELLS SEE ATTACHMENTS #11 & #13



CONTOUR INTERVAL 1'
INDEX CONTOUR INTERVAL 5'
– – – BOUNDARY OF RUNOFF AREA
⇨ INDICATES DIRECTION OF RUNOFF
—·— FLOW LINE OF DITCH
(#) RUNOFF AREA
△ MONITORING WELL
PL PROPERTY LINE
FL FLOW LINE
CL CENTER LINE

GAS VENT
—//— CHAIN LINK FENCE(SEE ATTACHMENT #1)
—X— BARBED WIRE FENCE(SEE ATTACHMENT #1)

FOR PERMITTING PURPOSES ONLY

BILLY D. SIMS

SHEET 1

REVISED SEPT. 9, 1997
REVISED AUG. 16, 1996

MARK	REVISION	BY	CK'D	DATE

KSA ENGINEERS, INC.
ENGINEERS - SURVEYORS
1313 SOUTH JOHN REDDITT DR. - P.O. BOX 151508
LUFKIN, TEXAS 75915-1508 (409) 637-6061
DALLAS LONGVIEW LUFKIN TYLER

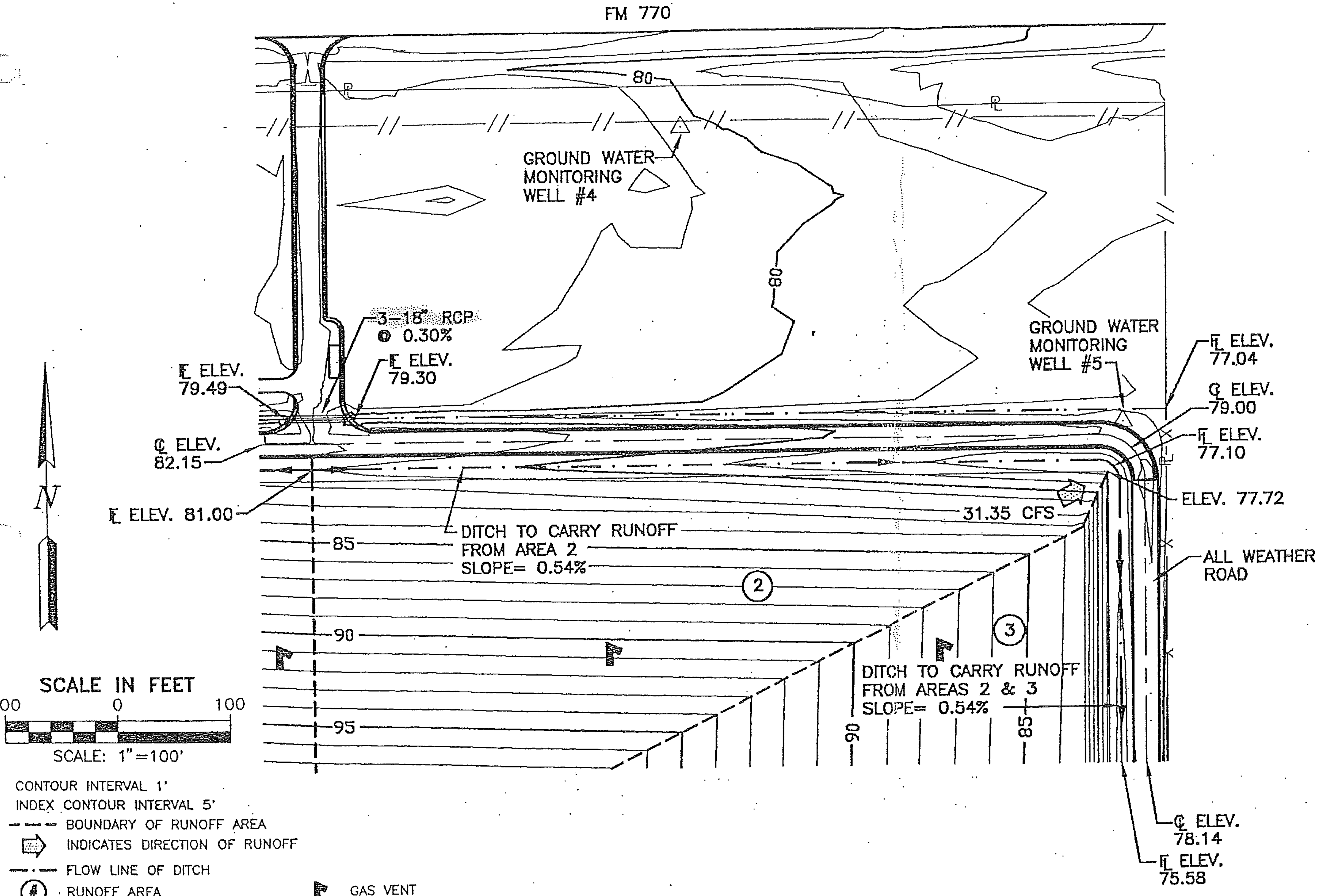
SURVEYED	DESIGNED	DRAWN	APPROVED	JOB NO.	DATE
N.A.	ALSII	DLM	BDS	HN-001	AUG. 1994

HARDIN COUNTY
LANDFILL PERMIT APPLICATION
PART III
PROJECT NAME:

SITE DEVELOPMENT PLAN
ATTACHMENT #6
FINAL DRAINAGE DETAIL
SHEET

DRAWING SCALE:
VERTICAL: NONE
HORIZONTAL: 1"=100'
PLOT DATE: SEPT. 1997
FILE NAME: FINSECT

SHEET NO.
35
OF



SCALE IN FEET

100 0 100

SCALE: 1"=100'

CONTOUR INTERVAL 1'
INDEX CONTOUR INTERVAL 5'

— — —	BOUNDARY OF RUNOFF AREA
⇨	INDICATES DIRECTION OF RUNOFF
—·—	FLOW LINE OF DITCH
(#)	RUNOFF AREA
△	MONITORING WELL
PL	PROPERTY LINE
FL	FLOW LINE
CL	CENTER LINE
⚑	GAS VENT
—//—	CHAIN LINK FENCE(SEE ATTACHMENT #1)
—x—	BARBED WIRE FENCE(SEE ATTACHMENT #1)

MAXIMUM DITCH SIDE SLOPE 3:1 HORIZONTAL:VERTICAL

ALL DITCHES SIZED TO CARRY FLOWS FROM THE 25 – YEAR STORM.

——→ DIRECTION OF FLOW

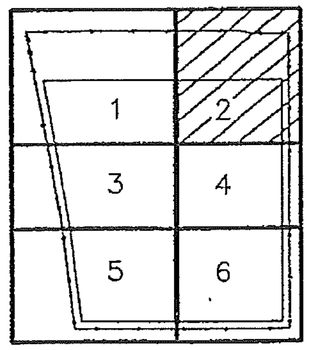
SIDE SLOPES 4:1 HORIZONTAL:VERTICAL

TOP SLOPES 2% MINIMUM AND 25% MAXIMUM.

FOR MORE INFORMATION ON FENCING AND ALL-WEATHER ROAD SEE ATTACHMENT #1

FOR MORE INFORMATION ON GAS VENTS SEE ATTACHMENT #14

FOR MORE INFORMATION ON MONITORING WELLS SEE ATTACHMENTS #11 & #13



FOR PERMITTING PURPOSES ONLY

BILLY D. SIMS

SHEET 2

REVISED SEPT.9, 1997
REVISED AUG. 16, 1996

MARK	REVISION	BY	CK'D	DATE

KSA ENGINEERS, INC.
ENGINEERS - SURVEYORS
1313 SOUTH JOHN REDDITT DR. – P.O. BOX 151508
LUFKIN, TEXAS 75915-1508 (409) 637-6061
DALLAS LONGVIEW LUFKIN TYLER

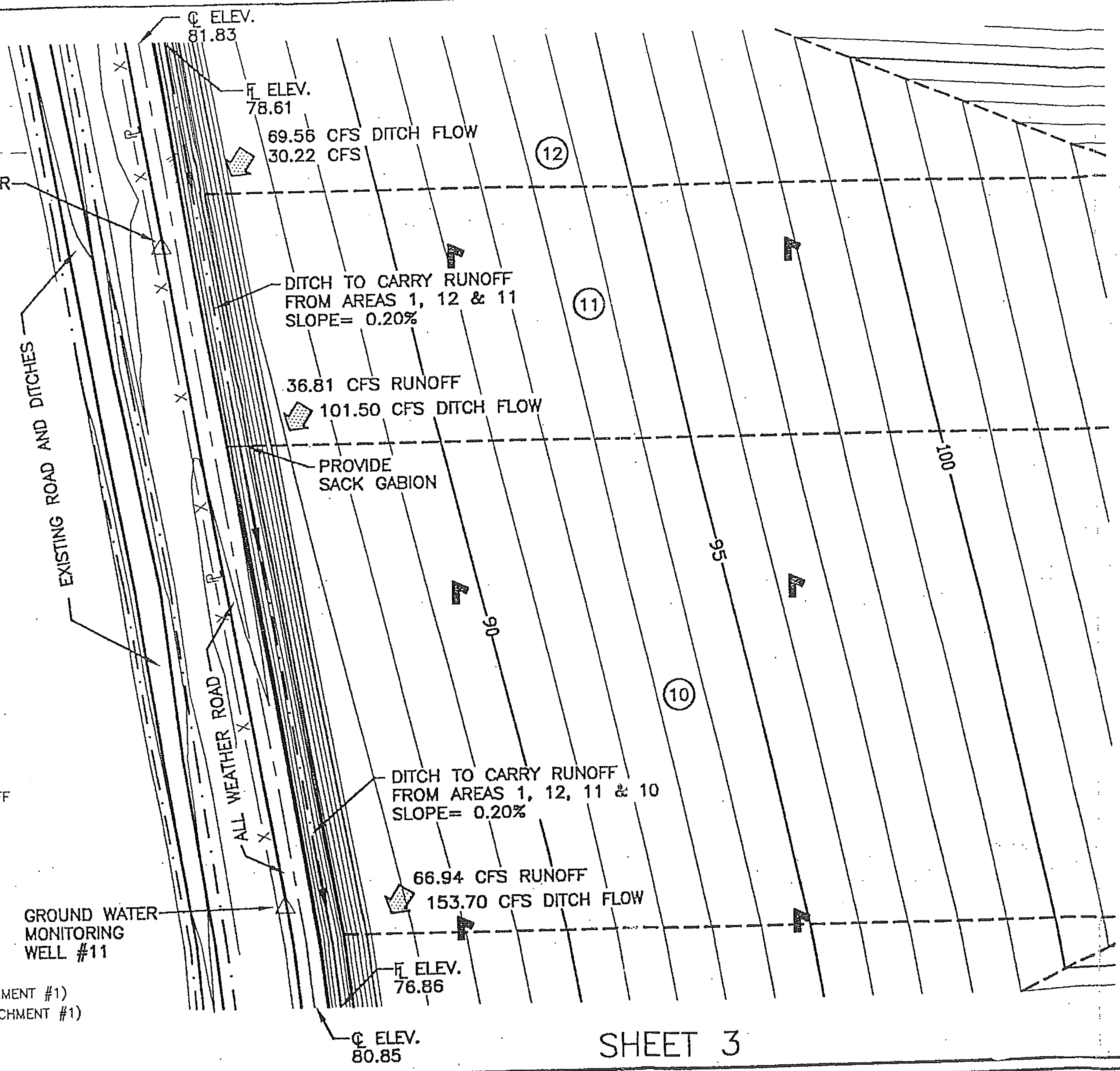
SURVEYED	DESIGNED	DRAWN	APPROVED	JOB NO.	DATE
N.A.	ALSII	DLM	BDS	HN-001	AUG. 1994

HARDIN COUNTY
LANDFILL PERMIT APPLICATION
PART III
PROJECT NAME:

SITE DEVELOPMENT PLAN
ATTACHMENT #6
FINAL DRAINAGE DETAIL
SHEET

DRAWING SCALE:
VERTICAL: NONE
HORIZONTAL: 1"=100'
PLOT DATE: SEPT. 1997
FILE NAME: FINSECT

SHEET NO.
36
OF



MAXIMUM DITCH SIDE SLOPE 3:1 HORIZONTAL:VERTICAL

ALL DITCHES SIZED TO CARRY FLOWS FROM THE 25 – YEAR STORM.

——→ DIRECTION OF FLOW

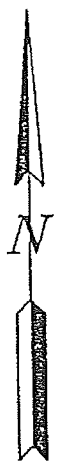
SIDE SLOPES 4:1 HORIZONTAL:VERTICAL

TOP SLOPES 2% MINIMUM AND 25% MAXIMUM.

FOR MORE INFORMATION ON FENCING AND ALL–WEATHER ROAD SEE ATTACHMENT #1

FOR MORE INFORMATION ON GAS VENTS SEE ATTACHMENT #14

FOR MORE INFORMATION ON MONITORING WELLS SEE ATTACHMENTS #11 & #13

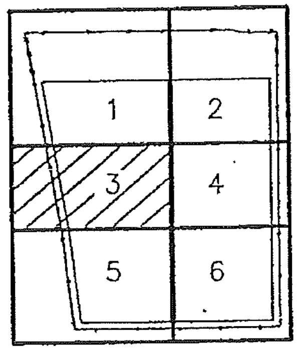


SCALE IN FEET

100 0 100

SCALE: 1"=100'

CONTOUR INTERVAL 1'
INDEX CONTOUR INTERVAL 5'
– – – – BOUNDARY OF RUNOFF AREA
⇨ INDICATES DIRECTION OF RUNOFF
—·— FLOW LINE OF DITCH
(#) RUNOFF AREA
△ MONITORING WELL
℞ PROPERTY LINE
℄ FLOW LINE
℄ CENTER LINE
⚑ GAS VENT
—//— CHAIN LINK FENCE(SEE ATTACHMENT #1)
—x— BARBED WIRE FENCE(SEE ATTACHMENT #1)



FOR PERMITTING PURPOSES ONLY

BILLY D. SIMS
53016

SHEET 3

REVISED SEPT. 9, 1997
REVISED AUG. 16, 1996

MARK	REVISION	BY	CK'D	DATE

KSA ENGINEERS, INC.
ENGINEERS - SURVEYORS
1313 SOUTH JOHN REDDITT DR. – P.O. BOX 151508
LUFKIN, TEXAS 75915-1508 (409) 637-6061
DALLAS LONGVIEW LUFKIN TYLER

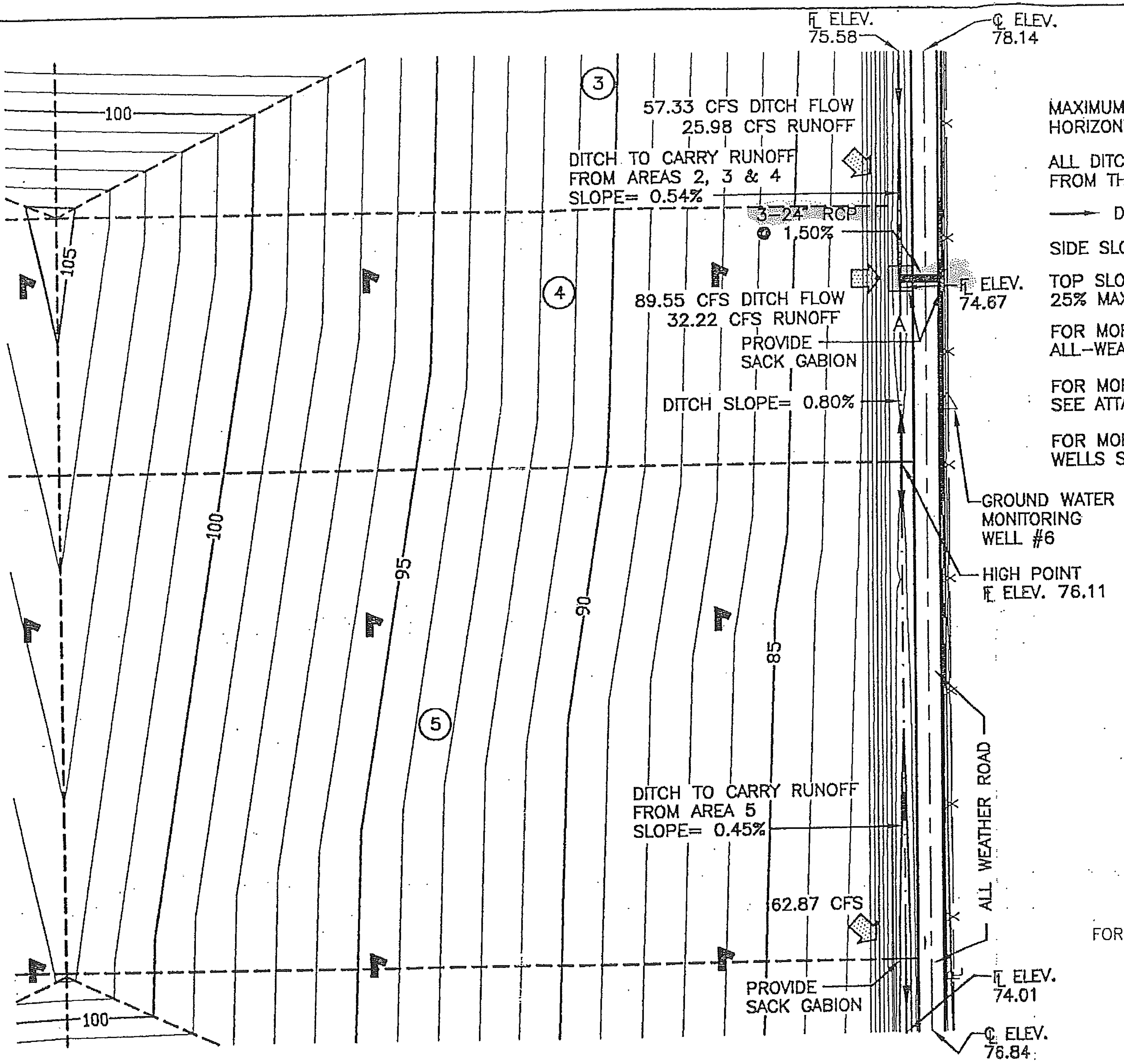
SURVEYED	DESIGNED	DRAWN	APPROVED	JOB NO.	DATE
N.A.	ALSII	DLM	BDS	HN–001	AUG. 1994

HARDIN COUNTY
LANDFILL PERMIT APPLICATION
PART III
PROJECT NAME:

SITE DEVELOPMENT PLAN
ATTACHMENT #6
FINAL DRAINAGE DETAIL
SHEET

DRAWING SCALE:
VERTICAL: NONE
HORIZONTAL: 1"=100'
PLOT DATE: SEPT. 1997
FILE NAME: FINSECT

SHEET NO.
37
OF



MAXIMUM DITCH SIDE SLOPE 3:1
HORIZONTAL:VERTICAL

ALL DITCHES SIZED TO CARRY FLOWS FROM THE 25 – YEAR STORM.

——→ DIRECTION OF FLOW

SIDE SLOPES 4:1 HORIZONTAL:VERTICAL

TOP SLOPES 2% MINIMUM AND 25% MAXIMUM.

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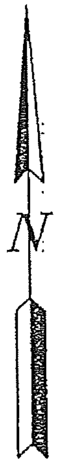
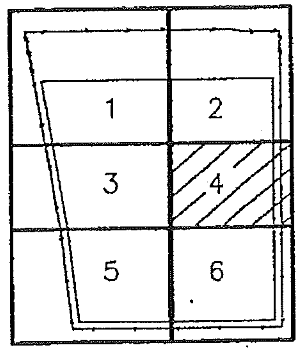
FOR MORE INFORMATION ON GAS VENTS SEE ATTACHMENT #14

FOR MORE INFORMATION ON MONITORING WELLS SEE ATTACHMENTS #11 & #13





SCALE IN FEET
100 0 100
SCALE: 1"=100'

CONTOUR INTERVAL 1'
INDEX CONTOUR INTERVAL 5'

— — —	BOUNDARY OF RUNOFF AREA
⇨	INDICATES DIRECTION OF RUNOFF
—·—	FLOW LINE OF DITCH
(#)	RUNOFF AREA
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⅊	PROPERTY LINE
℄	FLOW LINE
℄	CENTER LINE
⚑	GAS VENT
—//—	CHAIN LINK FENCE(SEE ATTACHMENT #1)
—×—	BARBED WIRE FENCE(SEE ATTACHMENT #1)

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BILLY D. SIMS
67016

SHEET 4

REVISED SEPT. 9, 1997
REVISED AUG. 16, 1996

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LUFKIN, TEXAS 75915-1506 (409) 637-6061
DALLAS LONGVIEW LUFKIN TYLER

SURVEYED	DESIGNED	DRAWN	APPROVED	JOB NO.	DATE
N.A.	ALSII	DLM	BDS	HN-001	AUG. 1994

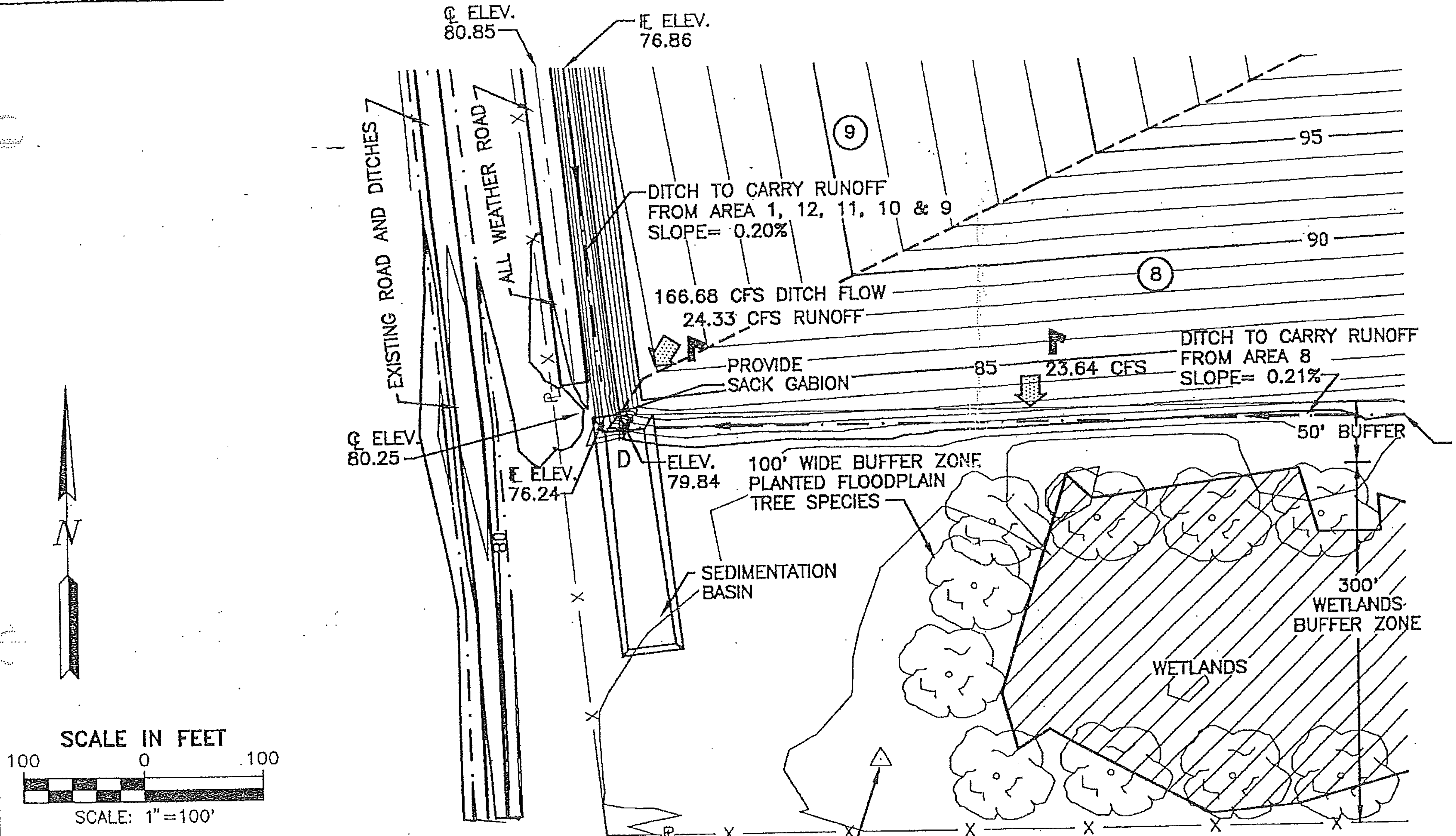
HARDIN COUNTY
LANDFILL PERMIT APPLICATION
PART III
PROJECT NAME:

SITE DEVELOPMENT PLAN
ATTACHMENT #6
FINAL DRAINAGE DETAIL
SHEET

DRAWING SCALE:
VERTICAL: NONE
HORIZONTAL: 1"=100'
PLOT DATE: SEPT 1997
FILE NAME: FINSECT

SHEET NO.
38
OF

IIIF-E-60



GROUND WATER MONITORING WELL #10

MAXIMUM DITCH SIDE SLOPE 3:1 HORIZONTAL:VERTICAL

ALL DITCHES SIZED TO CARRY FLOWS FROM THE 25 – YEAR STORM.

——→ DIRECTION OF FLOW

SIDE SLOPES 4:1 HORIZONTAL:VERTICAL

TOP SLOPES 2% MINIMUM AND 25% MAXIMUM.

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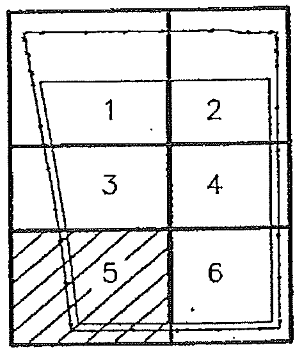

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BILLY D. SIMS

CONTOUR INTERVAL 1'
INDEX CONTOUR INTERVAL 5'
– – – BOUNDARY OF RUNOFF AREA
⇨ INDICATES DIRECTION OF RUNOFF
—·— FLOW LINE OF DITCH
(#) RUNOFF AREA
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℄ FLOW LINE
℄ CENTER LINE

GAS VENT
—//— CHAIN LINK FENCE(SEE ATTACHMENT #1)
—X— BARBED WIRE FENCE(SEE ATTACHMENT #1)

SHEET 5

REVISED SEPT. 9, 1997
REVISED AUG. 16, 1996

MARK	REVISION	BY	CK'D	DATE

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ENGINEERS - SURVEYORS
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LUFKIN, TEXAS 75915–1506 (409) 637–6061
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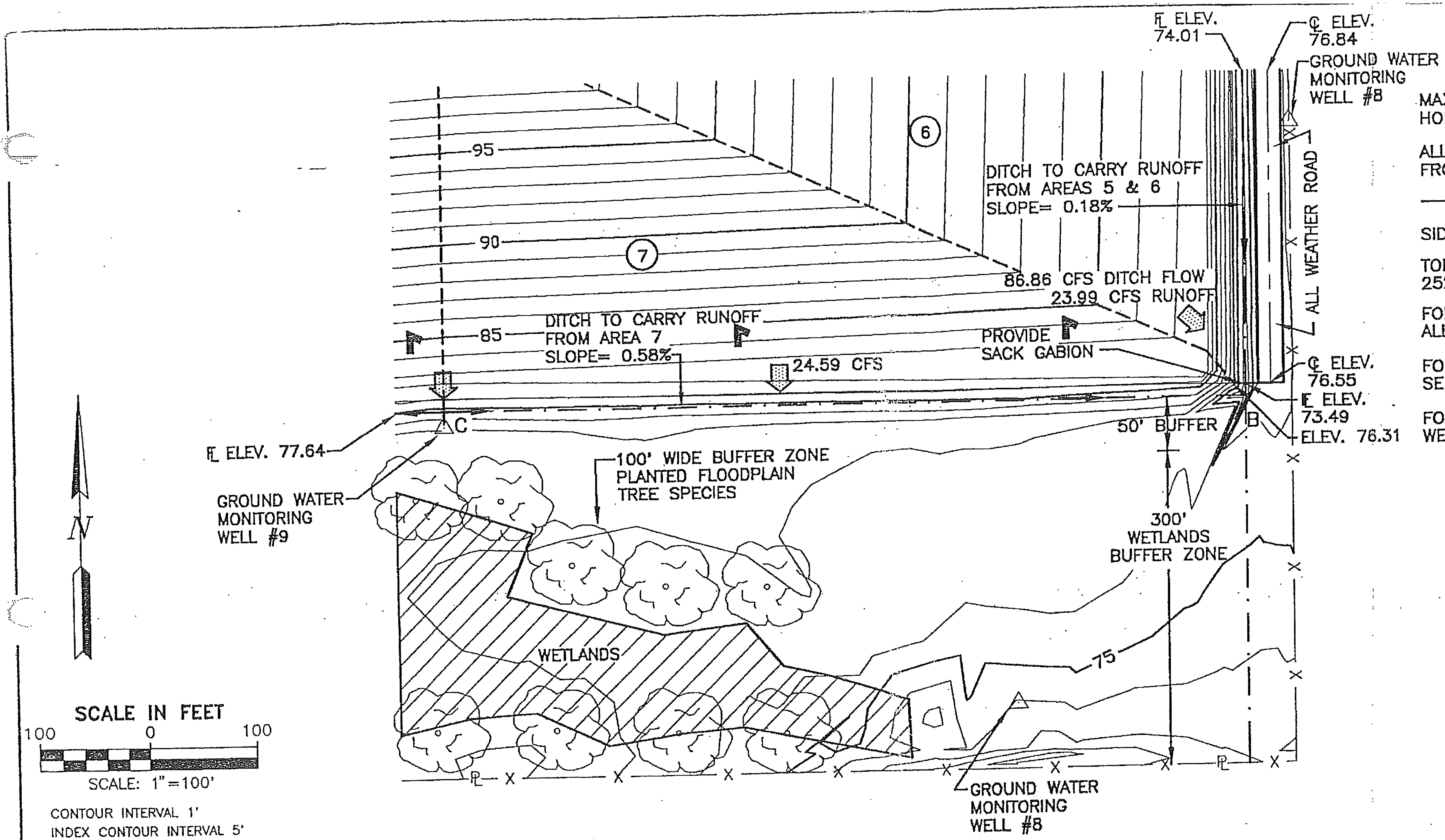
HARDIN COUNTY
LANDFILL PERMIT APPLICATION
PART III
PROJECT NAME:

SITE DEVELOPMENT PLAN
ATTACHMENT #6
FINAL DRAINAGE DETAIL
SHEET

DRAWING SCALE:
VERTICAL: NONE
HORIZONTAL: 1"=100'
PLOT DATE: SEPT. 1997
FILE NAME: FINSECT

SHEET NO.
39
OF



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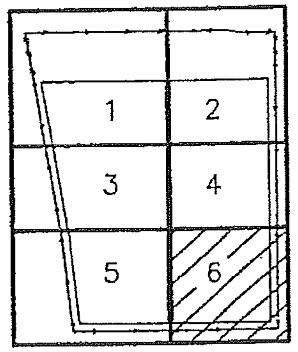
FOR MORE INFORMATION ON MONITORING WELLS SEE ATTACHMENTS #11 & #13



CONTOUR INTERVAL 1'
INDEX CONTOUR INTERVAL 5'

- – – – BOUNDARY OF RUNOFF AREA
- ⇨ INDICATES DIRECTION OF RUNOFF
- –·–·– FLOW LINE OF DITCH
- (#) RUNOFF AREA
- △ MONITORING WELL
- PL PROPERTY LINE
- FL FLOW LINE
- CL CENTER LINE
- ⚑ GAS VENT
- –//– CHAIN LINK FENCE(SEE ATTACHMENT #1)
- –X– BARBED WIRE FENCE(SEE ATTACHMENT #1)

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BILLY D. SIMS
53016

SHEET 6

REVISED SEPT. 9, 1997
REVISED AUG. 16, 1996

MARK	REVISION	BY	CK'D	DATE

KSA ENGINEERS, INC.
ENGINEERS - SURVEYORS
1313 SOUTH JOHN REDDITT DR. – P.O. BOX 151508
LUFKIN, TEXAS 75915–1508 (409) 637–6061
DALLAS LONGVIEW LUFKIN TYLER

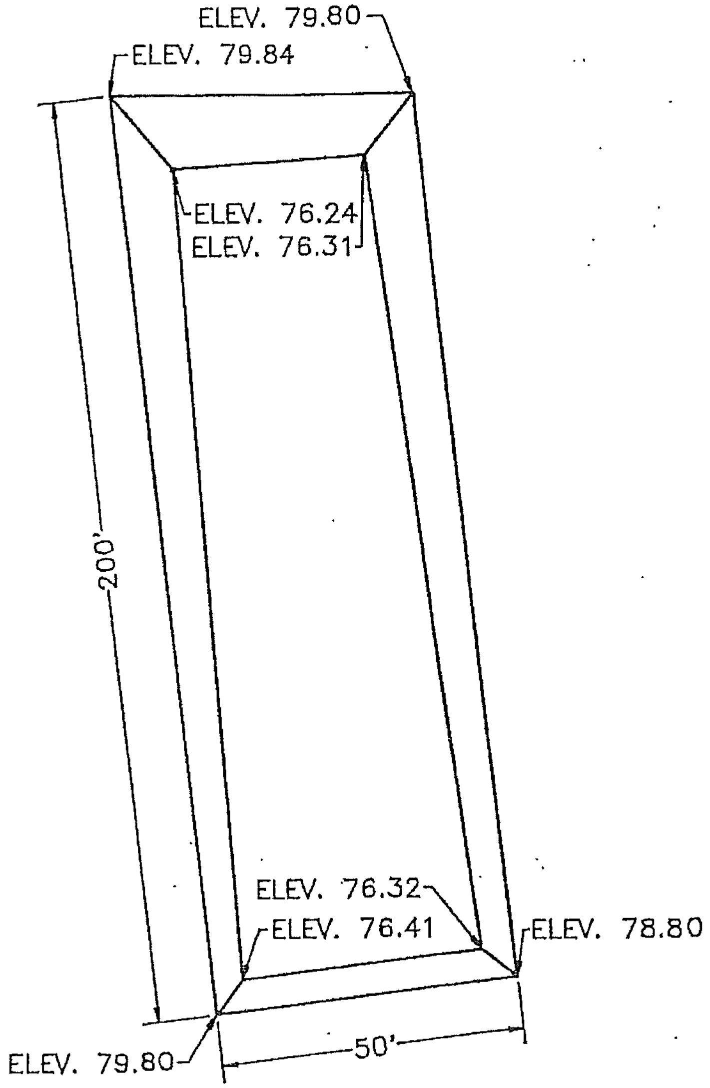
SURVEYED	DESIGNED	DRAWN	APPROVED	JOB NO.	DATE
N.A.	ALSII	DLM	BDS	HN–001	AUG. 1994

HARDIN COUNTY
LANDFILL PERMIT APPLICATION
PART III
PROJECT NAME:

SITE DEVELOPMENT PLAN
ATTACHMENT #6
FINAL DRAINAGE DETAIL
SHEET:

DRAWING SCALE:
VERTICAL: NONE
HORIZONTAL: 1"=100'
PLOT DATE: SEPT. 1997
FILE NAME: FINSECT

SHEET NO.
40
OF



NOTE: SIDE SLOPES VARY FROM APPROXIMATELY
3:1 HORIZONTAL:VERTICAL
TO 2:1 HORIZONTAL:VERTICAL

SEDIMENTATION BASIN DETAIL

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BILLY D. SIMS
50916

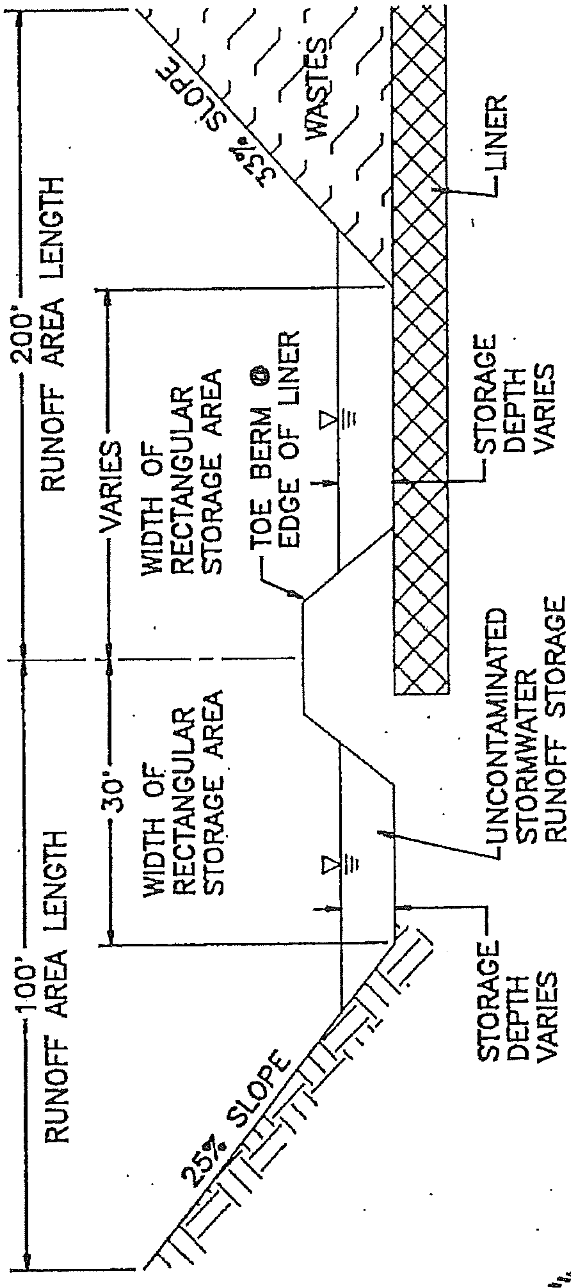
41

KSA ENGINEERS, INC.

1313 E. JOHN
REDDITT DR.
P.O. BOX 151608
LUFKIN, TEXAS
75915-1608
(409) 637-6041

SITE DEVELOPMENT PLAN
ATTACHMENT #6
SEDIMENTATION BASIN DETAIL

JOB NO.:HN-001
SCALE: NTS
FILE NM:BASIN



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BILLY D. SIMS

THIS DRAWING IS NOT TO SCALE

KSA ENGINEERS, INC.

1313 S. JOHN
REDDITT DR.
P.O. BOX 151808
LUFKIN, TEXAS
75915-1808
(409) 637-6041

SITE DEVELOPMENT PLAN
ATTACHMENT #6
TOE BERM CALCULATION DETAIL

JOB NO.:HN-001
SCALE: N.T.S.
FILE NM:TOEBERM

BILLY D. SIMS
53016

HARDIN COUNTY LANDFILL PERMIT APPLICATION
CALCULATION OF STORMWATER RUNOFF STORAGE DEPTHS
CALCUATIONS BASED ON QUADRATIC EQUATION

STORMWATER DEPTH (INCHES)	RUNOFF AREA LENGTH (FEET)	RUNOFF AREA WIDTH (FEET)	RUNOFF VOLUME (CUBIC FEET)	WIDTH OF RECTANGULAR STORAGE AREA (FEET)	EXCAVATION OR WASTE SLOPE (DECIMAL %)		A	B	C	STORAGE DEPTH (FEET)
9.00	300.00	235.00	52,875.00	10.00	0.25	0.33	826.06	2,350.00	-52,875.00	6.70
9.00	300.00	235.00	52,875.00	12.00	0.25	0.33	826.06	2,820.00	-52,875.00	6.47
9.00	300.00	235.00	52,875.00	14.00	0.25	0.33	826.06	3,290.00	-52,875.00	6.25
9.00	300.00	235.00	52,875.00	16.00	0.25	0.33	826.06	3,760.00	-52,875.00	6.04
9.00	300.00	235.00	52,875.00	18.00	0.25	0.33	826.06	4,230.00	-52,875.00	5.84
9.00	300.00	235.00	52,875.00	20.00	0.25	0.33	826.06	4,700.00	-52,875.00	5.65
9.00	300.00	235.00	52,875.00	25.00	0.25	0.33	826.06	5,875.00	-52,875.00	5.20
9.00	300.00	235.00	52,875.00	30.00	0.25	0.33	826.06	7,050.00	-52,875.00	4.80
9.00	200.00	235.00	35,250.00	10.00	0.33		356.06	2,350.00	-35,250.00	7.18
9.00	200.00	235.00	35,250.00	12.00	0.33		356.06	2,820.00	-35,250.00	6.75
9.00	200.00	235.00	35,250.00	14.00	0.33		356.06	3,290.00	-35,250.00	6.35
9.00	200.00	235.00	35,250.00	16.00	0.33		356.06	3,760.00	-35,250.00	5.98
9.00	200.00	235.00	35,250.00	18.00	0.33		356.06	4,230.00	-35,250.00	5.65
9.00	200.00	235.00	35,250.00	20.00	0.33		356.06	4,700.00	-35,250.00	5.34
9.00	200.00	235.00	35,250.00	25.00	0.33		356.06	5,875.00	-35,250.00	4.68
9.00	200.00	235.00	35,250.00	30.00	0.33		356.06	7,050.00	-35,250.00	4.14
9.00	100.00	235.00	17,625.00	10.00	0.25		470.00	2,350.00	-17,625.00	4.11
9.00	100.00	235.00	17,625.00	12.00	0.25		470.00	2,820.00	-17,625.00	3.82
9.00	100.00	235.00	17,625.00	14.00	0.25		470.00	3,290.00	-17,625.00	3.55
9.00	100.00	235.00	17,625.00	16.00	0.25		470.00	3,760.00	-17,625.00	3.31
9.00	100.00	235.00	17,625.00	18.00	0.25		470.00	4,230.00	-17,625.00	3.10
9.00	100.00	235.00	17,625.00	20.00	0.25		470.00	4,700.00	-17,625.00	2.91
9.00	100.00	235.00	17,625.00	25.00	0.25		470.00	5,875.00	-17,625.00	2.50
9.00	100.00	235.00	17,625.00	30.00	0.25		470.00	7,050.00	-17,625.00	2.18

IESI HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS
TCEQ PERMIT NO. MSW 2214A

PART III SITE DEVELOPMENT PLAN

ATTACHMENT 6A
EROSION AND SEDIMENT CONTROL PLAN

Prepared for

IESI TX Landfill LP

March 2007

Revised June 2007

KENNETH J. WELCH
60773

6/22/07

BIGGS & MATHEWS ENVIRONMENTAL
1700 Robert Road, Suite 100 ◆ Mansfield, Texas 76063 ◆ 817-563-1144

Kathleen Hartnett White, *Chairman*
Larry R. Soward, *Commissioner*
H. S. Buddy Garcia, *Commissioner*
Glenn Shankle, *Executive Director*

TEXAS COMMISSION ON ENVIRONMENTAL QUALITY

Protecting Texas by Reducing and Preventing Pollution

September 20, 2007

Mr. John Gustafson
Vice President
IESI Corporation
2301 Eagle Parkway, Suite 200
Fort Worth, TX 76177-2322

Re: Municipal Solid Waste (MSW) – Hardin County
IESI Hardin County Landfill - MSW Permit No. 2214A
Permit Modification - Erosion and Sedimentation Controls
WWC No. 11873215; RN103759643 / CN601668486

Dear Mr. Gustafson:

We have reviewed your application for a municipal solid waste permit modification dated September 21, 2006 and the revisions dated March 25, 2007, June 22, 2007, and August 16, 2007 requesting to update the surface water drainage plans to address the requirements of Title 30 Texas Administrative Code (30 TAC) Chapter 330 Subchapter G for erosion and sedimentation controls. The information presented is technically sufficient for a municipal solid waste permit modification and is approved with changes, in accordance with 30 TAC Section 305.70(g)(1)(A).

Enclosed is a copy of the above referenced modification which is now part of your permit and should be attached thereto. The documentation prepared and submitted to support the modification request shall be considered as requirements of the permit.

If you have questions concerning this matter, please contact Mr. Chandra S. Yadav at (512) 239-6727. When addressing written correspondence, please use Mail Code 124 (MC 124).

This action is taken under authority delegated by the Executive Director of the Texas Commission on Environmental Quality.

Sincerely,

Richard C. Carmichael

Richard C. Carmichael, Ph.D., P.E.
Manager, Municipal Solid Waste Permits Section
Waste Permits Division

RCC/CY/ff

cc: Mr. Kenneth Welch, P.E., Biggs & Mathews Environmental, Mansfield, Texas

Enclosure

P.O. Box 13087 • Austin, Texas 78711-3087 • 512-239-1000 • Internet address: www.tceq.state.tx.us

TEXAS COMMISSION ON ENVIRONMENTAL QUALITY

MODIFICATION TO
MUNICIPAL SOLID WASTE PERMIT NO. 2214A
IESI HARDIN COUNTY LANDFILL

Municipal Solid Waste Permit No. 2214A is hereby modified as follows:

Description of Change:

> Revisions to the surface water drainage plan to address the requirements of Title 30 Texas Administrative Code (30 TAC) Chapter 330 Subchapter G for erosion and sedimentation controls.

The details of this permit modification are contained in the application submittals dated September 21, 2006 and the revisions dated March 25, 2007, June 22, 2007, and August 16, 2007.

Part of Permit Modified:

> Part III Site Development Plan:
> Cover page and text pertaining to Attachment 6A on page 6-ii of Attachment 6,
> Attachment 6A - Erosion and Sediment Control Plan is added, and
> Appendix 6A-A to the SDP Attachment 6A, is added.

In accordance with 30 TAC Section 305.70(g)(1)(A), the permit modification includes the following condition:

- On Page 6A-1 in Attachment 6A, the following text is deleted,

  "Therefore, temporary erosion control structures may need to be installed on existing intermediate cover areas to control erosion and minimize soil loss if these existing intermediate cover areas have less than 60% vegetative coverage. However, existing intermediate cover areas that have existing well established vegetation (at least 60% coverage) will not be disturbed to construct temporary erosion control features."

  and is replaced by the following text,

  "Temporary structural erosion controls described in Attachment 6A shall apply to all intermediate cover on top domes and exterior side slopes. Intermediate cover existing at the time of Permit Modification issuance shall have the described temporary structural erosion controls installed within 180 days of the date of Permit Modification issuance."

This modification is a part of Permit No. 2214A and should be attached thereto.

APPROVED, ISSUED, AND EFFECTIVE in accordance with 30 TAC Section 305.70(l) and Chapter 330 Subchapter G.

ISSUED DATE: SEP 20 2007

For the Commission

CONTENTS

STATE OF TEXAS
KENNETH J. WELCH
60773
REGISTERED PROFESSIONAL ENGINEER

6/22/07

NARRATIVE

This attachment presents temporary erosion and sediment control structures for the intermediate cover phase of landfill development. Temporary means the time between the construction of intermediate cover and the construction of final cover or the placement of additional waste, as the case may be. Intermediate topslope surfaces and external side slopes, for the purposes of compliance with 30 TAC §330.305(d) are:

a) those above grade slopes that directly drain to the site perimeter stormwater management system (i.e., areas where the stormwater directly flows to a perimeter channel or detention pond),

b) have received intermediate or final cover; and,

c) have either reached their permitted elevation, or will subsequently remain inactive for longer than 180 days.

Slopes which drain to ongoing waste placement, pre-excavated areas, areas that have received only daily cover or areas under construction which have not received waste are not covered under this appendix.

EXISTING CONDITIONS SUMMARY

The stormwater runoff from interior intermediate slopes flows to a temporary interior stormwater channel and sump. This stormwater channel flows around the constructed cells to a temporary stormwater sump. The temporary stormwater channel and sump are protected by a temporary diversion berm which prevents runon into the temporary stormwater channel from areas outside the constructed limits of waste. As exterior intermediate cover slopes of sectors are developed, temporary erosion control measures will be constructed. Temporary erosion control structures will be installed over existing intermediate cover areas, if required, within 180 days from the date of this permit modification issuance. Existing intermediate cover areas that have existing well established vegetation (approximately 60% coverage), or areas that will have final cover constructed within 180 days from issuance of this permit modification will not be required to construct temporary erosion control features.

The IESI Hardin County Landfill has a stormwater pollution prevention plan (SWPPP), prepared consistent with the TPDES general permit. The SWPPP is up to date and maintained in the Site Operating Record. The SWPPP provides detailed Best Management Practices (BMPs) including training and implementation strategies to reduce the potential of pollutants in stormwater discharge. This plan also includes detailed stormwater and erosion control measures for current landfill construction activities.

EROSION AND SEDIMENT CONTROL LANDFILL COVER PHASES

The purpose of this section is to define the landfill cover phases and where they are addressed throughout the IESI Hardin County Landfill permit:

> Daily Cover – Daily cover is defined in §330.165(a). Daily cover consists of six inches of well-compacted earthen material not previously mixed with garbage, rubbish, or other solid waste applied at the end of each operating day. The placement and erosion control practices for daily cover areas are defined in Part IV - Site Operating Plan.
>
> Intermediate Cover – Intermediate cover is defined in §330.165(c). Intermediate cover consists of at least 12 inches of suitable earthen material and is graded and maintained to prevent erosion and ponding of water. The placement requirements and erosion control practices for intermediate cover areas are defined in this attachment.
>
> Final Cover – Final cover is defined in Subchapter K. The placement and erosion control practices for final cover areas are defined in Attachment 6. Final cover at the IESI Hardin County Landfill will be managed as provided for in the closure and postclosure plan required by 30 TAC 330 Subchapter K, Closure and Post-closure.

BEST MANAGEMENT PRACTICES

Vegetation and temporary erosion control structures provide the most effective means to reduce the amount of soil loss during operation of the landfill. Best management practices utilized for erosion and sediment control may be broadly categorized as nonstructural and structural controls. Nonstructural controls addressing erosion typically include, but are not limited to, the following:

- Minimization of the disruption of the natural features, drainage, topography, vegetative cover features
- Phased development to minimize the area of bare soil exposed at any given time
- Plans to disturb only the smallest area necessary to perform current activities
- Plans to confine sediment to the construction area during the construction phase
- Scheduling of construction activities during the time of year with the least erosion potential, when applicable
- Specific plans for the stabilization of exposed surfaces in a timely manner

Structural controls are preventative and also mitigative since they control erosion and sediment movement. Structural controls addressing erosion include, but are not limited to, the following:

- Vegetative and Non-Vegetative Stabilization. A soil stabilization and vegetation schedule is provided in this attachment.

- Check Dams. Check dams may be constructed using gravel, rock, gabions, compost socks, or sand bags to reduce flow velocity and therefore erosion in a perimeter channel or detention pond.

- Filter Berms. Filter berms may be constructed of mulch, woodchips, brush, compost, shredded woodwaste, or similar materials. Mesh socks may be filled with compost, mulch, woodchips, brush, shredded woodwaste, or similar materials. Filter berms or filled mesh socks may be installed at the bottom of slopes, throughout the perimeter drainage system, and on sideslopes. The maximum drainage area to the filter berm or filled mesh sock should not exceed two acres. Runoff must not be allowed to run under or around the filter berm or filled mesh sock.

- Baled Hay. Hay bales, straw bales, or baled hay shall be a minimum of 30 inches in length. Hay bales shall be bound by either wire or nylon or polypropylene string. The hay bales shall be composed entirely of vegetable matter and free of seed. Hay bales shall be embedded in the soil a minimum of 4 inches and where possible one-half the height of the hay bale. Hay bales should not be used for more than one year before being replaced.

- Sediment Traps. Sediment traps are small excavated areas that function as a sediment basin. Sediment traps allow for the settling of suspended sediment in stormwater runoff. Sediment traps may be constructed in perimeter channels, temporary internal channels, and at entrances to detention ponds. The maximum drainage area contributing to a sediment trap should not exceed 10 acres.

- Temporary Sediment Control Fence or Silt Fence. Silt fences or fabric filter fences may be used where there is sheet flow. The maximum drainage area to the silt fence should not exceed the manufacturer's specification but in no case be greater than 0.5 acre per 100 feet of fence. To ensure sheet flow, a gravel collar or level spreader may be used upslope of the silt fence.

- Swales. These structures will be constructed of a material with the top 6 inches capable of sustaining native plant growth. Rolled erosion control mats or blankets made from natural or synthetic fiber, grass, or compost/mulch/straw blankets, for example, may be used as erosion protection along the flowline, if necessary. These structures direct the flow to the drainage system. The use of these structures is to decrease down slope velocities of runoff that could cause erosion on the intermediate cover slopes.

- Letdown Chutes. Letdown chutes are bermed conveyance structures constructed on the intermediate cover slopes. Flow will be directed to the letdown chutes via swales, then conveyed to the perimeter drainage system.

The letdown chutes will be lined with HDPE geomembrane, turf reinforcement mats, blankets, riprap, concrete, gabions, or other appropriate material.

Erosion will be controlled by vegetation on topslopes, sideslopes, and in drainage conveyance structures with flow velocities less than or equal to 5 fps. For drainage conveyance structures with flow velocities greater than 5 fps, turf reinforcement, rock riprap, concrete, gabions, or other appropriate materials will be used for surface reinforcement.

During site development, both structural and non-structural BMPs will be employed to control erosion. Examples of erosion and sedimentation control features that will be used during the phased development of the site are shown on Attachments 6A-7 through 6A-9.

The potential for wind erosion of the intermediate cover surface will be mitigated through the placement of the temporary intermediate cover erosion control measures and establishment of the vegetative cover. Temporary measures to be used if wind erosion is observed include surface roughening, surface wetting, application of tackifiers, or hydromulching the intermediate cover surface.

SOIL STABILIZATION AND VEGETATION SCHEDULE

The soil stabilization and vegetation schedule is as follows:

- Areas that will remain inactive for periods greater than 180 days will receive intermediate cover.
- Intermediate cover on slopes will be stabilized by tracking into the slope. Soil stabilization can be enhanced by mulching, the addition of soil tackifiers, soil treatment, or any combination of these measures. The intermediate cover will be graded to provide positive drainage.
- Temporary erosion control structures will be installed within 180 days from when intermediate cover is constructed.
- The intermediate cover area will be seeded or sodded as soon as practical, following placement of intermediate cover and will be documented in the site operating record. Vegetative cover will be established over the intermediate cover areas or additional temporary erosion control measures consisting of placement of mulch, woodchips, or compost will be placed within 180 days following placement of intermediate cover.
- Mulch, woodchips, compost, or similar materials may be used as a layer placed over the intermediate cover to protect the exposed soil surface from erosive forces and conserve soil moisture until vegetation can be established. The mulch, woodchips, compost, or similar materials will be used to stabilize recently graded or seeded areas. The mulch, woodchips, compost, or similar materials will be spread evenly over a recently seeded area and tracked into the surface to

protect the soil from erosion and moisture loss, if required to promote the establishment of vegetation. These materials are not required for the establishment of vegetation on the intermediate cover; however, may be used if the IESI Hardin County Landfill determines they are needed to promote vegetative growth or to provide additional erosional stability to the intermediate cover surface. These materials will vary in thickness but will not be placed to a thickness to inhibit vegetative growth. These materials will be placed such that 25% of the soil is visible through the mulch, woodchips, compost, or similar materials.

- The intermediate cover and temporary erosion control structures will be maintained as detailed in the Stormwater System Maintenance Plan.
- Final cover will be constructed as the site develops. Temporary erosion control features will be removed as permanent erosion control structures are constructed.

STORMWATER SYSTEM MAINTENANCE PLAN

The IESI Hardin County Landfill will restore and repair temporary stormwater systems such as channels, drainage swales, chutes, and flood control structures in the event of wash-out or failure. In addition, the BMPs discussed in this attachment will also be replaced or repaired in the event of failure. Excessive sediment will be removed, as needed, so that the drainage structures function as designed. Site inspections by landfill personnel will be performed weekly or within 48 hours of a rainfall event of 0.5 inches or more.

The following items will be evaluated during the inspections:

- Erosion of intermediate cover areas, perimeter ditches, temporary chutes, swales, detention ponds, berms, and other drainage features
- Settlement of intermediate cover areas, final cover areas, perimeter ditches, chutes, swales, and other drainage features
- Silt and sediment build-up in perimeter ditches, chutes, swales, and detention ponds
- Presence of ponded water on intermediate cover or behind temporary erosion control structures
- Obstructions in drainage features
- Presence of erosion or sediment discharge at offsite stormwater discharge locations
- Temporary erosion and sediment control features

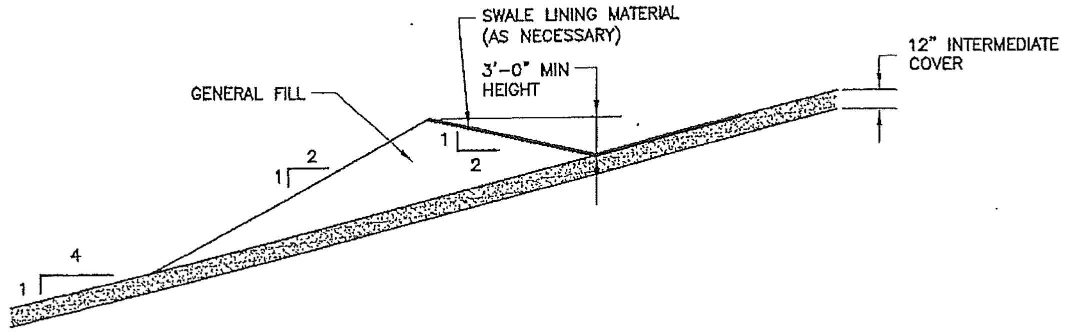
Maintenance activities will be performed to correct damaged or deficient items noted during the site inspections. These activities will be performed as soon as possible after the inspection. The time frame for correction of damaged or deficient items will vary based on weather, ground conditions, and other site-specific conditions.

Maintenance activities will consist of the following, as needed:

- Placement of additional temporary or permanent vegetation
- Placement, grading, and stabilization of additional soils in eroded areas or in areas which have settled
- Replacement of riprap or other structural lining
- Removal of obstructions from drainage features
- Removal of silt and sediment build-up from the temporary erosion control structures
- Removal of ponded water on the intermediate cover or behind temporary erosion control structures
- Repairs to erosion and sedimentation controls
- Installation of additional erosion and sedimentation controls

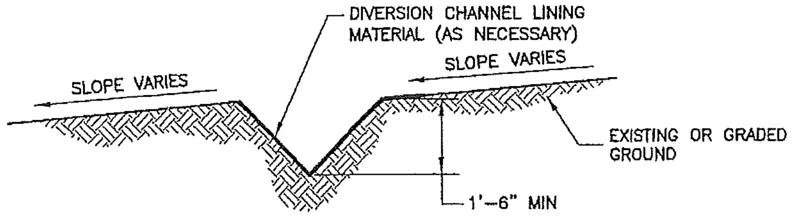
Inspection, maintenance, and recordkeeping frequencies and techniques are discussed below:

- Site inspections by landfill personnel will be performed weekly or within 48 hours of a rainfall event of 0.5 inches or more.
- Documentation of the inspection will be included in the Site Operating Record.
- Documentation of maintenance activities that were performed to correct damaged or deficient items noted during the site inspections will be included in the Site Operating Record.
- Landfill personnel will be trained to perform inspections, install and maintain temporary erosion control structures.



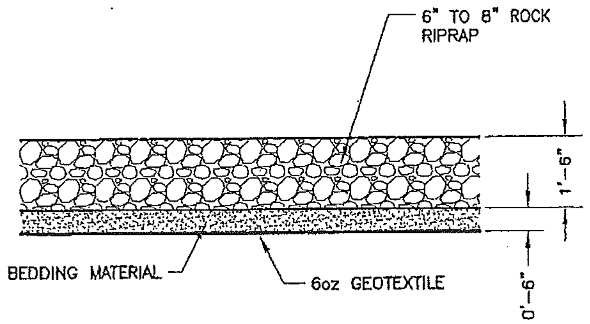
TEMPORARY DRAINAGE SWALE TD1 6A-7

NTS



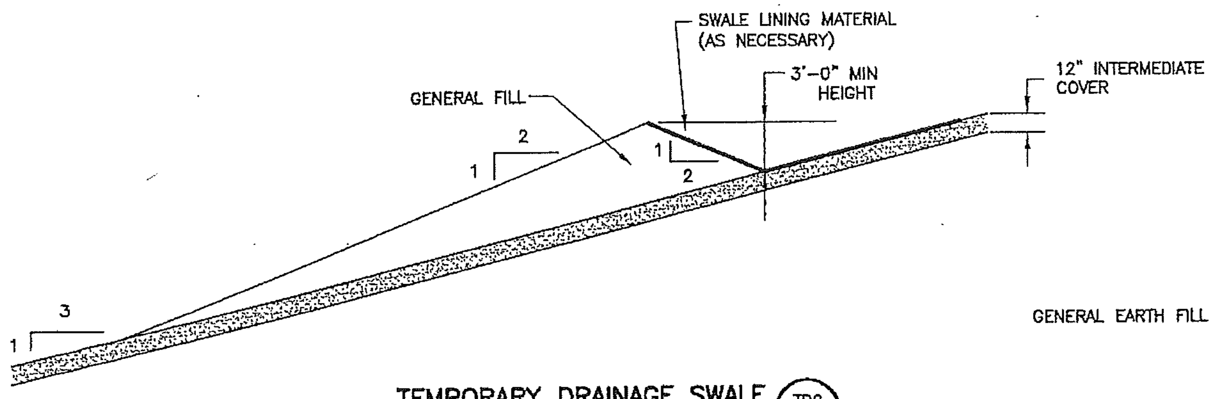
TEMPORARY DIVERSION CHANNEL TD4 6A-7

NTS



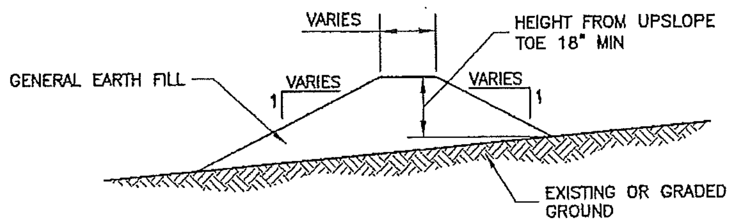
ROCK RIPRAP TD6 6A-7

NTS



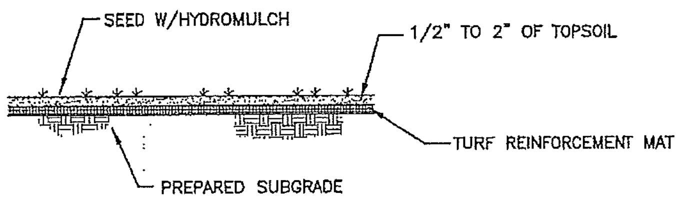
TEMPORARY DRAINAGE SWALE TD2 6A-7

NTS



TEMPORARY DIVERSION BERM TD5 6A-7

NTS

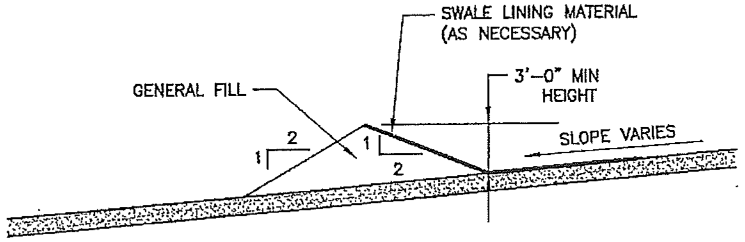


TEMPORARY TURF REINFORCEMENT MATTING TD7 6A-7

NTS

NOTE:

1. LINING MATERIAL FOR THE TEMPORARY DRAINAGE SWALES OR THE TEMPORARY DIVERSION CHANNEL, IF NECESSARY, WILL BE TURF REINFORCEMENT MATTING OR OTHER SUITABLE MATERIALS.



TEMPORARY DRAINAGE SWALE TD3 6A-7

NTS

TEMPORARY EROSION CONTROL STRUCTURES

1. TEMPORARY EROSION CONTROL STRUCTURE DETAILS DEPICT VARIOUS TYPES OF EROSION CONTROL FEATURES FOR CURRENT AND FUTURE DEVELOPMENT.
2. ALL TEMPORARY EROSION CONTROL STRUCTURES SHOWN MAY NOT BE CONSTRUCTED DEPENDING ON SITE CONDITIONS.
3. LANDFILL WILL SELECT EROSION CONTROL DETAILS TO BE USED FOR SITE SPECIFIC CONDITIONS.
4. ACTUAL DIMENSIONS OF TEMPORARY EROSION CONTROL STRUCTURES MAY VARY BASED ON SITE CONDITIONS.

KENNETH J. WELCH
60773

6/22/07

ISSUED FOR PERMITTING PURPOSES ONLY

REVISIONS

REV	DATE	DESCRIPTION	DWN BY	DES BY	CHK BY	APP BY
1	06/07	RESPONSE TO 1ST NOD	WCN	ALN	KJW	KJW

TEMPORARY EROSION CONTROL
STRUCTURES

IESI HARDIN COUNTY LANDFILL
TCEQ PERMIT NO. MSW-2214A
DRAINAGE PERMIT MODIFICATION

BIGGS & MATHEWS
ENVIRONMENTAL
CONSULTING ENGINEERS
ARLINGTON • DALLAS
MANSFIELD • WICHITA FALLS
817-563-1144

DSN. KJW
DWN. FAW
CHK. KJW
DATE : 03/07
SCALE : GRAPHIC
DWG : Erosion features.dwg
ATTACHMENT 6A-7

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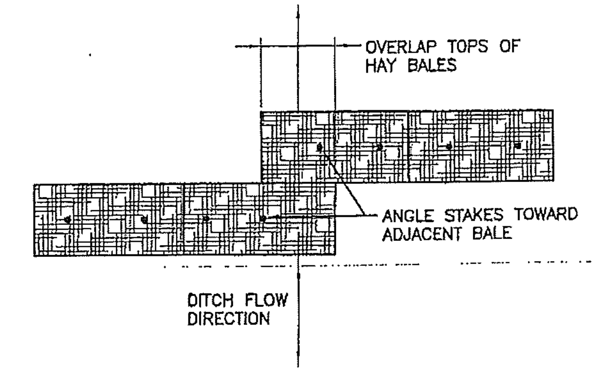
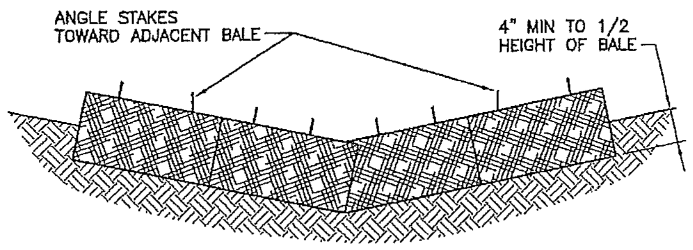


PLAN VIEW



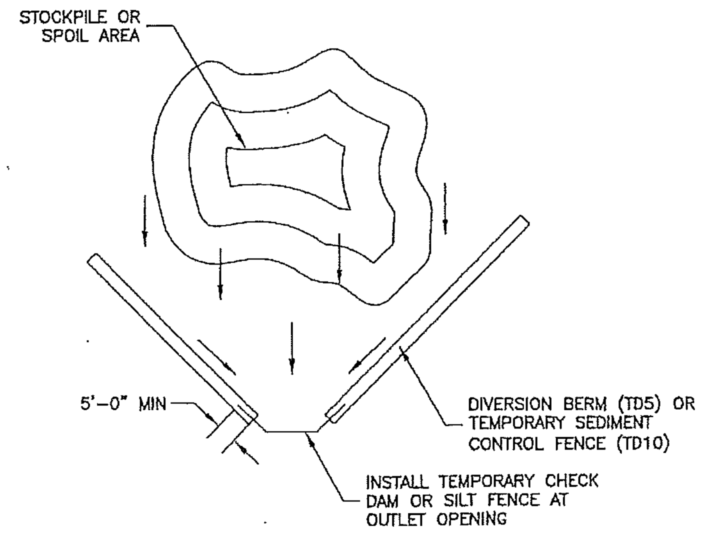
PROFILE VIEW

BALED HAY FOR EROSION CONTROL TD8 6A-8

NTS

HAY BALE NOTES:

1. HAY BALES SHALL BE BOUND BY EITHER WIRE OR NYLON OR POLYPROPYLENE STRING.
2. HAY BALES SHALL BE EMBEDDED IN THE SOIL A MINIMUM OF 4" AND WHERE POSSIBLE 1/2 THE HEIGHT OF THE BALE.

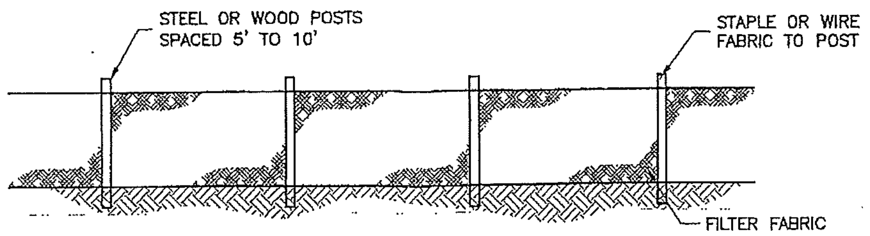


STOCKPILE EROSION CONTROL TD9 6A-8

NTS

NOTE:

1. CONSTRUCT DIVERSION DIKE TO DIVERT STORMWATER RUN-OFF FROM STOCKPILE OR SPOIL AREA THROUGH CHECK DAM, HAY BALES, OR SILT FENCE.

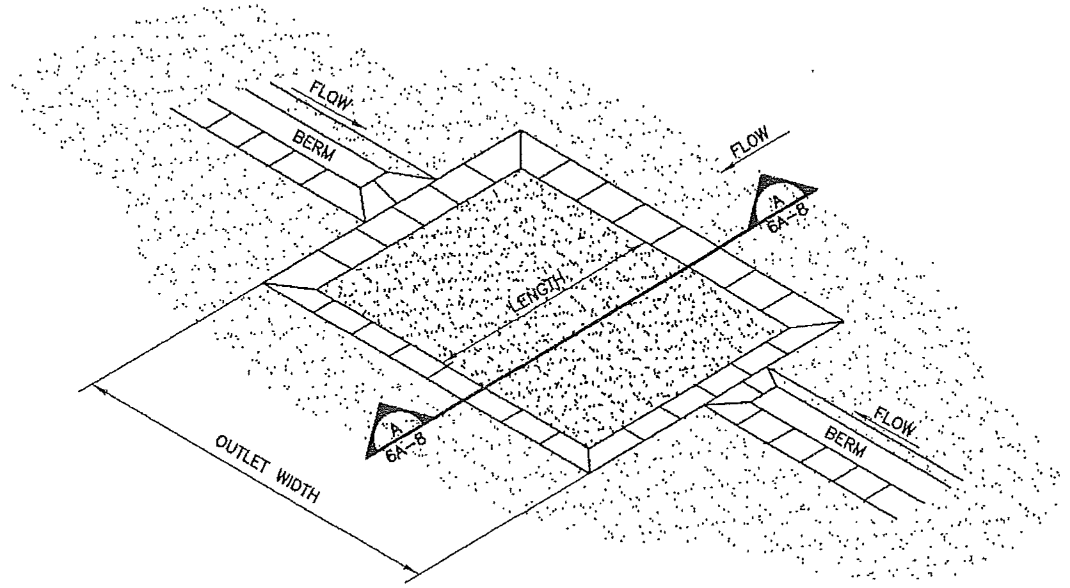


TEMPORARY SEDIMENT CONTROL (SILT) FENCE TD10 6A-8

NTS

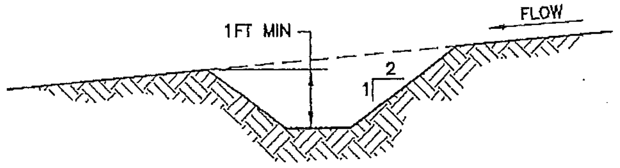
SILT FENCE NOTES:

1. MAXIMUM DRAINAGE AREA TO THE FENCE SHOULD NOT EXCEED THE MANUFACTURER'S SPECIFICATION BUT IN NO CASE BE GREATER THAN 0.5 ACRE PER 100 FEET OF FENCE.
2. TO ENSURE SHEET FLOW, A GRAVEL COLLAR OR LEVEL SPREADER MAY BE USED UPSLOPE OF THE SILT FENCE.



SEDIMENT TRAP PLAN

NTS



SEDIMENT TRAP SECTION A 6A-8

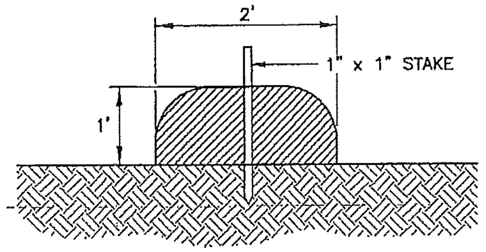
NTS

SEDIMENT TRAP TD11 6A-8

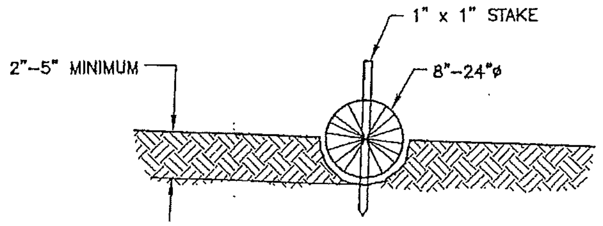
NTS

NOTE:

1. OULET INTO STABILIZED AREA (VEGETATION, ROCK, ETC.)
2. THE MAXIMUM AREA CONTRIBUTING TO A SEDIMENT TRAP SHOULD BE LESS THAN 10 ACRES.



OPTION 1



OPTION 2

FILTER BERM TD12 6A-8

NTS

FILTER BERM NOTES:

1. FILTER BERMS MAY BE CONSTRUCTED OF MULCH, WOODCHIPS, BRUSH, COMPOST, SHREDDED WOODWASTE, OR SIMILAR MATERIALS.
2. FILTER BERMS MAY ALSO CONSIST OF MESH SOCKS FILLED WITH MULCH, WOODCHIPS, BRUSH, COMPOST, SHREDDED WOODWASTE, OR SIMILAR MATERIALS
3. RUNOFF MUST NOT BE ALLOWED TO RUN UNDER OR AROUND THE COMPOST FILTER BERM.
4. STAKE WILL BE PLACED 2-5" DEEP.
5. MAXIMUM DRAINAGE AREA TO THE FILTER BERM SHOULD NOT EXCEED 2 ACRES.

TEMPORARY EROSION CONTROL STRUCTURES

1. TEMPORARY EROSION CONTROL STRUCTURE DETAILS DEPICT VARIOUS TYPES OF EROSION CONTROL FEATURES FOR CURRENT AND FUTURE DEVELOPMENT.
2. ALL TEMPORARY EROSION CONTROL STRUCTURES SHOWN MAY NOT BE CONSTRUCTED DEPENDING ON SITE CONDITIONS.
3. LANDFILL WILL SELECT EROSION CONTROL DETAILS TO BE USED FOR SITE SPECIFIC CONDITIONS.
4. ACTUAL DIMENSIONS OF TEMPORARY EROSION CONTROL STRUCTURES MAY VARY BASED ON SITE CONDITIONS.

KENNETH J. WELCH
60773
6/22/07

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1	06/07	RESPONSE TO 1ST NOD	WCM	ALN	KJW	KJW

TEMPORARY EROSION CONTROL
STRUCTURES

IESI HARDIN COUNTY LANDFILL
TCEQ PERMIT NO. MSW-2214A
DRAINAGE PERMIT MODIFICATION

BIGGS & MATHEWS
ENVIRONMENTAL
CONSULTING ENGINEERS
ARLINGTON ♦ DALLAS
MANSFIELD ♦ WICHITA FALLS
817-563-1144

DSN. KJW
DWN. FAW
CHK. KJW
DATE : 03/07
SCALE : GRAPHIC
DWG : Erosion features.dwg
ATTACHMENT
6A-8

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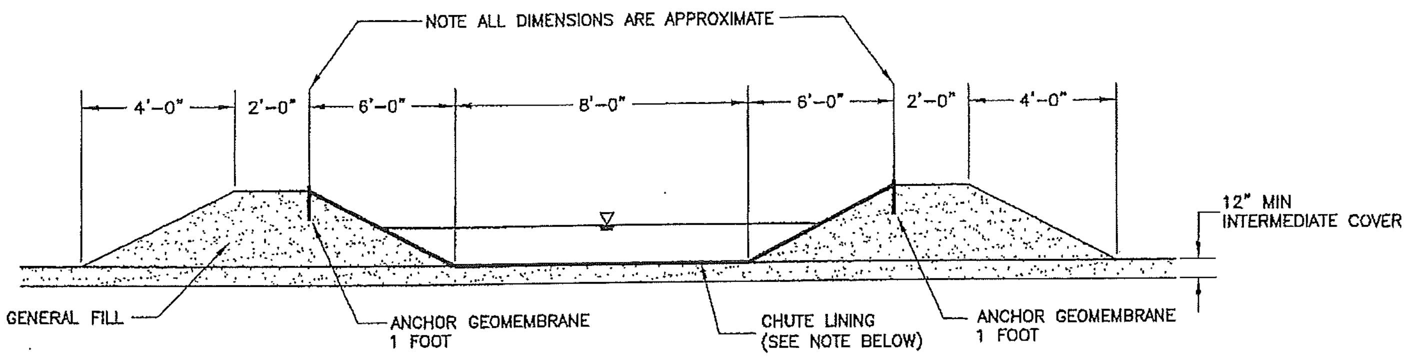
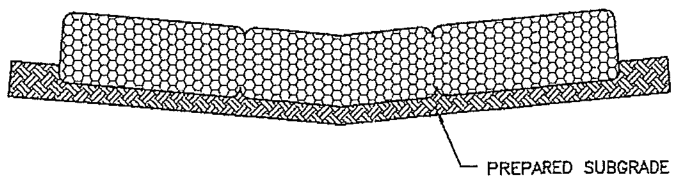


TEMPORARY LETDOWN CHUTE TD13 6A-9

NTS

NOTE:

CHUTE LINING COULD CONSIST OF (BUT NOT LIMITED TO) ONE OF THE FOLLOWING: TURF REINFORCEMENT, SACRIFICIAL GEOMEMBRANE, GABIONS, OR ROCK RIPRAP.

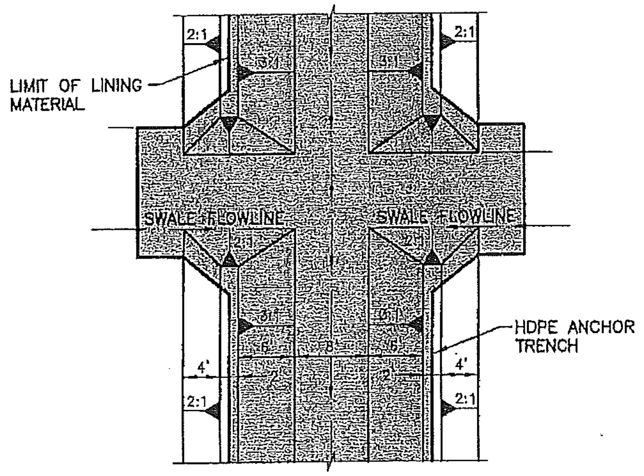


CHECK DAM TD15 6A-9

NTS

CHECK DAM NOTES:

1. MAY BE CONSTRUCTED USING GRAVEL, ROCK, GABIONS, COMPOST SOCKS, OR SAND BAGS.
2. PLACED ON PREPARED SUBGRADE OR BEDDING MATERIAL ALONG THE CONTOUR AT 0% GRADE OR AS NEAR AS POSSIBLE.
3. TOP WIDTH OF TWO FEET MINIMUM.
4. SIDESLOPES 2H:1V OR FLATTER.
5. MAY BE USED WHEN CONTRIBUTING DRAINAGE AREAS ARE LESS THAN 10 ACRES. MULTIPLE CHECK DAMS MAY BE INSTALLED IF DRAINAGE AREAS ARE GREATER THAN 10 ACRES.
6. CHECK DAMS SHOULD BE USED WHEN THE VOLUME OF RUNOFF IS TOO GREAT FOR OTHER EROSION CONTROL FEATURES (i.e. SILT FENCES, HAY BALES).



SWALE/CHUTE CONFLUENCE TD14 6A-9

NTS

TEMPORARY EROSION CONTROL STRUCTURES

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3. LANDFILL WILL SELECT EROSION CONTROL DETAILS TO BE USED FOR SITE SPECIFIC CONDITIONS.
4. ACTUAL DIMENSIONS OF TEMPORARY EROSION CONTROL STRUCTURES MAY VARY BASED ON SITE CONDITIONS.

KENNETH J. WELCH
60773

6/22/07

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REV	DATE	DESCRIPTION	DWN BY	DES BY	CHK BY	APP BY

TEMPORARY EROSION CONTROL STRUCTURES

IESI HARDIN COUNTY LANDFILL
TCEQ PERMIT NO. MSW-2214A
DRAINAGE PERMIT MODIFICATION

BIGGS & MATHEWS
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DSN. KJW	DATE :	03/07	ATTACHMENT	
DWN. FAW	SCALE :	GRAPHIC	6A-9	
CHK. KJW	DWG :	Erosion features.dwg		

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ATTACHMENT 6A

APPENDIX 6A-A

INTERMEDIATE COVER EROSION CONTROL STRUCTURE DESIGN

STATE OF TEXAS
KENNETH J. WELCH
60773
REGISTERED
PROFESSIONAL ENGINEER

6/22/07

Includes pages 6A-A-1 through 6A-A-28

CONTENTS

STATE OF TEXAS
KENNETH J. WELCH
60773
REGISTERED PROFESSIONAL ENGINEER

6/22/07

NARRATIVE

This appendix presents the supporting documentation to evaluate and design temporary erosion and sediment control structures for the intermediate cover phase of landfill development.

As intermediate cover is constructed, temporary chutes, and berms/swales will be constructed to prevent erosion and sedimentation. An erosion control feature (i.e., filter berms, rock berms, hay bales, or equivalent) will be constructed at the toe of filled areas to minimize erosion on existing grassed slopes. The filter berm or other erosion control feature can be removed once the intermediate cover has satisfactory grass coverage (60% coverage or greater) and temporary erosion controls structures have been established. An existing conditions summary and Best Management Practices are included in Attachment 6A. Example intermediate cover drainage calculations are included in this appendix for use in site operations.

INTERMEDIATE COVER EVALUATION

The intermediate cover evaluation is based on the Universal Soil Loss Equation (USLE) following Soil Conservation Service (SCS) procedures. The evaluation is based on a 12-inch thick intermediate cover layer with 60% vegetated cover. Sample calculations for the soil loss for intermediate cover on external 6%, 25%, and 33.3% slopes have been provided on pages 6A-A-3 through 6A-A-11.

SHEET FLOW DESIGN

The sheet flow calculations are presented for external 6%, 25%, and 33.3% slope configurations. The permissible non-erodible velocities should be less than 5 ft/sec (clayey soil) or 4 ft/sec (sandy soil) on vegetated intermediate cover. The Manning's Equation and Rational Method were used to calculate sheet flow velocity.

TEMPORARY DRAINAGE SWALE DESIGN

The temporary drainage swales are designed for typical drainage areas and flowline slopes. The procedures in the TxDOT Hydraulic Design Manual were used to determine peak flow, flow depth, flow velocity, and swale capacity. The Rational Method and the Manning's Equation were used to calculate the design parameters.

TEMPORARY DIVERSION CHANNEL DESIGN

The temporary diversion channels are designed for typical drainage areas and flowline slopes. The procedures in the TxDOT Hydraulic Design Manual were used to determine peak flow, flow depth, flow velocity, and diversion channel capacity. The Rational Method and the Manning's Equation were used to calculate the design parameters.

TEMPORARY DRAINAGE LETDOWN DESIGN

The temporary drainage letdowns are designed for typical drainage areas on a 25% external side slope. The procedures in the TxDOT Hydraulic Design Manual were used to determine peak flow, flow depth, flow velocity, and letdown capacity.. The Rational Method and the Manning's Equation were used to calculate the design parameters.

INTERMEDIATE COVER EVALUATION

INTERMEDIATE COVER EVALUATION

SOIL LOSS

This section presents the supporting documentation for evaluation of the potential for intermediate cover soil erosion loss at the IESI Hardin County Landfill. The evaluation is based on the premise of adding excess soil to increase the time required before maintenance is needed as recommended in the EPA Solid Waste Disposal Facility Criteria Technical Manual (EPA 530-R-93-017, November 1993).

The design procedure is as follows:

1. Minimum thickness of the intermediate cover is evaluated based on the maximum soil loss of 50 tons per acre per year.

	6% slope	25% slope	33.3% slope
Maximum Sheet Flow Length	1,000 ft	300 ft	150 ft
Soil Loss	6.03 tons/acre/year	43.38 tons/acre/year	49.19 tons/acre/year

2. Soil loss is calculated using the Universal Soil Loss Equation (USLE) by following SCS procedures. The soil loss is based on 60% vegetative cover as recommended in the TNRCC, "Use of the Universal Soil Loss Equation in Final Cover/Configuration Design Procedural Handbook" (October 1993). These calculations begin on page 6A-A-6.

3. Sheet flow velocities for a 25-year storm event are calculated to be less than permissible non-erodible velocities. The supporting calculations are presented on pages 6A-A-12 and 6A-A-13.

4. Temporary vegetation for the intermediate cover areas will be native and introduced grasses with root depths of 6 inches to 8 inches.

5. Native and introduced grasses will be hydroseeded, drill seeded, or broadcast seeded with fertilizer on the disked (parallel to contours) intermediate cover layer within 180 days of placement. Erosion control measures such as silt fences, hay bales, and wattles will be used to minimize erosion until the vegetation is established. Areas that do not readily vegetate will be reseeded until vegetation is established or will use alternatives to vegetation as defined below.

SHEET FLOW VELOCITY

The sheet flow velocity calculations are presented for the external 6%, 25%, and 33.3% slope configurations. The procedures outlined in the TxDOT Hydraulic Design Manual were used to determine velocities. Maximum sheet flow lengths for all three conditions were evaluated. Calculations are provided on page 6A-A-13.

Prep by: KJW
Date: 3/25/2007

Chkd by: SAN
Date: 3/25/07

IESI Hardin County Landfill
Intermediate Cover Erosion Loss Evaluation

Required: 1. Determine the erosion loss for the intermediate cover design based on a maximum soil loss of 50 tons/acre/year.

Method: Expected soil loss is calculated using the Universal Soil Loss Equation.

References:

1. Schwab, Glen O., *Soil and Water Conservation Engineering, 3rd Ed.*, 1981.
2. Texas Department of Transportation, *Hydraulic Design Manual,* March 2004.
3. United States Soil Conservation Service, *Hydrology for Small Watersheds,* December 1989.
4. TNRCC, *Use of the Universal Soil Loss Equation in Final Cover/Configuration Design Procedural Handbook,* October 1993.

Solution: Annual Soil Loss in tons/acre/year (A) = RKLSCP

Design Parameters	External Top Slope (6%)	External Side Slope (25%)	External Side Slope (33.3%)	
Rainfall Factor (R) =	450	450	450	Hardin County
Soil Erodibility Factor (K) =	0.25	0.25	0.25	(Sandy Clay Loam)
Longest Run =	1000	300	150	ft
Slope =	6	25	33.33	%
Topographic Factor (LS) =	2.13	10.20	11.57	
Crop Management Factor (C) =	0.042	0.042	0.042	(60% vegetative cover)
Erosion Control Practice Factor (P) =	0.50	0.90	0.90	
Soil Loss (A) =	5.02	43.38	49.19	tons/acre/year

Summary: As noted in the permit drawings, the intermediate cover will be a minimum of 12 inches thick. As shown above, the maximum soil loss is 49.19 tons/acre/year, which is less than the maximum allowable soil loss of 50 tons/acre/year.

Prep by: KJW
Date: 3/25/2007

Chkd by: SAN
Date: 3/25/07

IESI Hardin County Landfill
LS Factor Calculations

Required: Determine the Length/Slope Factor based on slope length and slope gradient.

References: 1. TNRCC, *Use of the Universal Soil Loss Equation in Final Cover/Configuration Design Procedural Handbook,* October 1993.

Solution: Length/Slope Factor (LS) = $(L / 72.6)^m * (65.41*\sin 2\ \theta + 4.56 * \sin \theta + 0.065)$

LS = Length/Slope Factor
L = Slope Length (ft)
θ = radians
m = exponent dependent on the slope gradient

m = 0.2 for S <= 1.0%
0.3 for 1.0% < S <= 3.5%
0.4 for 3.5% < S < 5.0%
0.5 for S => 5.0%

Length, L (ft)	Slope, S (%)	Slope, S (ft/ft)	θ (radians)	θ (degrees)	m	LS
1000	6	17	0.060	3.434	0.5	2.13
300	25	4	0.245	14.036	0.5	10.20
150	33.33	3	0.322	18.433	0.5	11.57

Reproduced from: Texas Natural Resource Conservatioon Commission, Municipal Solid Waste Division, Use of the Universal Soil Loss Equation in Final Cover/Configuration Design: Procedural Handbook, 1993.

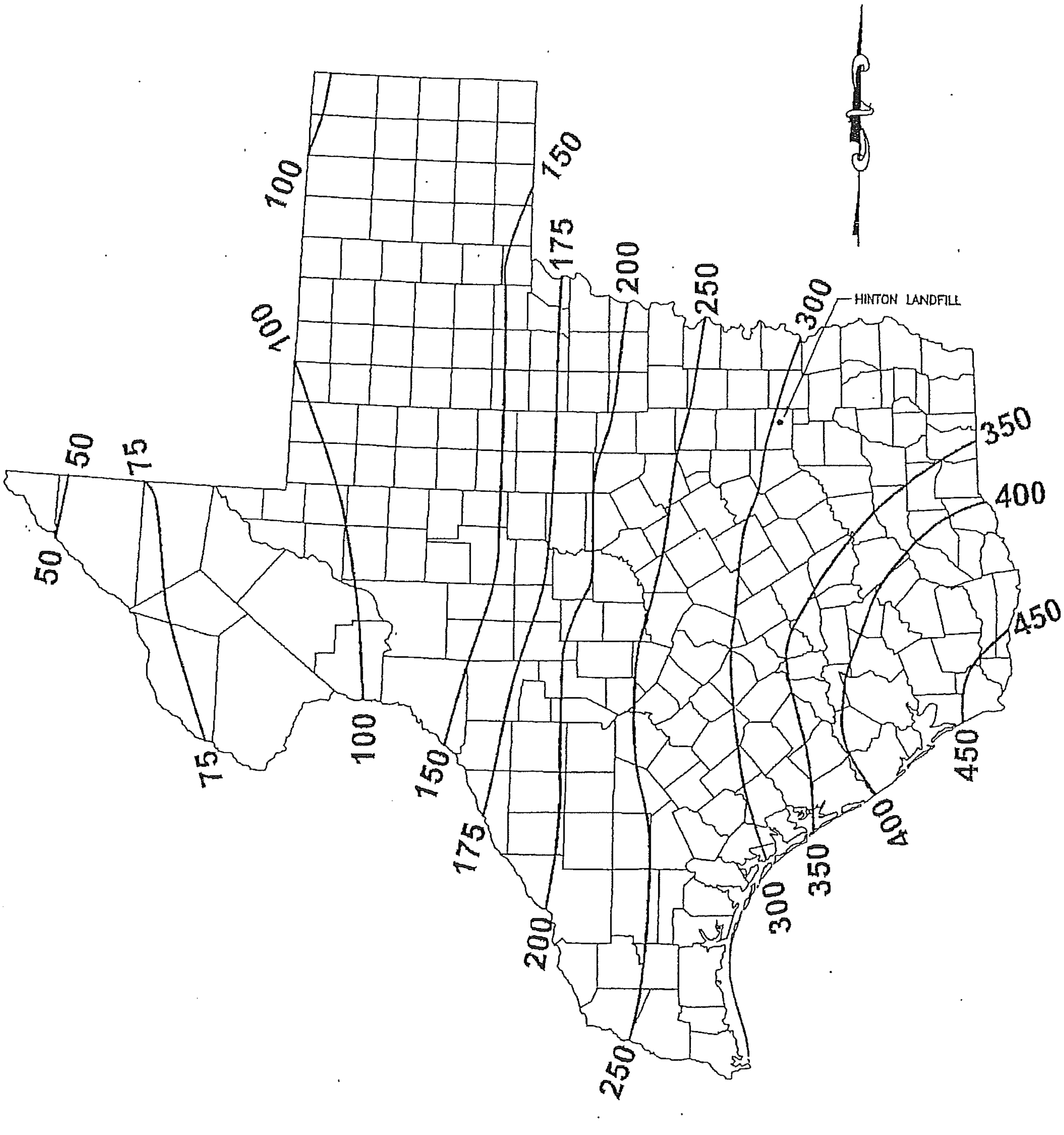


FIGURE 1 - AVERAGE ANNUAL VALUES OF THE RAINFALL EROSION INDEX

Table 1: Approximate Values of Factor K for USDA Textural Classes

Reproduced from: Texas Natural Resource Conservation Commission, Municipal Solid Waste Division, Use of the Universal Soil Loss Equation in Final Cover/Configuration Design: Procedural Handbook, 1993.

Texture Class	Organic Matter Content		
	<0.5%	2%	4%
	K	K	K
Sand	0.05	0.03	0.02
Fine Sand	0.16	0.14	0.10
Very Fine Sand	0.42	0.36	0.28
Loamy Sand	0.12	0.10	0.08
Loamy Fine Sand	0.24	0.20	0.16
Loamy Very Fine Sand	0.44	0.38	0.30
Sandy Loam	0.27	0.24	0.19
Fine Sandy Loam	0.35	0.30	0.24
Very Fine Sandy Loam	0.47	0.41	0.33
Loam	0.38	0.32	0.29
Silt Loam	0.48	0.42	0.33
Silt	0.60	0.52	0.42
Sandy Clay Loam	0.27	0.25	0.21
Clay Loam	0.28	0.25	0.21
Silty Clay Loam	0.37	0.32	0.26
Sandy Clay	0.14	0.13	0.12
Silty Clay	0.25	0.23	0.19
Clay	0.13 - 0.29		

The values shown are estimated averages of broad ranges of specific soil values. When a texture is near the borderline of two texture classes, use the average of the two K values.

Table 2: Factor C for Permanent Pasture, Range, and Idle Land[1]

Reproduced from: Texas Natural Resource Conservation Commission, Municipal Solid Waste Division, Use of the Universal Soil Loss Equation in Final Cover/Configuration Design: Procedural Handbook, 1993.

Vegetative Canopy		Cover that Contacts the Soil Surface					
Type and Height[2]	Percent Cover[3]	Percent Ground Cover					
		0	20	40	60	80	95+
No Appreciable Canopy		0.45	0.20	0.10	0.042	0.013	0.003
Tall weeds or short brush with average drop fall height of 20 in.	25	0.36	0.17	0.09	0.038	0.013	0.011
	50	0.26	0.13	0.07	0.035	0.012	0.003
	75	0.17	0.10	0.06	0.032	0.011	0.003

Extracted from: United States Department of Agriculture, AGRICULTURE HANDBOOK NUMBER 537

[1] The listed C values assume that the vegetation and mulch are randomly distributed over the entire area.

[2] Canopy height is measured as the average fall height of water drops falling from the canopy to the ground. Canopy effect is inversely proportional to drop fall height and is negligible if fall height exceeds 33 feet.

[3] Portions of total-area surface that would be hidden from view by canopy in a vertical projection (a bird's eye view).

Table 3: P Factors for Contouring, Contour Stripcropping and Terracing

Reproduced from: Texas Natural Resource Conservation Commission, Municipal Solid Waste Division, Use of the Universal Soil Loss Equation in Final Cover/Configuration Design: Procedural Handbook, 1993.

Land Slope	P Values		
%	Contouring†	Contour Stripcropping	Terracing†
2.0 to 7	0.50	0.25	0.50
8.0 to 12	0.60	0.30	0.60
13.0 to 18	0.80	0.40	0.80
19.0 to 24	0.90	0.45	0.90

(This table appeared in SCS (5), p.9)

† Contouring and terracing columns are suitable for MSWLF cover. Contour stripcropping is not suitable for the type of vegetative cover normally practiced at municpal landfills.

Table 4: Guide for Assigning Soil Loss Tolerance Values (T) to Solid Having Different Rooting Depths

Rooting Depth	Soil Loss Tolerance Values Annual Soil Loss (Tons/Acre)	
Inches	Renewable Soil a/	Renewable Soil b/
0 - 10	1	1
10 - 20	2	1
20 - 40	3	2
40 - 60	4	3
60	5	4

(This table appeared in SCS (6), p.4)

a/ Soil with favorable substrata that can be renewed by tillage, fertilizer, organic matter, and other management practices. This column does not represent MSWLF final covers under normal conditions.

b/ Soil with unfavorable substrata such as rock or soft rock that cannot be renewed by economical means. Most of the MSWLF covers with constructed clay cap and/or flexible membrane should use this performance criteria.

SHEET FLOW DESIGN

Prep by: KJW
Date: 3/25/2007

Chkd by: SAN
Date: 3/25/07

IESI Hardin County Landfill
Sheet Flow Velocity

Required: Determine the sheet flow velocity for the intermediate cover design and compare to the permissible non-erodible flow velocity.

Method:

1. Determine the flow using the Rational Method.
2. Calculate flow depth using Kinematic Wave Procedures.
3. Compare flow velocity to permissible non-erodibile velocity.

References:

1. Dodson and Associates, Inc., *Hands-On HEC-1,* June 1997.
2. Ponce, Victor M., *Engineering Hydrology Principles and Practices*, 1989.
3. Texas Department of Transportation, *Hydraulic Design Manual,* March 2004.
4. United States Soil Conservation Service, *Hydrology for Small Watersheds,* December 1989.

Solution: Use the typical case scenarios from the Erosion-USLE calculation to determine the expected sheet flow velocity for each slope.

Rainfall Intensity (I) is taken from the TxDOT's Hydraulic Design Manual for Hardin County, I = b/((tc + d)^e). The time of concentration (tc) will vary for each watershed, however, a minimum of 10 minutes was used for conservatism.

b = 80.0
d = 7.5
e = 0.720
Time of Concentration (tc) = 10 min
Rainfall Intensity (I) = 10.2 in/hr
Intermediate Cover Runoff Coefficient (C) = 0.70
Intermediate Cover Manning's Roughness (n) = 0.027 (60% vegetative cover)
25-Year Peak Flow = CIA

	External Top Slope (6%)	External Side Slope (25%)	External Side Slope (33.3%)	
Longest Run	1000	300	150	ft
Slope, s	0.06	0.25	0.33	ft/ft
Longest Run Area (1' wide)	0.023	0.007	0.003	acre
Q	0.164	0.049	0.025	cfs

Re-arranging the Manning's flow velocity formula to calculate depth of flow:

$$y = (Qn / 1.49\ S^{0.5})^{0.6}$$

Depth of flow (y)	0.0708	0.0224	0.0136	ft

Determine sheet flow velocity V=Q/A (where A is unit flow width of 1'):

Sheet flow velocity	**2.3130**	**2.1926**	**1.8114**	**ft/sec**

Summary: The permissible non-erodible velocity should be less than 5 ft/sec (clayey soil) or 4 ft/sec (sandy soil) on vegetated intermediate cover. Therefore, expected sheet flow velocity is acceptable on the external intermediate cover slopes with vegetation provided.

TEMPORARY DRAINAGE SWALE DESIGN

TEMPORARY DRAINAGE SWALE DESIGN

The temporary drainage swale design for intermediate cover areas is presented for the typical swale flowline of 0.5 percent. The procedures in the TxDOT Hydraulic Design Manual were used to determine peak flow, flow depth, and swale capacity. The temporary swales will be located on the intermediate cover to prevent erosion as follows.

Slope (%)	Maximum Sheet Flow Length (ft)	Maximum Drainage Area (acres)	Maximum Swale Length (ft)
6	1,000	18.2	791
25	200	5.7	825
33.3	80	4.7	1359

All temporary swales shall be designed to minimize erosion and provide a minimum 1 foot of freeboard. As noted in the calculations, the velocities in the swales are less than permissible non-erodible velocities. Example drainage swale calculations for a grassed intermediate cover are provided on pages 6A-A-16 through 6A-A-18.

TEMPORARY DIVERSION CHANNEL DESIGN

The temporary diversion channel design for diverting surface water runon around excavations is presented for three typical slopes of 0.5%, 1% and 2% and three typical drainage areas of 1, 5 and 10 acres. The procedures in the TxDOT Hydraulic Design Manual were used to determine peak flow, flow depth, flow velocity, and diversion channel capacity. Temporary diversion channels will be designed to minimize erosion and sedimentation.

Prep by: KJW
Date: 6/22/07

Chkd by: SAN
Date: 6/22/07

IESI Hardin County Landfill
Temporary Drainage Swale Analysis

Required: Determine the capacity of the external temporary drainage swale.

Method:
1. Determine peak flow and swale capacity using the Rational Method and the Manning's Equation.

References:
1. Dodson and Associates, Inc., *Hands-On HEC-1,* June 1997.
2. Ponce, Victor M., *Engineering Hydrology Principles and Practices,* 1989.
3. Texas Department of Transportation, *Hydraulic Design Manual,* March 2004.

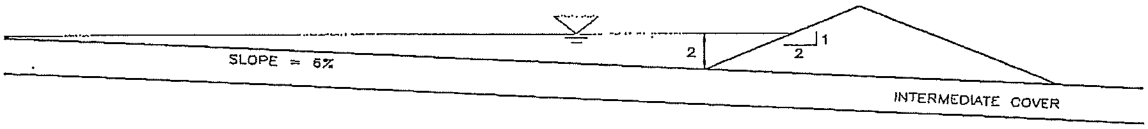
Solution: Drainage swales will be designed to function during the 25-year peak flow rate. The Rational Method was used to determine the peak runoff.

Design Parameters:

Rainfall Intensity (I) is taken from the TxDOT's Hydraulic Design Manual for Hardin County I = b/((tc + d)^e). The time of concentration (tc) will vary for each watershed, however, a minimum of 10 minutes was used for conservatism.

b = 80.0
d = 7.5
e = 0.720
Rainfall Intensity (I) = 10.2 in/hr
Post Developed Runoff Coefficient (C) = 0.7
Time of Concentration (tc) = 10.0 min

Swale Characteristics:



Max swale flow depth = 2.00 ft
Running swale slope = 0.5 %
Swale Manning's n = 0.03
Left slope = 16.67 :1
Right slope = 2 :1

Swale Evaluation (Mannings Equation):
Flow Area = 37.3 sf
Wetted Perimeter = 37.9 ft
Velocity = 3.47 ft/sec

Swale Capacity = 129.6 cfs

Use the Rational Method to determine the maximum drainage area:

A = Q/(CI)
Maximum Swale/Drainage Area = 18.2 acres

Determine the maximum swale length when the maximum sheet flow length is used:

Maximum Swale Length = (Maximum Area* 43,560)/Maximum Sheet Flow Length

Maximum Swale Length = 791 feet

Summary: The maximum sheet flow length will be 1,000 feet and the largest drainage area is less than 18.2 acres. The calculated velocity is less than the permissible non-erodible velocity.

Prep by: KJW
Date: 6/22/07

Chkd by: SAN
Date: 6/22/07

IESI Hardin County Landfill
Temporary Drainage Swale Analysis

Required: Determine the capacity of the temporary drainage swale.

Method:

1. Determine peak flow and swale capacity using the Rational Method and the Manning's Equation.

References:

1. Dodson and Associates, Inc., *Hands-On HEC-1*, June 1997.
2. Ponce, Victor M., *Engineering Hydrology Principles and Practices*, 1989.
3. Texas Department of Transportation, *Hydraulic Design Manual*, March 2004.

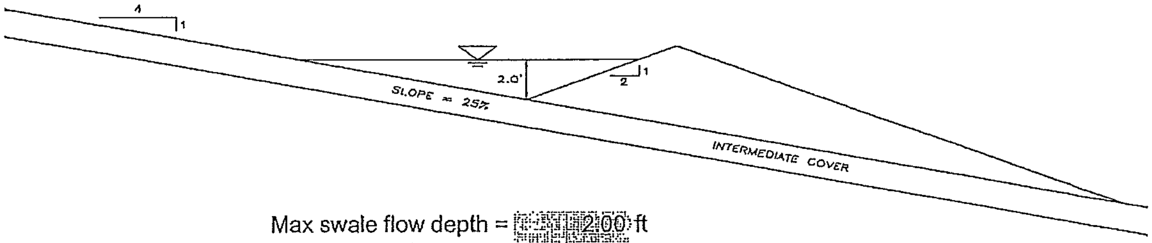
Solution: Drainage swales will be designed to pass the 25-year peak flow rate.
The Rational Method was used to determine the peak runoff.

Design Parameters:

Rainfall Intensity (I) is taken from the TxDOT's Hydraulic Design Manual for Hardin County, I = b/((tc + d)^e). The time of concentration (tc) will vary for each watershed, however, a minimum of 10 minutes was used for conservatism.

b = 80.0
d = 7.5
e = 0.720
Rainfall Intensity (I) = 10.2 in/hr
Post Developed Runoff Coefficient (C) = 0.7
Time of Concentration (tc) = 10.0 min

Swale Characteristics:



Max swale flow depth = 2.00 ft
Running swale slope = 0.5 %
Swale Manning's n = 0.03
Left slope = 4 :1
Right slope = 2 :1

Swale Evaluation (Manning Formula):
Flow Area = 12.0 s.f.
Wetted Perimeter = 12.7 ft
Velocity = 3.38 ft/sec

Swale Capacity = 40.5 cfs

Use the Rational Method to determine the maximum drainage area:

A = Q/(CI)
Maximum Swale/Drainage Area = 5.7 acres

Determine the maximum swale length when the maximum sheet flow length is used:

Maximum Swale Length = (Maximum Area* 43,560)/Maximum Sheet Flow Length
Maximum Swale Length = 825 feet

Summary: The maximum sheet flow length will be 300 feet and the largest drainage area is less than 5.7 acres. The calculated velocity is less than the permissible non-erodible velocity.

Prep by: KJW
Date: 6/22/07

Chkd by: SAN
Date: 6/22/07

IESI Hardin County Landfill
Temporary Drainage Swale Analysis

Required: Determine the capacity of the external temporary drainage swale.

Method:
1. Determine peak flow and swale capacity using the Rational Method and the Manning's Equation.

References:
1. Dodson and Associates, Inc., *Hands-On HEC-1,* June 1997.
2. Ponce, Victor M., *Engineering Hydrology Principles and Practices,* 1989.
3. Texas Department of Transportation, *Hydraulic Design Manual,* March 2004.

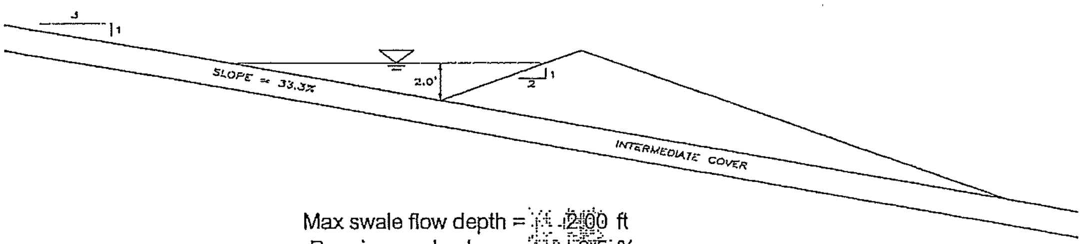
Solution: Drainage swales will be designed to pass the 25-year peak flow rate.
The Rational Method was used to determine the peak runoff.

Design Parameters:

Rainfall Intensity (I) is taken from the TxDOT's Hydraulic Design Manual for Hardin County, I = b/((tc + d)^e). The time of concentration (tc) will vary for each watershed, however, a minimum of 10 minutes was used for conservatism.

b = 80.0
d = 7.5
e = 0.720
Rainfall Intensity (I) = 10.2 in/hr
Post Developed Runoff Coefficient (C) = 0.7
Time of Concentration (tc) = 10.0 min

Swale Characteristics:



Max swale flow depth = 2.00 ft
Running swale slope = 0.5 %
Swale Manning's n = 0.03
Left slope = 3 :1
Right slope = 2 :1

Swale Evaluation (Manning Formula):

Flow Area = 10.0 s.f.
Wetted Perimeter = 10.8 ft
Velocity = 3.34 ft/sec

Swale Capacity = 33.4 cfs

Use the Rational Method to determine the maximum drainage area:

A = Q/(CI)
Maximum Swale/Drainage Area = 4.7 acres

Determine the maximum swale length when the maximum sheet flow length is used:

Maximum Swale Length = (Maximum Area* 43,560)/Maximum Sheet Flow Length
Maximum Swale Length = 1359 feet

Summary: The maximum sheet flow length will be 150 feet and the largest drainage area is less than 4.7 acres. The calculated velocity is less than the permissible non-erodible velocity.

TEMPORARY DIVERSION CHANNEL DESIGN

Prep by: KJW
Date: 3/25/2007

Chkd by: SAN
Date: 3/25/07

IESI Hardin County Landfill
Temporary Diversion Channel

Required: Determine the necessary dimensions of the temporary diversion channel for routing surface water around excavations.

Method:

1. Determine peak flow and swale capacity using the Rational Method and the Manning's Equation.

References:

1. Dodson and Associates, Inc., *Hands-On HEC-1,* June 1997.
2. Ponce, Victor M., *Engineering Hydrology Principles and Practices,* 1989.
3. Texas Department of Transportation, *Hydraulic Design Manual,* March 2004.

Solution: Diversion channels will be designed for the 25-year peak flow rate. The Rational Method was used to determine the peak runoff, Q = CIA.

Rainfall Intensity (I) is taken from the TxDOT's Hydraulic Design Manual for Hardin County, I = b/((tc + d)^e). The time of concentration (tc) will vary for each watershed; however, a minimum of 10 minutes was used for conservatism.

b = 80.0
d = 7.5
e = 0.720
Rainfall Intensity (I) = 10.2 in/hr.
Runoff Coefficient (C) = 0.7
Time of Concentration (tc) = 10 min
Runoff berm slope = varies %
Manning's n = 0.03
Right side slope = 3 :1

The Runoff Coefficient (C) was conservatively based on information in Reference 3, Table 5, for a high sloping area with minimal grassing.

Sample calculation for 1 acre drainage area:

Q = CIA
Q = (0.7) (10.2) (1)
Q = 7.1 cfs

Prep by:
Date: 3/25/2007

Chkd by: SAN
Date: 3/25/07

IESI Hardin County Landfill
Temporary Diversion Channel

Diversion channel drainage areas were based on the typical size that may occur during the development of the site. The diversion channels are intended to prevent surface water from entering the active or excavated areas. 1, 5 and 10 acre drainage areas were considered:

Diversion Channel Slope	Diversion Channel Area (Acres)	Flow (cfs)	Bottom Width (ft)	Side Slopes (H:V)	Manning's number (n)	Normal Depth (ft)	Flow Area (ft^2)	Wetted Perimeter (ft)	Velocity (ft/s)	Energy Head (ft)
0.5	1	7.1	0	3	0.03	1.04	3.26	6.59	2.19	1.12
0.5	5	35.7	0	3	0.03	1.91	10.89	12.05	3.27	2.07
0.5	10	71.3	0	3	0.03	2.47	18.32	15.63	3.89	2.71
1	1	7.1	0	3	0.03	0.91	2.51	5.79	2.84	1.04
1	5	35.7	0	3	0.03	1.67	8.40	10.58	4.25	1.95
1	10	71.3	0	3	0.03	2.17	14.12	13.72	5.05	2.57
2	1	7.1	0	3	0.03	0.80	1.94	5.08	3.68	1.01
2	5	35.7	0	3	0.03	1.47	6.48	9.29	5.51	1.94
2	10	71.3	0	3	0.03	1.91	10.89	12.05	6.55	2.57

1. The calculations shown in the table above are for normal depths from the 25-year, 24 hour storm event.
2. The required diversion channel depth will have 0.5 foot of freeboard.
3. Diversion channels shall be grassed. Erosion control features will be provided for velocities exceeding 5 fps.
4. During operation of the site, different configurations of diversion channels may be used to minimize erosion and erosive velocities. The landfill operator will determine the sizing of diversion channels if different lining material is used.
5. Shading represents sample calculation presented on pages 6A-A-22 and 6A-A-23.

Prep by: KJW
Date: 3/25/2007

Chkd by: SAN
Date: 3/25/07

IESI Hardin County Landfill
Sample Calculation: Temporary Diversion Channel

Example Calculation: Calculate the normal depth for the temporary diversion channel for a drainage area of 1 acre with a slope of 2% (see page 6A-A-21).

List of Symbols

Q_d = design flow rate for channel, cfs
A_f = flow area, sf
WP = wetted perimeter, ft
R = hydraulic radius, ft
S = channel slope, ft/ft
T = top width of flow, ft
b = bottom width of channel, ft
d = normal depth of flow for diversion channel, ft
z_r = z-ratio (ratio of run to rise for channel sideslope) for right sideslope of diversion berm
z_l = z-ratio (ratio of run to rise for channel sideslope) for left sideslope of diversion berm
n = Manning's roughness coefficient
g = gravitational acceleration = 32.2 ft/s^2

The program uses an iterative process to calculate the normal depth of the diversion berm to satisfy Manning's Equation.

$$Q = \frac{1.486}{n} A R^{2/3} S^{1/2}$$

Design Inputs:
Q_d = 7.1 cfs (from page 6A-A-21)
S = 0.02 ft/ft
b = 0 ft
z_r = 3 (H) : 1 (V)
z_l = 3 (H) : 1 (V)
n = 0.03

Step 1 - Based on the geometry of the swale cross-section, solve for R and A_f:

$$A_f = bd + 1/2d^2(z_r + z_l)$$

$$WP = b + d((z_l^2 + 1)^{0.5} + (z_r^2 + 1)^{0.5})$$

$$R = \frac{A_f}{WP}$$

assume: d = 0.80 ft

A_f = 1.94 sf

WP = 5.08 ft

R = 0.381 ft

solve for Q: Q = 7.1

If Q is not equal to Q_d, select a new d and repeat calculations.

IESI Hardin County Landfill
Sample Calculation: Temporary Diversion Channel

Step 2 - solve for velocity, T, Froude number, velocity head, and energy head:

$Q = VA \Rightarrow V = Q/A$

$V = 3.68$ ft/s

$T = b + d(z_l + z_r)$

$T = 4.82$ ft

$$F_r = \frac{V}{(gA/T)^{0.5}}$$

$F_r = 1.02$

$$\text{Velocity Head} = \frac{V^2}{2g}$$

Velocity Head = 0.21 ft

Energy Head = water elevation + velocity head

Energy Head = 1.01 ft

TEMPORARY DRAINAGE LETDOWN DESIGN

TEMPORARY DRAINAGE LETDOWN DESIGN

The temporary letdowns design is applicable for external sideslopes of the landfill with intermediate cover. Temporary letdown chutes will typically consist of berm channels lined with erosion control material (e.g., turf reinforcement matting, HDPE geomembrane). The flow capacity of the letdown structures was determined based on the Manning's Equation. The maximum flow calculated from the Manning's Equation is used to determine the maximum drainage area based on the Rational Method. The design calculations presented on pages 6A-A-26 through 6A-A-27 represent typical calculations for a HDPE geomembrane lined and turfmat lined channel on a 25% slope. If sustained erosion is observed, facility management will evaluate the use and construction of temporary letdowns.

Prep by: KJW
Date: 6/22/07

IESI Hardin County Landfill
Temporary Letdown Chute Flow Evaluation

Chkd by: SAN
Date: 6/22/07

Required: 1. Determine the capacity of the temporary letdown chute.

Method: Using the Rational Method and Manning's Equation determine chute capacity.

References:
1. Dodson and Associates Inc., *Hands on HEC-1*, June 1997.
2. Ponce, Victor M., *Engineering and Hydrology Principles and Practices*, 1989.
3. Texas Department of Transportation, *Hydraulic Design Manual,* March 2004.

Solution: 1. Chutes will be designed to function during the 25-year storm event.

Where: Q = Chute capacity (cfs)
n = Manning's Coefficient (unitless)
A = Cross sectional area (ft^2)
WP = Wetted Perimeter (ft)
R = Hydraulic Radius (ft)
S = Letdown slope (ft/ft)
d = Normal Depth (ft)
b = Bottom Width of Chute (ft)
z = Chute Side Slope (ft/ft)

$$A = bd + zd^2$$

$$WP = b + 2\,[(zd)^2 + d^2]^{0.5}$$

$$R = A / WP$$

$$Q = \frac{1.486(A)(R^{2/3})(S^{1/2})}{n}$$

HDPE Geomembrane Chute

Depth d (ft)	Bottom Width b (ft)	Letdown Slope S (ft/ft)	Chute Side Slope z (ft/ft)	Manning's Coefficient* n	Area A (sf)	Wetted Perimeter WP (ft)	Hydraulic Radius R (ft)	Velocity V (fps)	Flow Rate Q (cfs)
0.5	8	0.25	3	0.015	4.75	11.16	0.426	28.02	133
0.5	30	0.25	3	0.015	15.75	33.16	0.475	30.15	475

* Manning's coefficient selected for a temporary HDPE Geomembrane chute lining. Other lining material (e.g., turf reinforcement mat, gabions, riprap) will have different coefficients and will require design calculations.

Turf Reinforcement Mat Chute

Depth d (ft)	Bottom Width b (ft)	Letdown Slope S (ft/ft)	Chute Side Slope z (ft/ft)	Manning's Coefficient* n	Area A (sf)	Wetted Perimeter WP (ft)	Hydraulic Radius R (ft)	Velocity V (fps)	Flow Rate Q (cfs)
0.5	8	0.25	3	0.025	4.75	11.16	0.426	16.81	80
0.5	30	0.25	3	0.025	15.75	33.16	0.475	18.09	285

* Manning's coefficient selected for a temporary Turf Reinforcement Mat chute lining. Other lining material (e.g., geomembrane, gabions, riprap) will have different coefficients and will require design calculations.

Prep by: KJW
Date: 6/22/07

Chkd by: SAN
Date: 6/22/07

IESI Hardin County Landfill
Temporary Letdown Chute Flow Evaluation

2. The letdown chutes will be designed to pass the 25-year storm event. From the flow rate calculated above use the Rational Method to determine the maximum drainage area.

Design Parameters:

Rainfall Intensity (I) is taken from the TxDOT's Hydraulic Design Manual for Hardin County, I = b/((tc + d)^e). The time of concentration (tc) will vary for each watershed; however, a minimum of 10 minutes was used for conservatism.

b = 80.0
d = 7.5
e = 0.720
Rainfall Intensity (I) = 10.2 in/hr
Developed Runoff Coefficient (C) = 0.7
Time of Concentration (tc) = 10.0 min

Using the letdown flow rate calculated above and re-arranging the rational formula, the maximum drainage area is determined as follows:

Q = Flow Rate
A = Maximum Drainage Area
A = Q/(CI)
A = 133/(0.7*10.2)
A = 18.7 acres

HDPE Geomembrane Chute

Bottom Width	Flow Rate	Maximum Drainage Area
(ft)	(cfs)	(acres)
8	133	18.7
30	475	66.6

Turf Reinforcement Mat Chute

Bottom Width	Flow Rate	Maximum Drainage Area
(ft)	(cfs)	(acres)
8	80	11.2
30	285	40.0

DESIGN SUMMARY

The IESI Hardin County Landfill will implement the erosion and sediment control features on the intermediate cover as the landfill develops. The following items will be implemented as filling operations are ongoing:

- Intermediate cover will be established on all areas that have received waste but will remain inactive for periods greater than 180 days.
- Sufficient permanent and temporary erosion and sediment control features shall be constructed to redirect surface water and prevent erosion.
- Temporary erosion and sediment control features shall be constructed within 180 days of placement of intermediate cover.
- Final cover will be constructed as the site develops. Temporary erosion control features will be removed as permanent erosion controls are constructed.

APPENDIX IIIF-F

EROSION CONTROL PLAN FOR ALL PHASES OF LANDFILL OPERATION

STATE OF TEXAS
NEVZAT TURAN
84059
LICENSED PROFESSIONAL ENGINEER

12-5-2017

Includes pages IIIF-F-1 through IIIF-F-7

CONTENTS

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PROFESSIONAL ENGINEER

12-5-2017

EROSION CONTROL PLAN FOR ALL PHASES OF LANDFILL OPERATION

1.0 Introduction

The purpose of this appendix is to provide an Erosion Control Plan (ECP) to meet the requirements of Title 30 Texas Administrative Code (TAC) Chapter §330.305(d), which are listed below.

> *"The landfill design must provide effective erosional stability to top dome surfaces and external embankment side slopes during all phases of landfill operation, closure, and post-closure care in accordance with the following.*
>
> *(1) Estimated peak velocities for top surfaces and external embankment slopes should be less than then permissible non-erodible velocities under similar conditions.*
>
> *(2) The top surfaces and external embankment slopes of municipal solid waste landfill units must be designed to minimize erosion and soil loss through the use of appropriate side slopes, vegetation, and other structural and nonstructural controls, as necessary. Soil erosion loss (tons/acre) for the top surfaces and external embankment slopes may be calculated using the Soil Conservation Service of the United States Department of Agriculture's Universal Soil Loss Equation, in which case the potential soil loss should not exceed the permissible soil loss for comparable soil-slope lengths and soil-cover conditions."*

This ECP has also been developed to meet the requirements of the Texas Commission on Environmental Quality (TCEQ) guidance document titled, "Guidance for Addressing Erosional Stability During All Phases of Landfill Operation." As noted in the above guidance document, landfill cover phases are defined as daily cover, intermediate cover, and final cover. Top dome surfaces and external embankment side slopes are:

- Those above grade slopes that directly drain to the site perimeter stormwater management system (i.e., areas where the stormwater directly flows to a perimeter channel or detention pond designed in accordance with 30 TAC §§330.63(c), 330.303, and 330.305);
- Above grade slopes that have received intermediate or final cover; and
- Above grade slopes that have either reached their permitted elevation, or will subsequently remain inactive for longer than 180 days. For example, after an above grade slope has reached the permitted elevation and intermediate cover has been placed, the structural erosion control features (e.g., vegetation, drainage

swales, and letdown structures) will be in-place 180 days after intermediate cover has been placed.

Slopes which drain to ongoing waste placement areas, pre-excavated areas, areas that have received only daily cover, and areas under construction which have not received waste are not considered external side slopes.

The ECP for daily cover areas and top dome surfaces and external side slopes that drain directly to the site perimeter stormwater management system, have received intermediate cover, and either reached their permitted configuration or will remain inactive for longer than 180 days are addressed in the following sections. Erosion control measures for final cover areas are addressed in the TCEQ-approved Site Development Plan (SDP).

Inspection, maintenance, and recordkeeping requirements are included in the site's currently TCEQ-approved Site Operating Plan (SOP) and discussed in Section 2.4. The word "temporary" is used throughout the ECP to describe any erosion control feature that is not a permanent erosion control feature that is included in the approved SDP.

2.0 Erosion Control Plan for Top Dome Surfaces and External Side Slopes with Intermediate Cover

Erosion control for above grade top dome surfaces and external embankment side slopes that drain directly to the site perimeter stormwater management system, have received intermediate cover, and either reached their permitted configuration or will remain inactive for longer than 180 days will be managed using a system of nonstructural and structural erosion and sediment controls to meet rule requirements for the intermediate cover phase of landfill construction. Structural controls including vegetation, temporary swales, and letdown structures will be the primary and minimum control measures.

The structural controls will consist of a combination of vegetation, temporary add-on swales, and letdown structures. These structural controls will be configured in a manner that will result in a net soil loss of 50 tons/acre/year or less from the external slope area. As shown on Figure 1, stormwater runoff will be collected in swales and conveyed to drainage letdown structures down the 25 percent slopes to the perimeter drainage system. The primary goal will be to establish a minimum vegetation cover of 60 percent on all external top dome surfaces and external embankment slopes. This minimum vegetation cover percentage will result in a net soil loss of 50 tons/acres/year or less for each drainage swale and letdown combination specified on Figures 1 and 2 (refer to Section 2.1 for additional information).

Mulch, woodchips, compost, straw/hay or similar TCEQ approved materials may be used as a layer placed over the intermediate cover to protect the exposed soil surface from erosive forces and conserve soil moisture until vegetation can be established. The mulch, woodchips, compost, straw/hay or similar TCEQ approved materials will be used to stabilize recently graded or seeded areas. If needed, the mulch, woodchips, compost, straw/hay, or similar TCEQ approved materials will be spread evenly over a recently seeded area and tracked into the surface to protect the soil from erosion and moisture loss, and provide additional erosional stability to the intermediate cover surface during the establishment of vegetation. These materials are not required for the establishment of vegetation on the intermediate cover unless they are needed to provide additional erosional stability to the intermediate cover surface. These materials will vary in thickness but the mulch, woodchips, compost, straw/hay, or similar TCEQ approved materials will be placed so as not to inhibit the growth of vegetation. In the event that 60 percent vegetation coverage is not obtained within 180 days after intermediate cover is placed on a top dome or external side slope, mulch, woodchips, compost, straw/hay, or similar TCEQ approved materials may be used as a secondary measure to limit soil loss to 50 tons/acre/year or less. The mulch, woodchips, compost, straw/hay or similar TCEQ approved materials will be maintained to ensure equivalent 60 percent coverage until 60 percent vegetation coverage is achieved.

As an alternative to mulch, wood chips, compost, or straw/hay, a detention/sedimentation pond may be used as a secondary measure to limit soil loss to 50 tons/acre/year or less (refer to Section 2.2 for additional information) if the 60 percent vegetation goal is not obtained within 180 days after intermediate cover is placed on the top dome or external side slopes. In this case, the detention/sedimentation pond will remain in place until the specified percent vegetation goal is met (i.e., 60 percent vegetation on the external embankment slopes and top dome surfaces).

2.1 Drainage Swale and Letdown Structure Requirements

Figure 1 shows a typical layout for erosion control structures, including temporary add-on swales and drainage letdowns. Figure 2 provides a swale design summary, which includes spacing and vegetation cover requirements for the swales. Supporting calculations for the specifications listed on Figure 2 are provided in Appendix IIIF-F-1 – Temporary Add-on Swale Design. Appendix IIIF-F-1 also includes a demonstration to show that sheet flow velocities for the grass established surfaces for all swale spacings are less than 5 ft/s and sheet flow velocities for "nearly bare ground" is less than 3.5 ft/s (consistent with §330.305(d)(1)).

Letdown structures will be located and constructed in a manner that minimizes erosion loss. The letdowns are designed to convey runoff from the 25-year frequency storm event (refer to Appendix IIIF-F-2 – Temporary Letdown Design for more information). Figure 3 shows letdown details and the letdown design summary. As shown on Figure 3, the letdowns may consist of either a lined open channel structure or a pipe letdown. The type, size and number of letdowns will be determined based on the size of the drainage area using the design information specified on Figure 3. As noted on Figure 3, the use of pipe letdowns will be limited to 1 inlet per letdown.

Temporary open channel letdowns will be inspected for erosion/hollowing through and under the lining materials (e.g., gabions, grouted riprap, and turf reinforcement) and repaired as necessary to ensure the letdown is functioning as designed.

As stated previously, the primary goal is to obtain a vegetation coverage percentage of 60 percent on top dome surfaces and external embankments.

2.2 Sedimentation Pond Design

As noted on Figures 1 and 2, if vegetation cover for any surface is maintained at or above the percentages given for swale spacing distances, the estimated soil loss is less than 50 tons/acre/year. In the event that certain percent ground cover that limits the soil loss to 50 tons/acre/year is not achieved and soil loss is temporarily greater than 50 tons/acre/year, a sedimentation pond will be used along with other structural and non-structural BMPs approved as part of this plan to limit the discharge of eroded soil. Figure 4 provides a procedure for determining the required pond size. Supporting calculations for the procedure listed on Figure 4 are included in Appendix IIIF-F-3 – Sediment Control Pond Design. If a sediment control pond is used to limit the off-site discharge of eroded soil to 50 tons/acre/year or less from the external slope area, a demonstration noting how the pond was sized will be documented and maintained in the Site Operating Record.

The sedimentation pond option is a secondary erosion control option, similar to mulch, wood chips, compost, or straw/hay, and will only be used if the 60 percent vegetation specification is not met. If the sedimentation pond option is implemented, the swales and letdowns specified will remain in-place. The sedimentation pond option simply allows for the control of sediment while vegetation is being established.

For example, if intermediate cover is placed over a 20-acre external side slope area that is at the permitted elevation on December 31, then the operator will install swales and letdowns on the 20-acre slope consistent with the design and specifications listed in Section 2.1. The operator then has 180 days (which for this example would be June 29) to obtain 60 percent vegetation coverage on the 20-acre area. If in early June it becomes apparent that the percent vegetation will only be 40 percent on June 29, then the operator may install a sedimentation pond downstream of the 20-acre area, consistent with the requirements shown on Figure 4. Consistent with Section II.D of the TCEQ guidance document titled, "Guidance for Addressing Erosional Stability During All Phases of Landfill Operation," the sedimentation pond will remain in-place so that the net annual soil loss from the 20-acre area that could leave the facility boundary is less than 50 tons/acre/year until the 60 percent vegetation specification is met.

If a sedimentation pond is used as a source to maintain soil loss equal to or less than 50 tons/acre/year, the following procedure will be used to verify that an acceptable intermediate cover thickness is maintained.

- Intermediate cover areas will be inspected to detect erosion gullies and vegetation loss.
- After identifying the areas requiring additional soil, these areas will be replenished with additional soil and graded to provide uniform surfaces prior to reseeding.

- Any damaged concentrated flow drainage structures such as swales will be repaired to eliminate uncontrolled concentrated flow.

Sedimentation pond maintenance will occur as listed in Section 2.4. The primary goal of the erosion control effort is to obtain the specified vegetation coverage percentage on top dome surfaces and external embankments. The sedimentation pond will only be used until the specified vegetation coverage percentage is obtained. The sedimentation pond may only be used for a period of 12 months after the 180-day period has expired (e.g., 12 months after the June 29 date used in the above example). Once the vegetation percentage reaches the 60 percent specification, then the sedimentation pond will no longer be needed (but may remain in-place as an additional BMP). If the percent vegetation does not meet the 60 percent specification within the 12-month period, then additional erosion control measures will be implemented. These measures will include: (1) adjusting the swale spacing, (2) applying mulch, wood chips, compost, or straw/hay, or similar TCEQ approved materials, or (3) the submittal of a permit modification to revise this erosion control plan to provide additional erosion protection measures that will allow the site to meet the goals of this plan.

2.3 Other Erosion Control BMPs

Other best management practices (BMPs) that can be used in conjunction with the above erosion control measures:

- Check Dams – These structures may be used in channels to slow down flow velocities and improve sediment capture.
- Silt Fences – These structures may be used in capturing sediment transported by sheet flow and for diversion of flow for controlling sediment discharge.
- Compost Filter Berms – These structures may be used in capturing sediment transported by sheet flow and for diversion of flow for controlling sediment discharge. Wood mulch may also be used for filter berms.

These erosion control measures can be used on slopes, in perimeter channels, and in the sediment control pond. Refer to Figure 6 for details of typical BMPs.

Nonstructural controls that may be used at the site to minimize erosion loss include: plans and designs to minimize disruption of the natural features, drainage, topography, vegetation cover features; phased development to minimize the area of bare soil exposed at any given time; plans to disturb only the smallest area necessary to perform current activities; scheduling of construction activities during the time of year with the least erosion potential; and specific plans for the stabilization of exposed surfaces in a timely manner.

2.4 Schedule and Recordkeeping Requirements

After an external side slope or top dome surface reaches the final permitted grade or will remain inactive for longer than 180 days, the erosion control features (e.g. vegetation, drainage swales, letdown structures) will be in place within 180 days from when intermediate cover is placed. During this 180 day period, the structural erosion control structures will be constructed and vegetation established.

At the end of this 180 day period, the cover log will be updated to document the external side slope and top dome surface area, the structural controls that were installed, and a demonstration showing how the structural controls meet the 50 tons/acre/year or less soil loss requirement (e.g. percent vegetation coverage, swale spacing, letdowns installed). If 60 percent vegetation coverage is not achieved within the 180-day period, secondary erosion control measures such as mulch, wood chips, compost, or similar TCEQ approved materials may be used to limit the soil loss to the 50 tons/acre/year or less. In addition, a detention/sedimentation pond may also be used until the 60 percent vegetation coverage is achieved. Any secondary measure used will be documented in the Site Operating Record at the end of the 180-day period to document compliance with this plan. In addition, the date 60 percent vegetation cover is achieved and the date that the secondary measure is no longer needed will also be documented in the Site Operating Record. Inspection requirements and schedules are listed in the SOP for all drainage features, including intermediate cover areas.

As stated in Section 8.19.7 of the SOP, intermediate cover areas are inspected weekly and within 72 hours of a rainfall event of 0.5 inches or more for proper placement, thickness, erosion, and compaction (the intermediate cover inspection will also include inspection of any temporary swales and/or letdowns constructed on intermediate cover).

During the inspection of non-structural controls if erosion gullies or washed-out areas of the intermediate cover are detected, the intermediate cover will be repaired in accordance with section 8.19.7 of the SOP. An eroded area is considered to be deep enough to jeopardize the intermediate cover if it exceeds 4 inches in depth as measured perpendicular with the horizontal slope face or surface. Should additional time be required for repairs due to weather related delays, the landfill will request from the TCEQ region office approval of an alternate schedule. If significant soil loss is identified on a given intermediate cover area, impacted areas will be replenished with additional soil. Prior to application of temporary erosion controls and seeding, the area will be graded to eliminate preferential path ways or any other uneven surface due to settlement to prevent concentrated flow over the intermediate cover areas. Soil for replenishment of intermediate cover areas may be borrowed from sedimentation pond or any other soil source. If sediment collected in wet retention pond(s) is used for intermediate cover layer replenishment, it may be stockpiled outside the ponds to dry out prior to being used for intermediate cover layer replenishment.

If a sedimentation pond is used to achieve the specified vegetation cover percentage, inspection of the pond to ensure it is functioning as designed (e.g., excess sediment removed, outlet structures intact) will be performed on a weekly basis and after a rainfall event of 0.5 inches or more.

2.5 Construction Activities on Top Dome Surfaces and External Side Slopes with Intermediate Cover

Occasionally, top dome surfaces and external side slopes that have been stabilized through the use of swales, letdown structures, and compliance with the minimum 60 percent vegetation cover specification will be disturbed due to various construction activities such as the installation or repair of a landfill gas system, regrading of an area due to ponded water caused by uneven waste settlement, the repair of erosion rills, or damage due to an extreme storm event or natural disaster. Each of these events will be documented in the Site Operating Record. Recorded information will include the date of construction, approximate area disturbed, and the date re-seeding of the disturbed area occurred. Structural erosion control measures will be promptly restored after completion of the activities listed in this section.

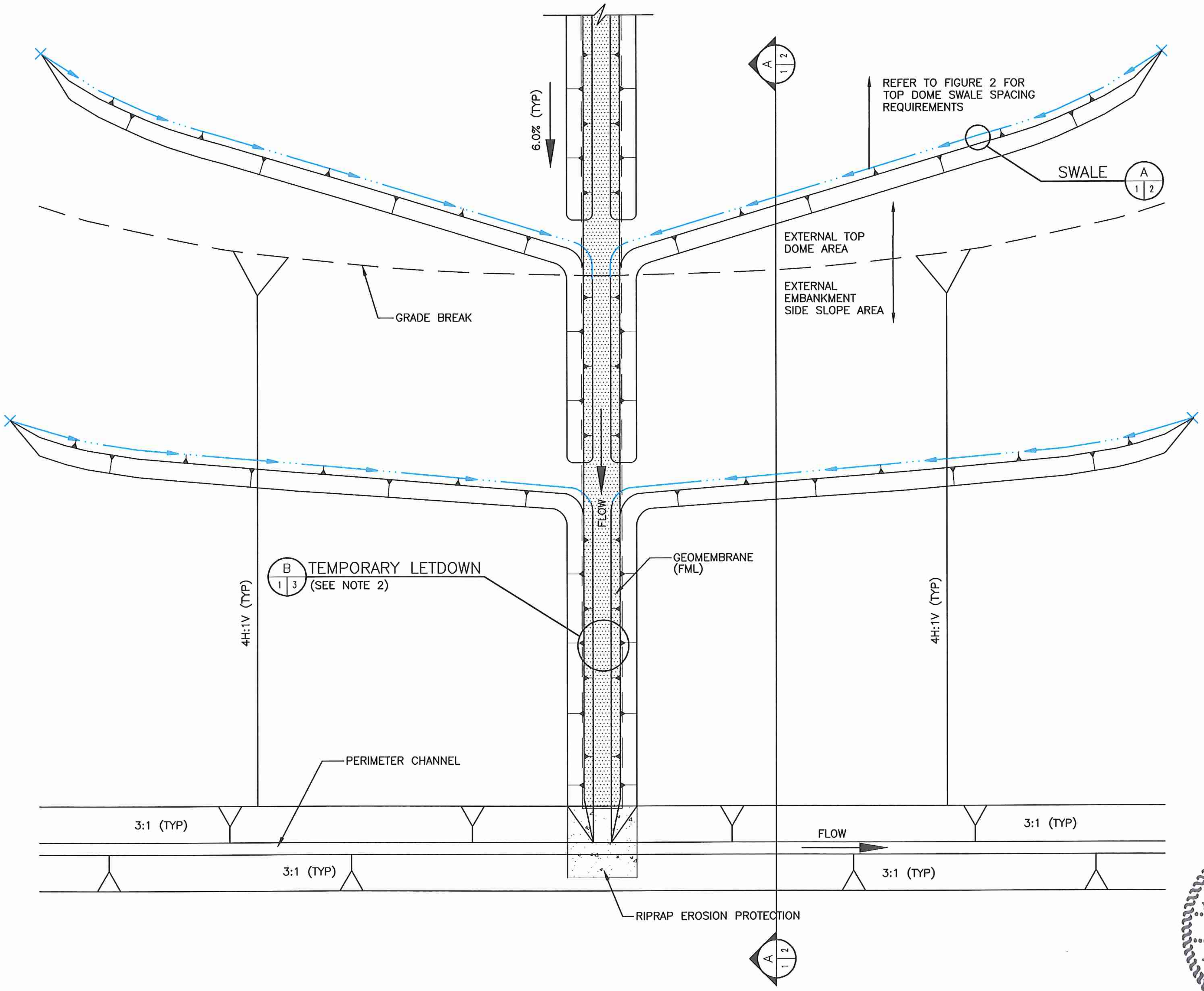
3.0 Erosion Control Plan for Daily Cover Areas and Intermediate Cover Areas for Non-External Side Slopes

BMPs will be employed to control erosion. BMPs may include the use of temporary rock riprap, silt fences, straw bales, check dams, interceptor swales and berms, temporary and permanent seeding and sodding, surface roughening, matting and mulching, sediment traps, and surface wetting for dust control.

Examples of erosion and sedimentation control features that will be used during the phased development of the site are shown in Figure 1 through 6 of this appendix. The following provides general guidelines of how the erosion control features will minimize sediment discharge from the site.

- As noted in the SOP, vegetation will be established on above-grade intermediate cover areas that remain inactive. The temporary vegetation cover will minimize erosion potential.
- Typically, uncontaminated stormwater runoff from the site will be channeled through the perimeter channel system to detention ponds before being discharged from the site. Sediment that collects in the channels and detention ponds will be removed consistent with the stormwater system maintenance plan presented in Figure 6.
- Erosion will be controlled by vegetation in drainage structures with flow velocities less than or equal to 5 fps. For drainage structures with flow velocities greater than 5 fps, rock riprap, gabions, or other alternate materials will be used for surface reinforcement.

Typical erosion control features are shown on Figure 6. Inspection items and schedules are listed in the SOP for all drainage features, daily cover, and intermediate cover areas.



LEGEND

→· · ·	DRAINAGE SWALE FLOWLINE
═	LETDOWN
▨	GEOMEMBRANE LINING WITHIN LETDOWN (SEE NOTE 2)
⁘	RIPRAP EROSION PROTECTION

NOTES:

1. THE ACCEPTABLE SOIL LOSS IS LESS THAN OR EQUAL TO 50 TONS/ACRE/YEAR. THE SOIL LOSS FOR TOP DOME SURFACES AND EXISTING EXTERNAL EMBANKMENT SIDE SLOPES WILL VARY DEPENDING ON SWALE SPACING AND PERCENT VEGETATIVE COVER (REFER TO FIGURE 2 FOR SOIL LOSS ESTIMATES).
2. TEMPORARY LETDOWN IS SHOWN AS AN OPEN CHANNEL WITH A GEOMEMBRANE LINER. AS NOTED ON FIGURE 3, OTHER CHANNEL LININGS MAY BE USED (e.g., GABIONS, GROUTED CONCRETE RIPRAP, AND TURF REINFORCEMENT MAT). IN ADDITION, PIPE LETDOWNS MAY ALSO BE USED. HOWEVER, IF PIPE LETDOWNS ARE USED THEY WILL BE LIMITED TO 1 INLET AS SHOWN ON FIGURE 3.

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12-5-2017

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DATE: 03/2017
FILE: 0120-758-11
CAD: FIG 1-EROS CONTROL STRUCT.DWG

DRAWN BY: SRF
DESIGN BY: AE
REVIEWED BY: NT

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

PREPARED FOR
BFI WASTE SYSTEMS OF NORTH AMERICA, LLC

REVISIONS

NO.	DATE	DESCRIPTION
1	11/2017	OWNERSHIP CHANGE

EROSION CONTROL PLAN TYPICAL EROSION CONTROL STRUCTURE LAYOUT

HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS

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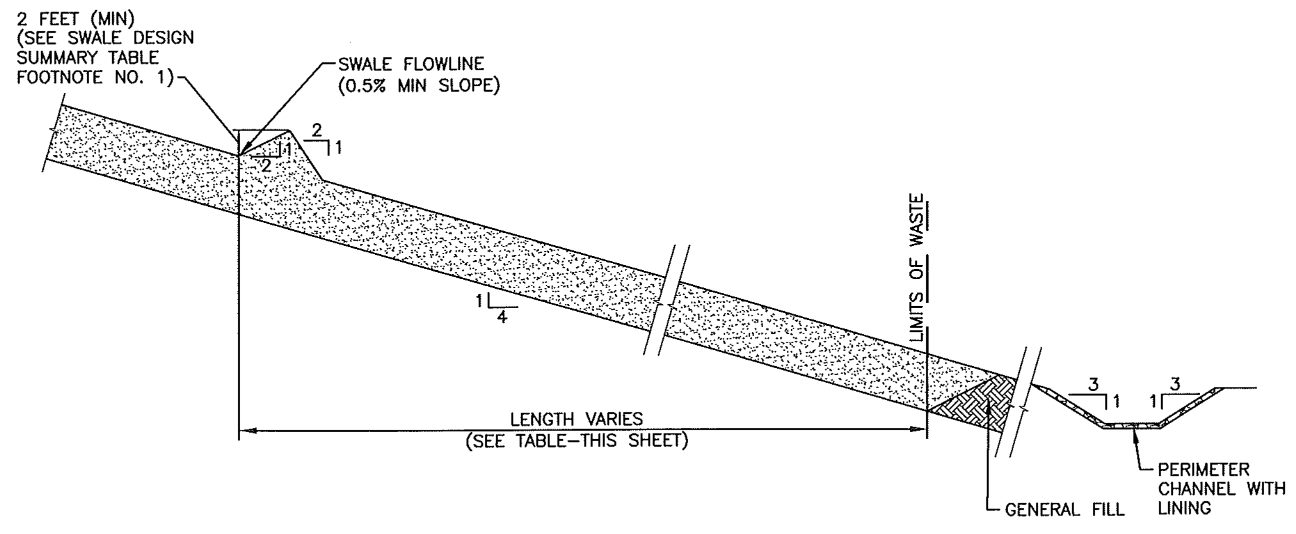
FIGURE 1



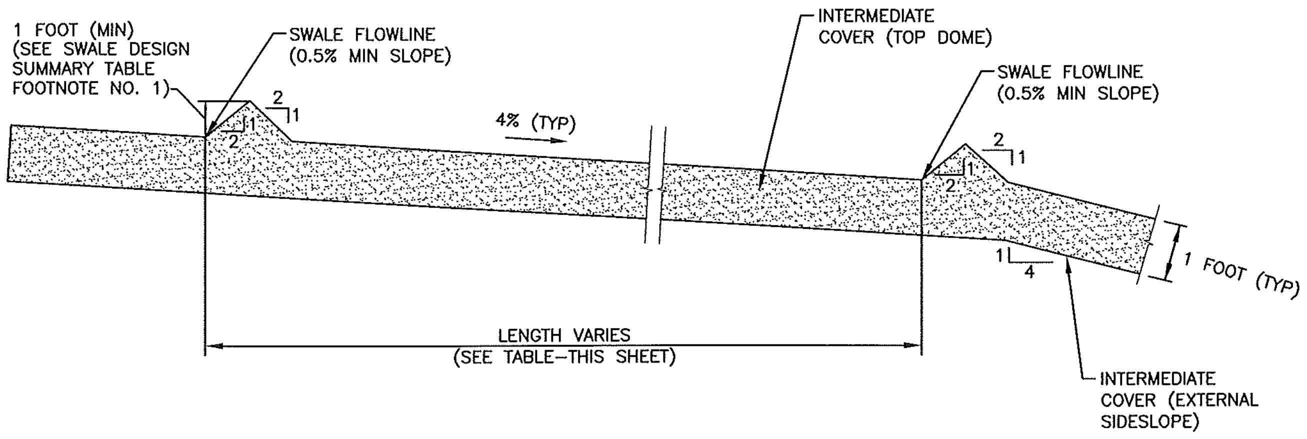
A 1|2 SIDE SLOPE DRAINAGE SWALE

SWALE DESIGN SUMMARY[1]

SIDE SLOPE (4H:1V)				TOP SLOPE (4%)			
VEGETATIVE COVER PERCENTAGE	DISTANCE BETWEEN SWALES (FT)	ESTIMATED SOIL LOSS (TONS/ACRE/YEAR)	ADDITIONAL SEDIMENT CAPTURE REQUIRED[2]	VEGETATIVE COVER PERCENTAGE	DISTANCE BETWEEN SWALES (FT)	ESTIMATED SOIL LOSS (TONS/ACRE/YEAR)	ADDITIONAL SEDIMENT CAPTURE REQUIRED[2]
60	300	35.1	NO	60	500	2.6	NO
70	300	14.2	NO	70	500	1.1	NO
80	300	10.9	NO	80	500	0.8	NO
90	300	5.3	NO	90	500	0.4	NO
70	400	16.3	NO	60	600	2.8	NO
80	400	12.5	NO	70	600	1.1	NO
90	400	6.1	NO	80	600	0.9	NO
70	500	17.1	NO	90	600	0.4	NO
80	500	13.0	NO	60	700	3.2	NO
90	500	6.4	NO	70	700	1.3	NO
80	600	13.6	NO	80	700	1.0	NO
90	600	6.7	NO	90	700	0.5	NO

[1] REFER TO APPENDIX IIIF–F–1 FOR SUPPORTING CALCULATIONS.

[2] IF SITE SPECIFIC CONDITIONS YIELD A MAXIMUM HORIZONTAL DISTANCE BETWEEN THE TOE OF THE SLOPE AND GRADE BREAK OF LESS THAN 300 FEET FOR SIDE SLOPES AND A DISTANCE OF 500 FEET FROM THE GRADE BREAK TO THE PEAK OF THE TOP SLOPES, ESTABLISHMENT OF 60% VEGETATION WILL BE SUFFICIENT MEANS OF EROSION CONTROL WITHOUT THE ADDITION OF TEMPORARY SWALES AND LETDOWNS GIVEN THAT THE TOTAL SOIL LOSS FOR THE SIDE SLOPE AND TOP SLOPE IS LESS THAN 50 TONS/ACRE/YEAR.



A 1|2 TOP DOME SURFACE DRAINAGE SWALE

SWALE DRAINAGE AREA SUMMARY

CONDITION (SWALE HEIGHT)	MAXIMUM DRAINAGE AREA (ACRES)	MINIMUM SWALE SPACING[1] (FEET)	MAXIMUM SWALE LENGTH[2] (FEET)
TOP SLOPE (2 FT SWALE)	17.5	500	1,525
TOP SLOPE (1.5 FT SWALE)	7.5	500	653
TOP SLOPE (1 FT SWALE)	2.0	500	174
SIDE SLOPE (2 FT SWALE)	5.0	300	726

[1] MINIMUM SWALE SPACING IS OBTAINED FROM THE CALCULATIONS PROVIDED ON PAGE IIIF–F–1–9.

[2] MAXIMUM SWALE LENGTH CALCULATED USING THE FOLLOWING EQUATION:

MAXIMUM DRAINAGE AREA x (43,560 SF/ACRE)/MINIMUM SWALE SPACING

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EROSION CONTROL PLAN
SWALE DESIGN SUMMARY

HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS

DATE: 03/2017
FILE: 0120–758–11
CAD: FIG 2– SWALE DESIGN SUMMARY.DWG

DRAWN BY: SRF
DESIGN BY: AE
REVIEWED BY: NT

REVISIONS

NO.	DATE	DESCRIPTION
1	08/2017	FIRST NOD RESPONSE
2	11/2017	OWNERSHIP CHANGE

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FIGURE 2

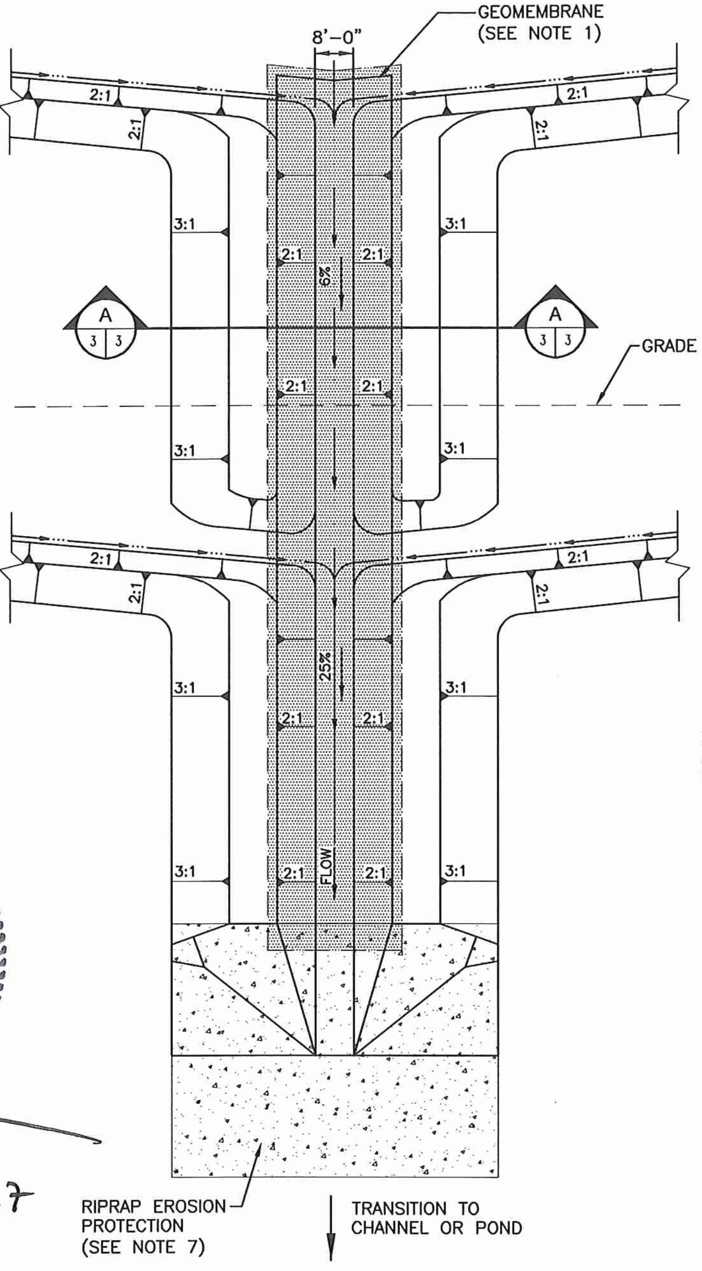
O:\0120\758\2214B EXPANSION\IIIF\FIGURES\FIG 2–SWALE DESIGN.dwg, 11/15/2017 9:52:22 AM, rsellers, 1:2

OPEN CHANNEL LETDOWN DESIGN SUMMARY

DESIGN IS APPLICABLE FOR A DRAINAGE AREA UP TO 20 ACRES.

25% SLOPE
MAXIMUM FLOW DEPTH = 0.45 FT.
BOTTOM WIDTH = 8 FT.

5% SLOPE
MAXIMUM FLOW DEPTH = 0.72 FT.
BOTTOM WIDTH = 8 FT.



TEMPORARY OPEN CHANNEL LETDOWN (B 1|3)

0 20 40
SCALE IN FEET

STATE OF TEXAS
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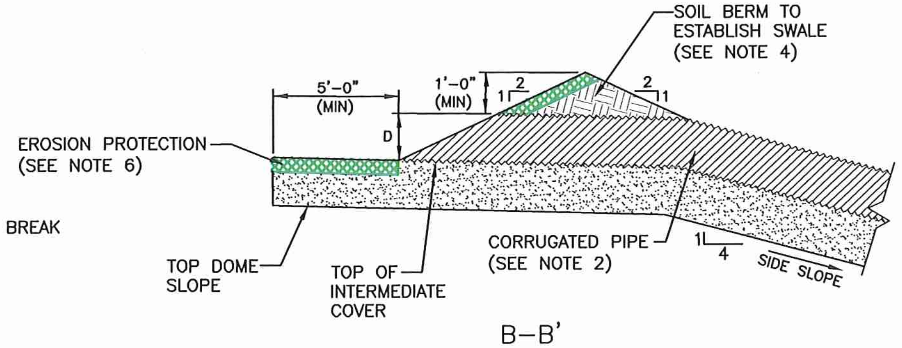
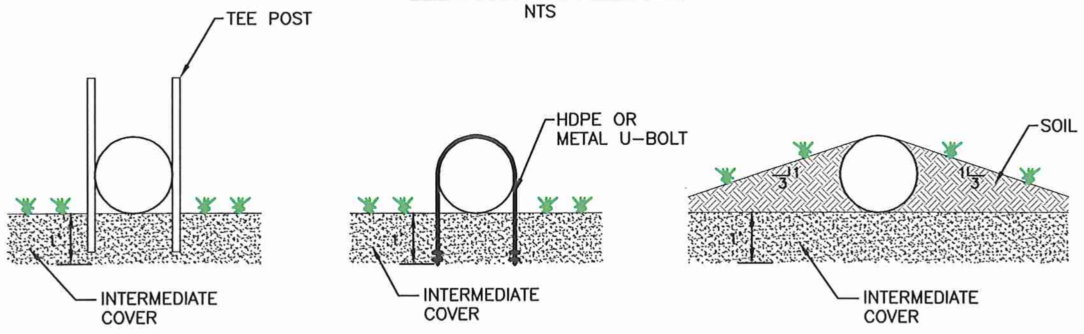
12-5-2017



B–B'

PIPE INLET PROFILE

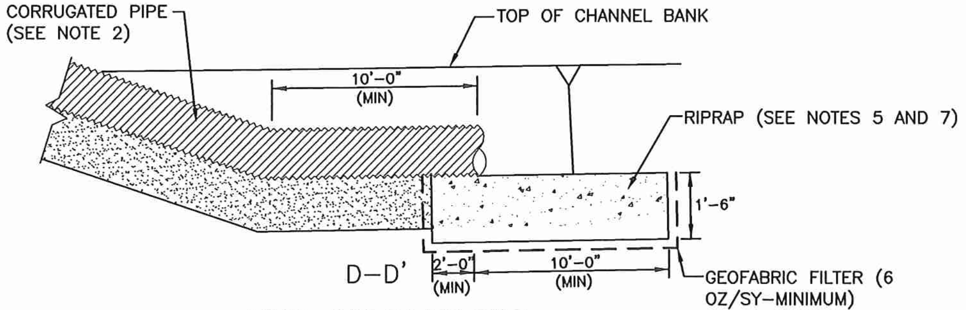
NTS



C–C'

ANCHORING DETAILS

NTS



D–D'

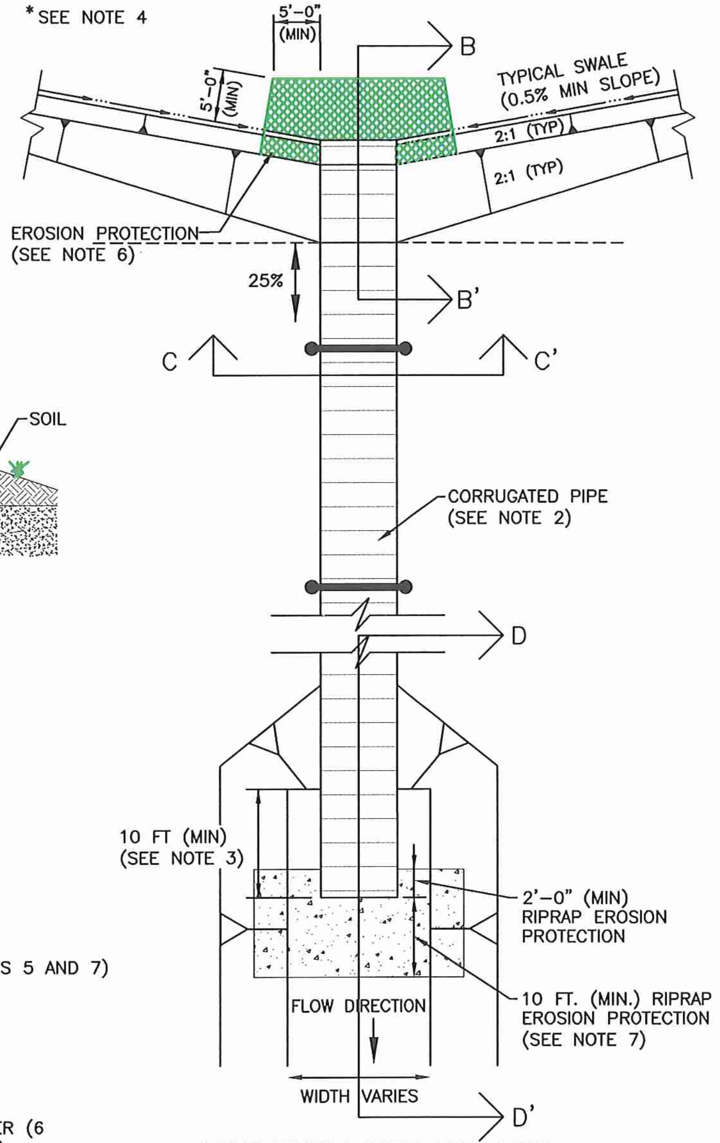
PIPE OUTLET PROFILE
(SEE NOTE 3)

NTS

PIPE LETDOWN DESIGN SUMMARY*
(USE OF PIPE LETDOWN IS LIMITED TO 1-INLET)

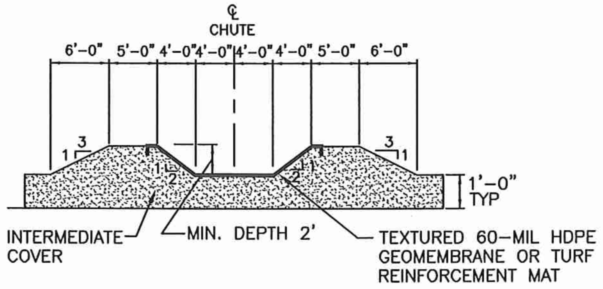
DRAINAGE AREA (ACRE)	DESIGN FLOW RATE (CFS)	REQUIRED PIPE DIAMETER (FT)
1.6	13	1.5
2.7	22	2.0

* SEE NOTE 4



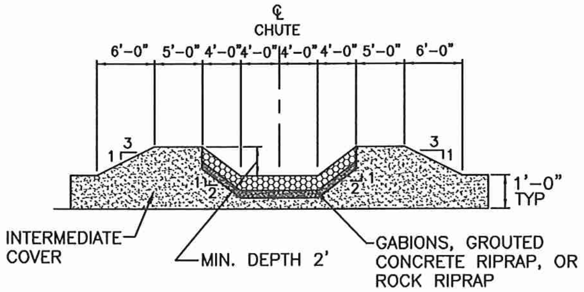
TEMPORARY PIPE LETDOWN

NTS



(A1 3|3) TEMPORARY GEOMEMBRANE OR TURF REINFORCEMENT LETDOWN OPTION

0 10 20
SCALE IN FEET



(A2 3|3) TEMPORARY GABION LETDOWN OPTION

0 10 20
SCALE IN FEET

NOTES:

1. 60 MIL HDPE GEOMEMBRANE TEXTURED BOTH SIDES WILL BE USED FOR GEOMEMBRANE LETDOWN LINING. AS AN ALTERNATIVE, TEMPORARY LETDOWN CAN BE LINED WITH GABIONS, GROUTED CONCRETE RIPRAP, TURF REINFORCEMENT MAT, OR ROCK RIPRAP.
2. PIPE DRAINAGE LETDOWN WILL BE ANCHORED BY USING SOIL BERM AT THE INLET LOCATED WITHIN THE SWALE. ADDITIONAL ANCHORING ON THE SIDE SLOPE MAY BE PROVIDED USING SOIL, HDPE, METAL U-BOLTS, T-POSTS OR EQUIVALENT MATERIALS.
3. PIPE WILL BE EXTENDED INTO THE CHANNEL TO MINIMIZE EROSION.
4. PIPE LETDOWNS WILL BE LIMITED TO 1 INLET PER LETDOWN. SOIL BERMS AROUND THE PIPE INLET WILL BE EXTENDED A MINIMUM 1-FOOT ABOVE THE LETDOWN PIPE INLET. REFER TO PAGE IIIF-F-2-9 FOR HYDRAULIC ANALYSIS.
5. RIPRAP APRON DESIGN IS PROVIDED ON PAGES IIIF-F-2-12 AND 13. D_{50} FOR RIPRAP IS 5-INCHES MINIMUM.
6. RIPRAP, GROUTED RIPRAP, GABIONS, GEOMEMBRANE, EXISTING VEGETATION, OR TURF REINFORCEMENT MAY BE USED FOR INLET EROSION PROTECTION.
7. OTHER EROSION PROTECTION (e.g., RIPRAP, GROUT, GROUTED RIRRAP, GABIONS, OR TURF REINFORCEMENT) MAY BE USED AT TEMPORARY LETDOWN OUTFALLS.

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EROSION CONTROL PLAN
LETDOWN DESIGN SUMMARY

DATE: 03/2017
FILE: 0120-758-11
CAD: FIG 3-LETDOWN DESIGN.DWG

DRAWN BY: SRF
DESIGN BY: AE
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NO.	DATE	DESCRIPTION
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HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS

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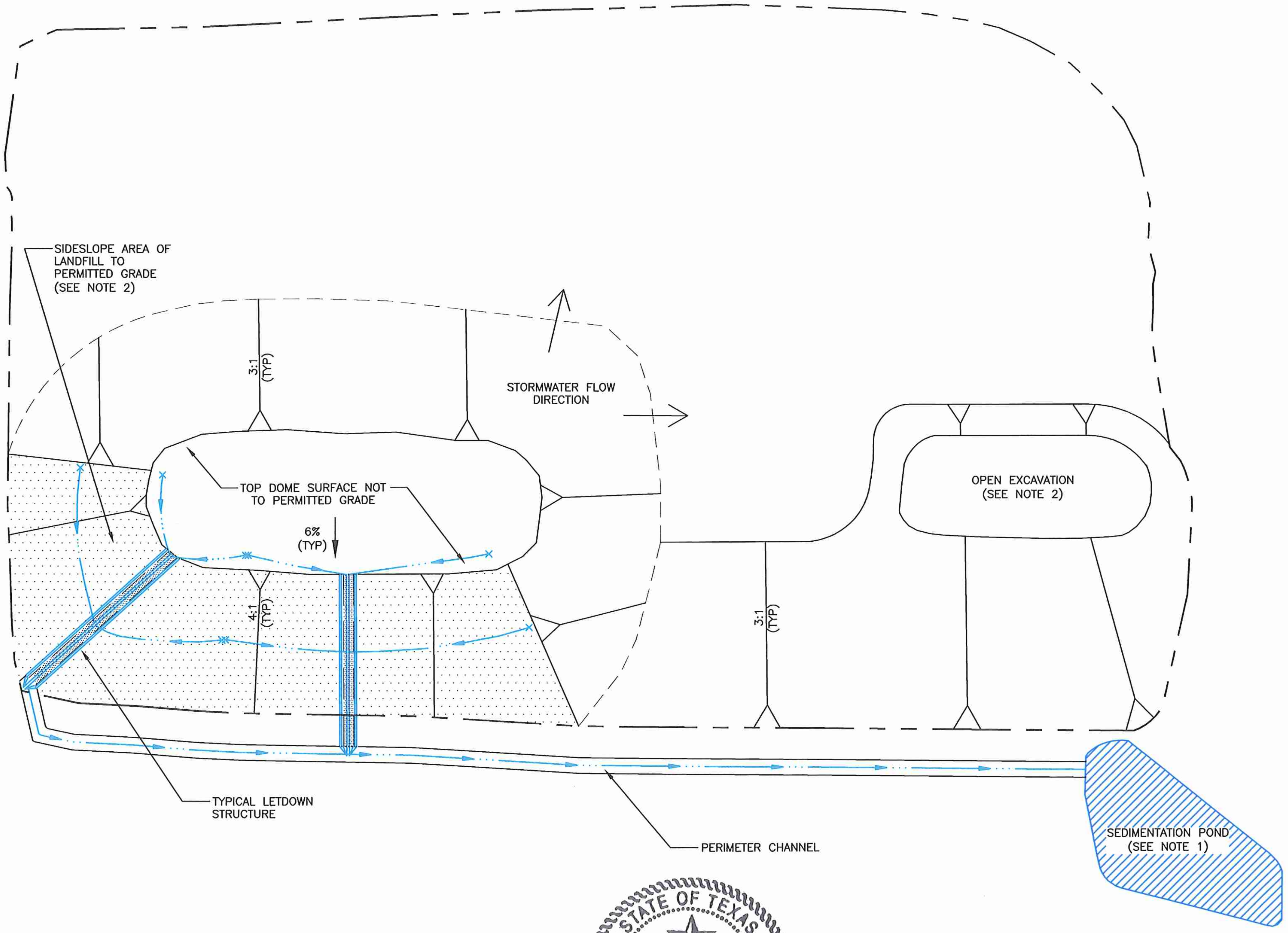
FIGURE 3

O:\0120\758\2214B EXPANSION\IIIF\FIGURES\FIG 3-LETDOWN DETAILS.dwg, 11/15/2017 9:52:54 AM, rsellers, 1:2



EXAMPLE CALCULATION

REQUIRED POND SIZE = EXTERNAL EMBANKMENT AREA (ACRES) X POND AREA REQUIRED/UNIT DRAINAGE AREA FACTOR

EXTERNAL EMBANKMENT AREA DRAINING TO POND = 50 ACRES

ADDITIONAL UPLAND AREA DRAINING TO POND = 0 ACRES (SEE NOTE 1)

REQUIRED SEDIMENT REMOVAL FROM EXTERNAL SIDE SLOPE AREA = 80 TONS/ACRE/YEAR TO 50 TONS/ACRE/YEAR

POND AREA REQUIRED/UNIT DRAINAGE AREA FACTOR (FROM TABLE BELOW) = 0.085

REQUIRED POND SIZE = 50 ACRES X 0.085 = 4.25 ACRES

SIZE OF POND REQUIRED[1]

REQUIRED SEDIMENT REMOVAL (TONS/ACRE/YEAR)	POND AREA REQUIRED/ UNIT DRAINAGE AREA FACTOR	EFFICIENCY OF POND (DYNAMIC AND QUIESCENT)
60 TO 50	0.035	19.5%
70 TO 50	0.060	30.5%
80 TO 50	0.085	39.3%
90 TO 50	0.110	46.2%
100 TO 50	0.130	51.4%
200 TO 50	0.300	75.5%

[1] REFER TO APPENDIX IIIF–F–3 FOR MORE INFORMATION. THE POND DESIGN AND DEMONSTRATION ARE PROVIDED TO ENSURE THAT THE SEDIMENT DISCHARGE FROM THE SITE WILL BE PREVENTED DURING INITIAL ESTABLISHMENT OF VEGETATION OVER THE SIDESLOPES AND TOP DOME SURFACES.

NOTES:

1. EXAMPLE POND CONFIGURATION IS SHOWN. A DEMONSTRATION WILL BE INCLUDED IN THE SITE OPERATING RECORD TO SHOW THAT THE POND HAS THE CAPABILITY TO CAPTURE SEDIMENT SUCH THAT DISCHARGE IS LESS THAN OR EQUAL TO 50 TONS/ACRE/YEAR FROM THE EXTERNAL SIDE SLOPE AND TOP DOME AREA. THE DEMONSTRATION WILL ACCOUNT FOR THE ADDITIONAL SEDIMENT CREATED BY THE UPLAND AREA THAT FLOWS TO THE POND. FOR DEMONSTRATION PURPOSES, THE POND DEPTH WILL BE AN AVERAGE OF 4 FEET. OVERALL SEDIMENT DISCHARGE FROM THE SITE MUST COMPLY WITH THE CURRENT TPDES PERMIT FOR THE SITE.
2. EXCAVATED FUTURE CELL AREAS OR SOIL BORROW AREAS CAN ALSO BE USED AS SEDIMENTATION PONDS. IF THESE AREAS ARE USED FOR PONDS, A DEMONSTRATION NOTING THAT THE EXCAVATED FUTURE CELL AREA OR SOIL BORROW AREA HAS MORE CAPACITY THAN THE VOLUME PRODUCED BY THE 25–YEAR, 24–HOUR STORM WILL BE DOCUMENTED AND MAINTAINED IN THE SITE OPERATING RECORD.

LEGEND

⬚	EXTERNAL SIDE SLOPE TO PERMITTED GRADE
═	LETDOWN
×—▸···—	DRAINAGE SWALE
→	STORMWATER FLOW DIRECTION

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CAD: FIG 4–DETENTION POND PLAN.DWG

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DESIGN BY: AE
REVIEWED BY: NT

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EROSION CONTROL PLAN
SEDIMENT CONTROL POND PLAN

HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS

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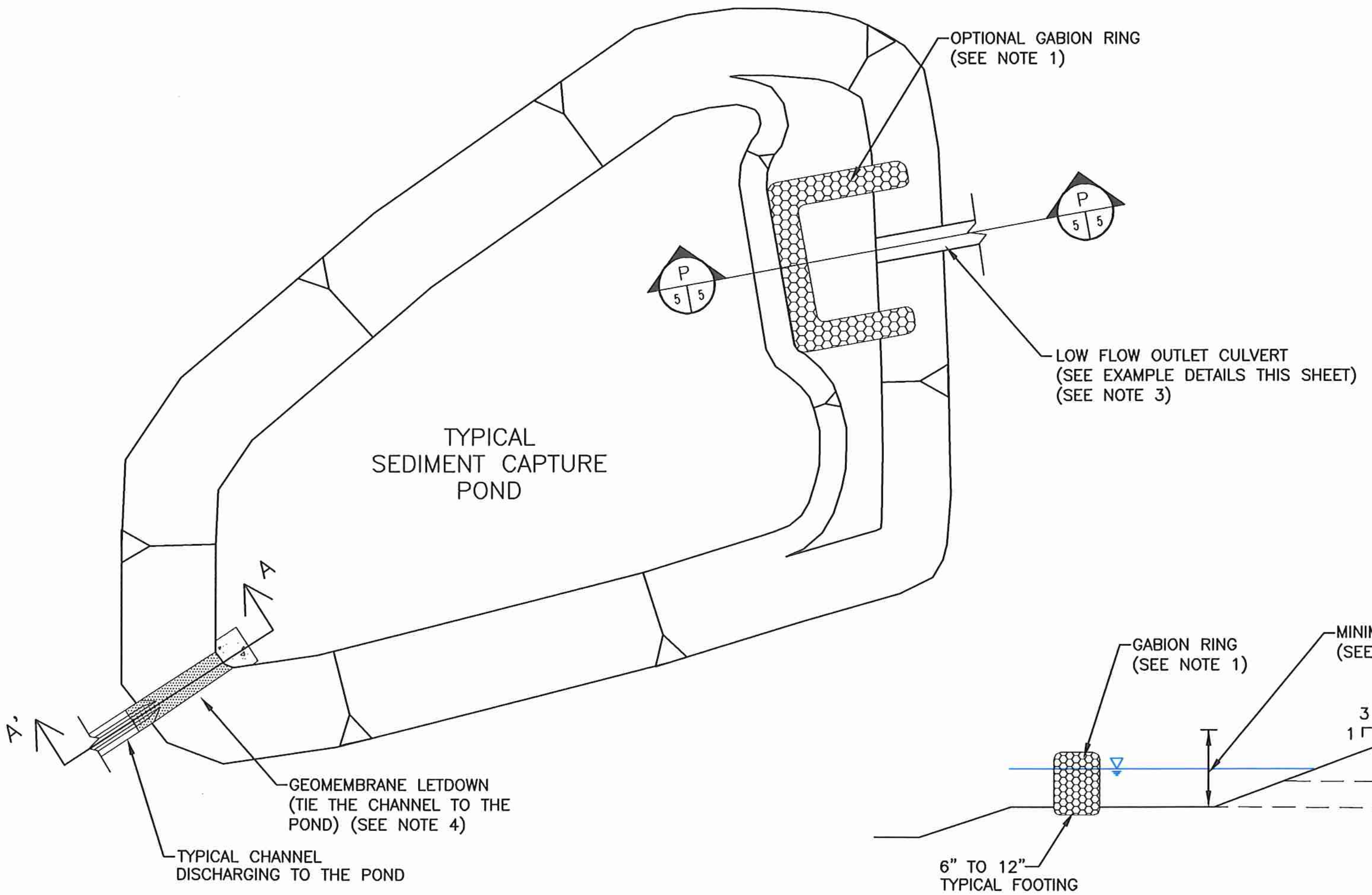
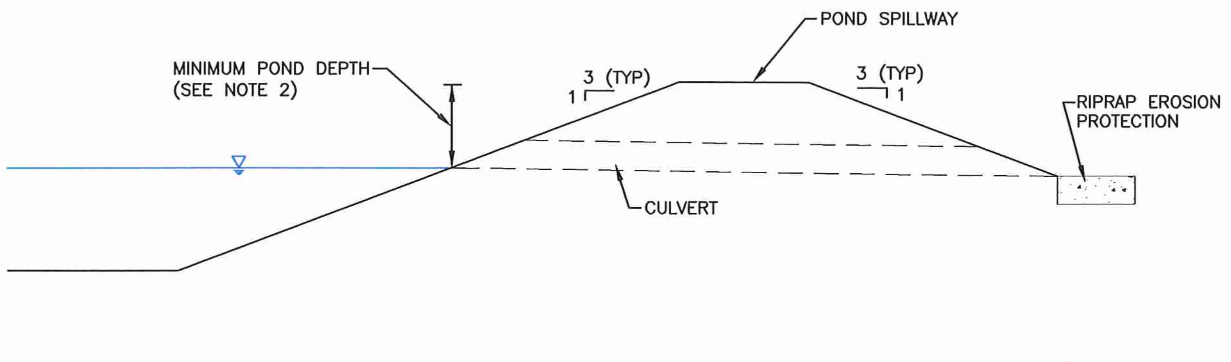
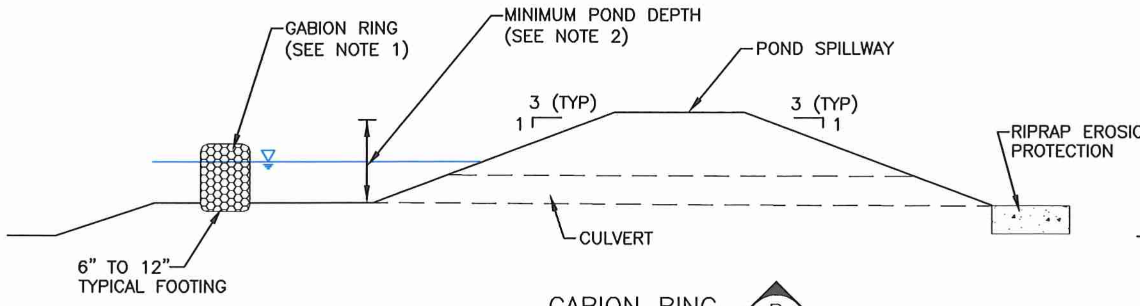
FIGURE 4

O:\0120\758\2214B EXPANSION\IIIF\FIGURES\FIG 4-SED CAPTURE DET POND DESIGN.dwg, 11/15/2017 9:53:28 AM, rsellers, 1:2

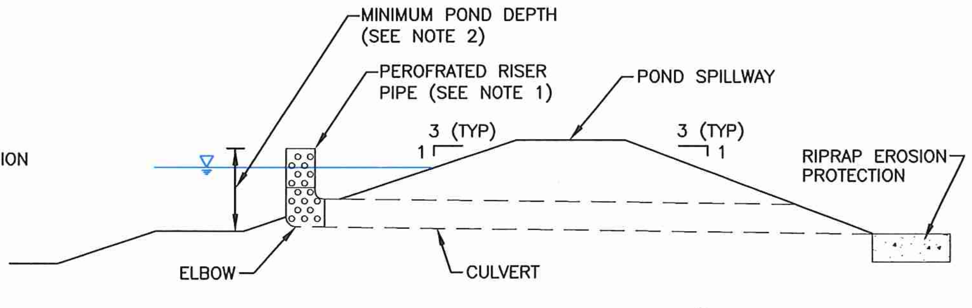




OUTLET STRUCTURE FOR PERMANENT POOL (P 5 5)



GABION RING (P 5 5)

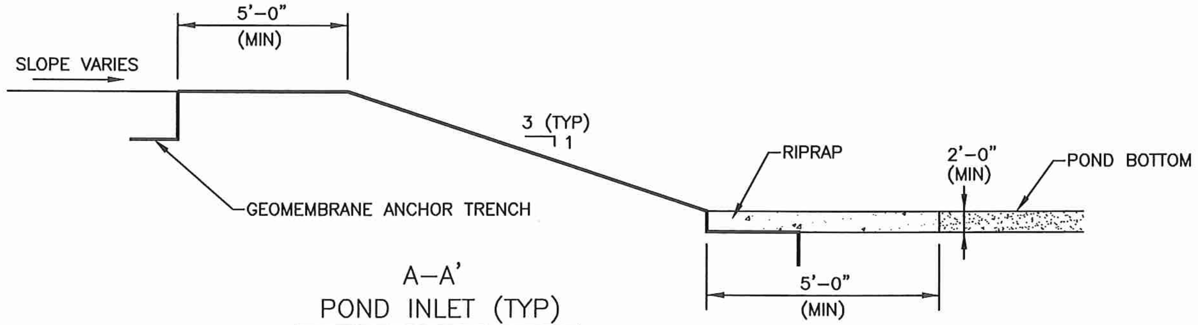


PERFORATED RISER PIPE (P 5 5)

OPTIONS FOR DRY PONDS

NOTES:

1. AS AN OPTION TO THE GABION RING, A PERFORATED RISER PIPE (SHOWN ON THIS SHEET) MAY ALSO BE USED, AS WELL AS A ROCK CHECK DAM.
2. MINIMUM POND DEPTH IS 4 FEET.
3. IF THE POND IS INSTALLED WITHOUT A LOW FLOW OUTLET THEN SEE NOTE 2 ON FIGURE 4.
4. VEGETATIVE SURFACING, GROUTED RIPRAP, RIPRAP, GABIONS, OR TURF REINFORCEMENT MAY BE USED TO ENSURE THE STABILITY OF THE POND INLET.



A–A'
POND INLET (TYP)
NTS

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DESIGN BY: AE
REVIEWED BY: NT

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EROSION CONTROL PLAN
TYPICAL POND OUTLET
STRUCTURES

HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS

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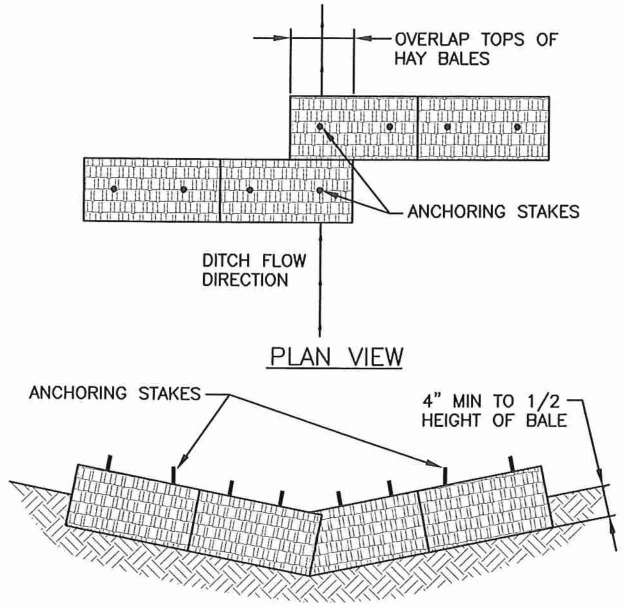
FIGURE 5

O:\0120\758\2214B EXPANSION\IIIF\FIGURES\FIG 5-TYPICAL OUTLET STRUCTURES.dwg, 11/15/2017 9:53:55 AM, rsellers, 1:2

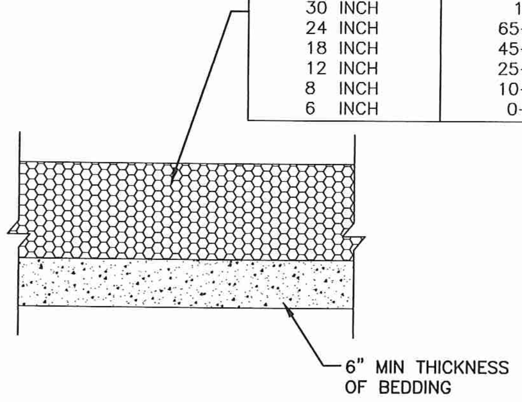


PLAN VIEW

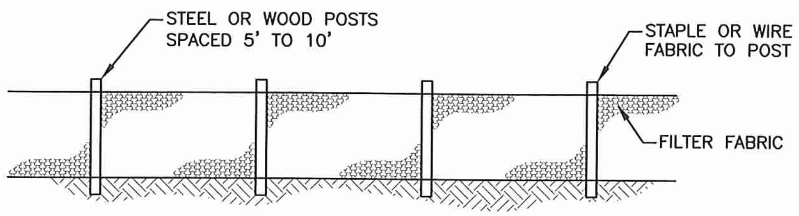
PROFILE VIEW

BALED HAY FOR EROSION CONTROL
NTS
(SEE NOTE 1)

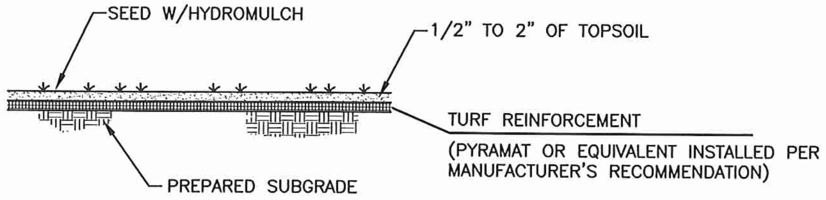
24" THICKNESS OF RIPRAP	
SIEVE SIZE SQUARE MESH	PERCENT PASSING
30 INCH	100
24 INCH	65–100
18 INCH	45–75
12 INCH	25–50
8 INCH	10–30
6 INCH	0–15



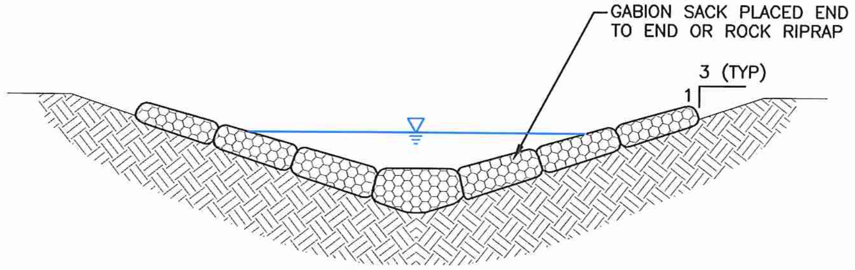
ROCK RIPRAP
NTS
(SEE NOTE 2)



TEMPORARY SEDIMENT CONTROL FENCE
NTS
(SEE NOTE 3)



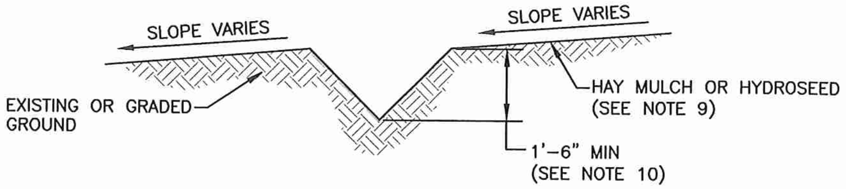
TURF REINFORCEMENT
NTS
(SEE NOTE 4)



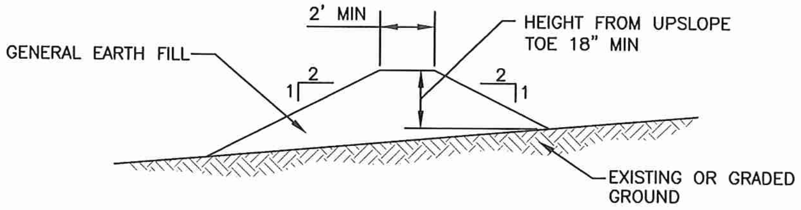
ROCK CHECK DAM
NTS
(SEE NOTE 5)

NOTES:

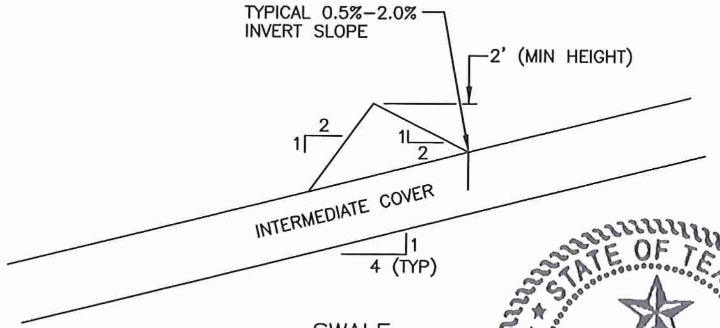
1. BALED HAY MAY BE USED IN NEWLY ESTABLISHED COVER AREAS OR DISTURBED/REGRADED SURFACES TO MAINTAIN SHEET FLOW UNTIL VEGETATION IS ESTABLISHED.
2. ROCK RIPRAP MAY BE USED IN AREAS WHERE CONCENTRATED FLOW WITH HIGH VELOCITIES MAY OCCUR (e.g., CULVERT INLETS/OUTLETS).
3. A TEMPORARY SEDIMENT CONTROL FENCE MAY BE USED IN CAPTURING SEDIMENT TRANSPORTED BY SHEET FLOW AND FOR DIVERSION OF FLOW FOR CONTROLLING SEDIMENT DISCHARGE.
4. TURF REINFORCEMENT MAY BE USED ON NEWLY ESTABLISHED SURFACES SUCH AS INTERMEDIATE COVER AND IN CHANNELS WHERE MODERATELY HIGH FLOW VELOCITIES ARE EXPECTED.
5. A ROCK CHECK DAM MAY BE USED IN CHANNELS TO SLOW DOWN FLOW VELOCITIES AND IMPROVE SEDIMENT CAPTURE.
6. A TEMPORARY DIVERSION CHANNEL MAY BE USED FOR SHORTENING SHEET FLOW DISTANCES IN UNDEVELOPED AREAS OR IN LARGER CHANNELS TO PROVIDE MEANDERING AND SLOWER FLOW VELOCITIES TO PREVENT IN-CHANNEL EROSION.
7. A TEMPORARY DIVERSION BERM MAY BE USED IN AREAS TO DIVERT FLOW FROM ENTERING STEEP SLOPED AREAS (e.g., TOP OF EXCAVATION) AND TO REDUCE SHEET FLOW LENGTHS.
8. A SWALE MAY BE USED IN AREAS TO DIVERT FLOW FROM ENTERING STEEP SLOPED AREAS (e.g., TOP OF EXCAVATION) AND TO REDUCE SHEET FLOW LENGTHS.
9. HAY MULCH AND HYDROSEED MAY ALSO BE USED FOR NEWLY ESTABLISHED SURFACES TO PROMOTE VEGETATION ESTABLISHMENT AND PREVENT EROSION.
10. THE VALUE SHOWN IS AT THE TIME OF CHANNEL INSTALLATION; CHANNEL WIDTH AND DEPTH VARY.



TEMPORARY DIVERSION CHANNEL
NTS
(SEE NOTE 6)



TEMPORARY DIVERSION BERM
NTS
(SEE NOTE 7)



SWALE
NTS
(SEE NOTE 8)

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DESIGN BY: AE
REVIEWED BY: NT

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EROSION CONTROL PLAN TYPICAL BMPs

HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS

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FIGURE 6

O:\0120\758\2214B EXPANSION\IIIF\FIGURES\FIG 6-TYPICAL BMPs.dwg, 11/15/2017 9:54:21 AM, rsellers, 1:2

APPENDIX IIIF-F-1

TEMPORARY ADD-ON SWALE DESIGN

Includes pages IIIF-F-1-1 through IIIF-F-1-12

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12-5-2017

SWALE DESIGN

This appendix includes the expected soil loss calculations for various swale spacing intervals on the side slope and top dome surfaces. An example calculation is provided on pages IIIF-F-1-2 through IIIF-F-1-4 for a vegetation cover of 60 percent. For the results of various percent vegetation covers and swale spacing intervals, refer to the table on page IIIF-F-1-5 and to Figure 2 – Swale Design Summary. If 60 percent vegetation coverage is not achieved within the 180-day period, secondary erosion control measures such as mulch, wood chips, compost, or similar TCEQ approved materials may be used to limit the soil loss to 50 tons/acre/year or less. In addition, a detention/sedimentation pond may also be used until the 60 percent vegetation coverage is achieved. Any secondary measure used will be documented in the Site Operating Record at the end of the 180-day period to document compliance with this plan. In addition, the date 60 percent vegetation coverage is achieved and the secondary measure is no longer needed will also be documented in the Site Operating Record.

Also included in this appendix are the sheet flow velocities for all swale spacing intervals on the side slopes and top dome surfaces. As noted in these calculations (pages IIIF-F-1-6 through IIIF-F-1-8), all velocities are acceptable.

Additionally, this appendix includes a calculation for the maximum drainage area that each swale can drain, as well as the maximum swale length. These calculations are included on pages IIIF-F-1-9 through IIIF-F-1-12.

Required: Determine the required spacing of the drainage swales for different percentages of vegetative cover for top dome surfaces and external embankment side slopes.

Method:

1. Estimate soil loss per acre based on percent ground cover and swale spacing for top dome surface and external side slope.
2. Summary

Notes:

1. The following example calculation procedure has been developed for 60 percent ground cover.
2. The table on page IIIF-F-1-5 includes the results of the following procedure for 60, 70, 80, and 90 percent ground cover, and various swale spacings. The results are also summarized on Figure 2 of Appendix IIIF-F

References:

1. SCS National Engineering Handbook, Chapter 3 - Erosion.
2. TNRCC, *Use of the USLE in Final Cover/Configuration Design*, 1993.
3. United States Environmental Protection Agency, *Solid Waste Disposal Facility Criteria Technical Manual*, 1993.

Solution:

1. Estimate soil loss per acre based on percent ground cover and swale spacing for top dome surface and external side slope.

Soil Loss Equation: $A=RKL_SCP$

Where:

A= Soil loss (tons/ac/yr)
R= Rainfall factor
K= Soil erodibility factor
L_S= Slope length/slope gradient factor
C= Plant cover or cropping management factor
P= Erosion practice factor

The rainfall factor, R, represents the average intensity for the maximum intensity, 30 minute storms over a 22 year period of record compiled by the SCS. Using Figure 1 (Ref 2), Average Annual Values of the R Factor, the R factor for Hardin County is:

R = 440

The soil erodibility factor, K, factor represents the resistance of a soil surface to erosion as a function of the soil's physical and chemical properties. The final cover top soil will consist of soils comparable to clay, which has K values that range from 0.13 to 0.29. Additionally, compost will be added to intermediate cover as necessary to protect against erosion. Therefore, the following is a conservative K value for the site (Table 1 on page 6, Ref. 2).

K = 0.19

The slope length/slope gradient factor, L_s, represents the erosion of the soil due to both slope length and degree of slope.

Case 1. Topslope
slope = 4 %
length = 500 ft

Case 2. Side Slope
slope = 25 %
length = 300 ft

Case 3. Topslope
slope = 4 %
length = 600 ft

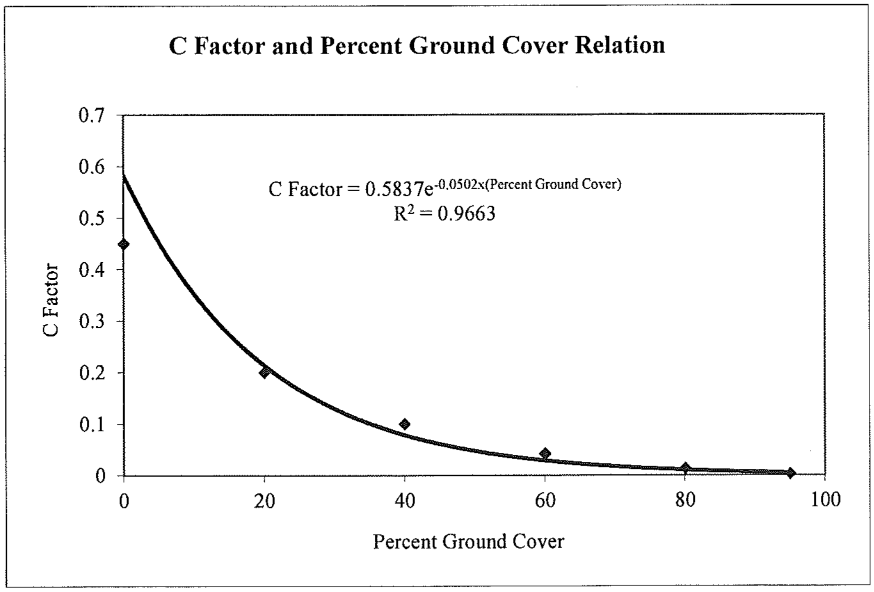
Case 4. Topslope
slope = 5 %
length = 700 ft

Using the above information and Figure 2 (Ref 2, p.9), the L_s factors are determined.

Case	Slope (%)	Slope Length (ft)	L_s
1. Top Slope	4	500	0.75
2. Side Slope	25	300	10.00
3. Top Slope	4	600	0.80
4. Top Slope	4	700	0.90

The plant cover or cropping management factor, C, represents the percentage of soil loss that would occur if the surface were partially protected by some combination of cover and management practices. C Factor for Permanent Pasture, Range, and Idle Land with No Appreciable Canopy has the following relation with percent ground cover (GC) (from Ref 2, p.7) .

% GC	C Factor:
0	0.45
20	0.2
40	0.1
60	0.042
80	0.013
95	0.003



C Factor= $0.5837e^{(-0.0502x60)}$
C Factor= 0.0420

The erosion control practice factor, P, measures the effect of control practices that reduce the erosion potential of the runoff by influencing drainage patterns, runoff concentration , and runoff velocity. Contouring for this site will be done only to establish vegetation.

P = 1.00

2. Estimate soil loss given percent ground cover and swale spacing.

Slope Condition	R	K	L_s	C	P	A (tons/ac/yr)
1. Top Slope 4% slope 500 ft length	440	0.19	0.75	0.0420	1.0	2.6
2. Side Slope 25% slope 300 ft length	440	0.19	10.00	0.0420	1.0	35.1
3. Top Slope 4% slope 600 ft length	440	0.19	0.80	0.0420	1.0	2.8
4. Top Slope 4% slope 700 ft length	440	0.19	0.90	0.0420	1.0	3.2

3. Summary
For a summary of soil loss rates for various percentages of ground cover, see Figure 2 in Appendix III-F and page IIIF-F-1-5.

Prep By: JRP
Date: 3-31-17

HARDIN COUNTY LANDFILL
0120-758-11-02
SOIL LOSS EVALUATION

Chkd By: NT
Date: 3-31-17

SOIL LOSS ESTIMATE SUMMARY TABLE

Case	Slope (%)	Length (ft)	L_s	Percent Ground Cover	C Factor	A (tons/ac/yr)
Top Slope	4	500	0.75	60	0.042	2.6
Top Slope	4	500	0.75	70	0.017	1.1
Top Slope	4	500	0.75	80	0.013	0.8
Top Slope	4	500	0.75	90	0.0064	0.4
Top Slope	4	600	0.80	60	0.042	2.8
Top Slope	4	600	0.80	70	0.017	1.1
Top Slope	4	600	0.80	80	0.013	0.9
Top Slope	4	600	0.80	90	0.0064	0.4
Top Slope	4	700	0.90	60	0.042	3.2
Top Slope	4	700	0.90	70	0.017	1.3
Top Slope	4	700	0.90	80	0.013	1.0
Top Slope	4	700	0.90	90	0.0064	0.5
Side Slope	25	300	10.00	60	0.042	35.1
Side Slope	25	300	10.00	70	0.017	14.2
Side Slope	25	300	10.00	80	0.013	10.9
Side Slope	25	300	10.00	90	0.0064	5.3
Side Slope	25	400	11.50	70	0.017	16.3
Side Slope	25	400	11.50	80	0.013	12.5
Side Slope	25	400	11.50	90	0.0064	6.1
Side Slope	25	500	12.00	70	0.017	17.1
Side Slope	25	500	12.00	80	0.013	13.0
Side Slope	25	500	12.00	90	0.0064	6.4
Side Slope	25	600	12.50	80	0.013	13.6
Side Slope	25	600	12.50	90	0.0064	6.7

*If top slope and side slope both exist, then the soil loss should be summed from each to determine whether or not additional sediment capture is required. This total will vary depending on percent vegetative cover and swale spacing for each slope.

Prep By: JRP
Date: 3-31-17

HARDIN COUNTY LANDFILL
0120-758-11-02
SHEET FLOW VELOCITY

Chkd By: NT
Date: 3-31-17

Required: Determine the sheet flow velocity for the top dome surfaces and external embankment side slopes and compare to the permissible non-erodible flow velocity.

Method:

1. Determine the peak velocities for the cases listed on page IIIF-F-1-5.
2. Compare to permissible velocities.
3. Conclusion.

References:

1. National Engineering Handbook, Section 4, Hydrology. *Chapter 15 - Travel Time, Time of Concentration and Lag.*

Solution: Use the typical case scenarios from the USLE calculation to determine the expected peak sheet flow velocity.

Case 1. Topslope			Case 2. Side Slope		
slope =	0.04	ft/ft	slope =	0.25	ft/ft
length =	300	ft	length =	300	ft
Case 3. Topslope			Case 4. Side Slope		
slope =	0.04	ft/ft	slope =	0.25	ft/ft
length =	400	ft	length =	400	ft
Case 5. Topslope			Case 6. Side Slope		
slope =	0.04	ft/ft	slope =	0.25	ft/ft
length =	500	ft	length =	500	ft
Case 7. Topslope			Case 8. Side Slope		
slope =	0.04	ft/ft	slope =	0.25	ft/ft
length =	600	ft	length =	600	ft

1. Determine the peak velocities for the cases listed on page IIIF-F-1-2.

Cultivated Straight Row (Overland Flow)

From Figure 15.2 (page 15-8 in Ref. 1), determine the velocities for all cases.

Case 1.	V =	1.8	ft/s
Case 2.	V =	4.5	ft/s
Case 3.	V =	1.8	ft/s
Case 4.	V =	4.5	ft/s
Case 5.	V =	1.8	ft/s
Case 6.	V =	4.5	ft/s
Case 7.	V =	1.8	ft/s
Case 8.	V =	4.5	ft/s

Note: Figure 15.2 is reproduced on page IIIF-F-1-8.

2. **Compare to permissible velocities.**

Summary of Velocities

	Condition	Minimum Percent Ground Coverage	Peak Velocity (ft/s)	Permissible Velocity[1] (ft/s)
Cultivated Straight Row	4%, 300 ft	>60%	1.8	5.0
	25%, 300 ft	>60%	4.5	5.0
	4%, 400 ft	>60%	1.8	5.0
	25%, 400 ft	>70%	4.5	5.0
	4%, 500 ft	>60%	1.8	5.0
	25%, 500 ft	>70%	4.5	5.0
	4%, 600 ft	>80%	1.8	5.0
	25%, 600 ft	>80%	4.5	5.0

[1] Permissible velocity information is from USACE EM 1110-0-1418, Chapter 5 - Evaluation of Stability.

3. **Conclusion.**

The peak velocities for each case are listed in the above summary table. As shown peak velocities are below permissible velocities for the conditions analyzed. After 180 days, at least 60 percent vegetation will be established in order to maintain permissible non-erodible velocities.

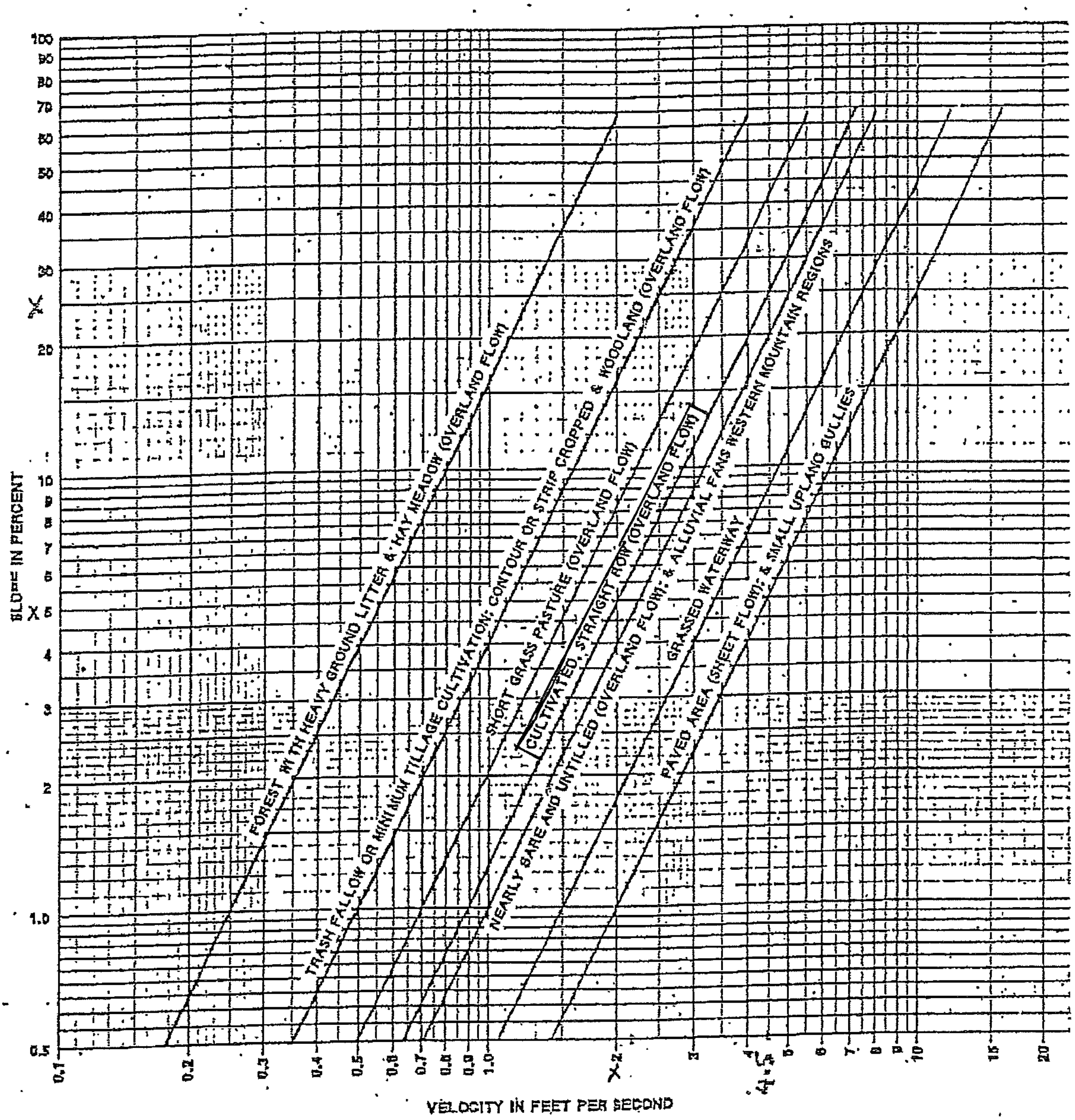


Figure 15.2.—Velocities for upland method of estimating T_c

Prep By: JRP
Date: 3-31-17

Chkd By: NT
Date: 3-31-17

HARDIN COUNTY LANDFILL
0120-758-11-02
SWALE ANALYSIS

Required: Analyze swales to determine the adequacy of the swale design.

Method:

1. Determine the 25-year frequency flow rates for a maximum swale drainage area for top slopes and side slopes using the Rational Method.
2. Determine maximum swale length that corresponds to the maximum swale drainage area.

Reference:

1. State of Texas, Department of Transportation, Bridge Division, Hydraulic Manual, March 2004.

Solution:

1. Determine the 25-year frequency flow rates for a maximum swale drainage area for top slopes and side slopes using the Rational Method.

$$Q = CIA$$

Where:
C= 0.8 (runoff coefficient, Ref 1.)
I = intensity, in/hr
A= drainage area, ac

$$I = \frac{b}{(t_c + d)^e}$$

b = 80 From Ref. 1, for Hardin County
d = 7.5 25-year storm event
e = 0.720
t_c is assumed to be 10 min.

I = 10.19 in/hr

For Top Slope:

Maximum Drainage Area (2 ft swale) =	12.0	acres
Maximum Drainage Area (1.5 ft swale) =	4.0	acres
Maximum Drainage Area (1 ft swale) =	2.0	acres
		acres
Flow Rate (2 ft swale) =	97.8	cfs
Flow Rate (1.5 ft swale) =	32.6	cfs
Flow Rate (1 ft swale) =	16.3	cfs

For Side Slope:

Maximum Drainage Area =	4.0	acres
Flow Rate (2 ft swale) =	32.6	cfs

2. Determine maximum swale length that corresponds to the maximum swale drainage area.

Condition (swale height)	Maximum Drainage Area (acres)	Minimum Swale Spacing[1] (ft)	Maximum Swale Length[2] (ft)
Top Slope (2 ft swale)	12.0	500	1,045
Top Slope (1.5 ft swale)	4.0	500	348
Top Slope (1 ft swale)	2.0	500	174
Side Slope (2 ft swale)	4.0	300	581

[1] Minimum swale spacing is taken from calculations provided on page IIIF-F-1-2.

[2] Maximum swale length calculated using the following equation:

Maximum Drainage Area x (43,560 sf/acre) / Minimum Swale Spacing

Prep By: JRP
Date: 3-31-17

Chkd By: NT
Date: 3-31-17

HARDIN COUNTY LANDFILL
0120-758-11-02
SWALE ANALYSIS

Flow Rate (cfs)	Bottom Slope (ft/ft)	n-value	Side Slope (left)	Side Slope (right)	Bottom Width (ft)	Normal Depth (ft)	Flow Vel. (fps)	Froude No.	Velocity Head (ft)	Energy Head (ft)	Flow Area (sq. ft.)	Top Width of Flow (ft)
2 ft Top Slope and Side Slope Swale												
97.8	0.005	0.03	2	17	0	1.78	3.22	0.601	0.16	1.94	30.35	33.96
32.6	0.005	0.03	2	4	0	1.83	3.20	0.587	0.16	1.99	10.20	11.06
1.5 ft Top Slope Swale												
32.6	0.005	0.03	2	17	0	1.18	2.45	0.562	0.09	1.27	13.30	22.48
1 ft Top Slope Swale												
16.3	0.005	0.03	2	17	0	0.91	2.06	0.536	0.07	0.97	7.93	17.36

Note: Calculations were performed using the HYDROCALC HYDRAULICS program developed by Dodson and Associates (Version 1.2a, 1996).

Maximum flow depth is 1.83 ft < 2.0 ft (swale height).

Design is acceptable.

Example Calculation: Calculate the normal depth for the swale for the maximum size side slope drainage area.

List of Symbols

$Q_d =$ design flow rate for channel, cfs
$R =$ hydraulic radius, ft
$n =$ Manning's roughness coefficient
$S =$ channel slope, ft/ft
$b =$ bottom width of channel, ft
$z_r =$ z-ratio (ratio of run to rise for channel sideslope) for right side slope of swale
$z_l =$ z-ratio (ratio of run to rise for channel sideslope) for left side slope of swale
$A_f =$ flow area, sf
$g =$ gravitational acceleration $= 32.2$ ft/s^2
$T =$ top width of flow, ft
$d =$ normal depth of swale, ft

The program uses an iterative process to calculate the normal depth of the swale to satisfy Manning's Equation

$$Q = \frac{1.486}{n} A R^{0.67} S^{0.5}$$

Design Inputs:

$Q_d =$ 32.6 cfs
$S =$ 0.005 ft/ft
$b =$ 0.1 ft
$z_r =$ 4 (H) : 1 (V)
$z_l =$ 2 (H) : 1 (V)
$n =$ 0.03

Step 1 - Based on the geometry of the swale cross-section, solve for R and A_f

$$R = \frac{bd + 1/2d^2(z_r + z_l)}{b + d((z_l^2 + 1)^{0.5} + (z_r^2 + 1))}$$

$$A_f = bd + 1/2d^2(z_r + z_l)$$

assume: $d =$ 1.83 ft

$R =$ 0.872 ft

$A_f =$ 10.23 sf

solve for Q: $Q =$ 32.6

if Q is not equal to Q_d, select a new d and repeat calculations

Step 2 - solve for velocity, T, Froude number, velocity head, and energy head

$Q = VA \Rightarrow \quad V = Q/A$

$V =$ 3.19 ft/s

$T = b + d(z_l + z_r)$

$T =$ 11.08 ft

$$F_r = \frac{V}{(gA/T)^{0.5}}$$

$F_r =$ 0.586

$$\text{Velocity Head} = \frac{V^2}{2g}$$

Velocity Head = 0.16 ft

Energy Head = water elevation + velocity head

Energy Head = 1.99 ft

APPENDIX IIIF-F-2

TEMPORARY LETDOWN DESIGN

Includes pages IIIF-F-2-1 through IIIF-F-2-33

STATE OF TEXAS
NEVZAT TURAN
84059
LICENSED
PROFESSIONAL ENGINEER

12-5-2017

LETDOWN (OR CHUTE) DESIGN

The temporary letdown structure options include open channel flow letdowns and pipe letdowns. Open channel flow letdowns will be lined with geomembrane, turf reinforcement mat, gabions, grouted concrete riprap, or rock riprap. The pipe letdowns are typically corrugated plastic pipe. Both types of letdowns will have an energy dissipater structure at the bottom of the letdown. Typical letdown details are shown on Figure 3 – Letdown Design Summary.

This appendix includes a demonstration to show that the letdown structure sizes shown on Figure 3 will contain the peak flow rate produced by the 25-year storm event. The geomembrane-lined and gabion-lined chutes (as well as turf reinforcement, rock riprap and grouted riprap-lined chutes) were analyzed for peak flow rates generated from drainage areas ranging from 5 acres to 20 acres. This analysis (pages IIIF-F-2-2 through IIIF-F-2-5) shows that the 2-foot-deep chutes (8 feet minimum bottom width) are adequate to handle the flow for up to 20 acres (i.e., the maximum flow depth calculated is 1.72 feet).

Also included in this appendix is an analysis for the 18-inch- and 24-inch-diameter temporary pipe letdowns for 25 percent slopes. The maximum flow that these pipes were capable of conveying was determined, and from this design flow rate a maximum drainage area size was calculated. The drainage area corresponds to the area that could drain to the pipe at each inlet. As noted on Figure 3, the use of pipe letdowns will be limited to 1 inlet per letdown. A hydraulic jump in the pipe letdown is provided on pages IIIF-F-2-14 through IIIF-F-2-33. The design summary for geomembrane-lined letdowns and pipe letdowns is provided on Figure 3.

CHUTE ANALYSIS

Required: Analyze chutes to determine size of chute for drainage area that ranges from 5 acres to 20 acres.

Method:

1. Determine the 25-year frequency flow rates for various sizes of chute drainage areas using the Rational Method.

Reference:

1. State of Texas, Department of Transportation, Bridge Division, Hydraulic Manual, March 2004.

Solution:

1. Determine the 25-year intensity flow rates.

$$Q = CIA$$

Where:

C= 0.8 (runoff coefficient, Ref 1.)
I = intensity, in/hr
A= drainage area, ac

$$I = \frac{b}{(t_c + d)^e}$$

b = 80 From Ref. 1, for Hardin County
d = 7.5 25-year storm event
e = 0.720
t_c is assumed to be 10 min.

I = 10.19 in/hr

Area (ac)	Flow (cfs)
5.0	40.8
10.0	81.5
15.0	122.3
20.0	163.0

Prep By: JRP
Date: 3-31-17

Chkd By: NT
Date: 3-31-17

HARDIN COUNTY LANDFILL
0120-758-11-02
EROSION CONTROL STRUCTURE DESIGN
GEOMEMBRANE-LINED CHUTE

Uniform flow design for the geomembrane-lined chutes on 4% slope.

Flow Rate (cfs)	Bottom Slope (ft/ft)	Manning's n	Side Slope (left)	Side Slope (right)	Bottom Width (ft)	Normal Depth (ft)	Flow Vel. (fps)	Froude Number	Velocity Head (ft)	Energy Head (ft)	Flow Area (sf)	Flow Top Width (ft)
40.8	0.04	0.01	2	2	8	0.34	13.69	4.282	2.91	3.26	2.98	9.37
81.5	0.04	0.01	2	2	8	0.51	17.53	4.544	4.77	5.29	4.65	10.06
122.3	0.04	0.01	2	2	8	0.65	20.13	4.690	6.30	6.95	6.08	10.61
163.0	0.04	0.01	2	2	8	0.77	22.15	4.793	7.62	8.40	7.36	11.08

Uniform flow design for the geomembrane-lined chutes on 25% slope.

Flow Rate (cfs)	Bottom Slope (ft/ft)	Manning's n	Side Slope (left)	Side Slope (right)	Bottom Width (ft)	Normal Depth (ft)	Flow Vel. (fps)	Froude Number	Velocity Head (ft)	Energy Head (ft)	Flow Area (sf)	Flow Top Width (ft)
40.8	0.25	0.01	2	2	8	0.20	24.41	9.872	9.26	9.46	1.67	8.80
81.5	0.25	0.01	2	2	8	0.30	31.55	10.499	15.47	15.77	2.58	9.20
122.3	0.25	0.01	2	2	8	0.38	36.55	10.871	20.76	21.14	3.35	9.53
163.0	0.25	0.01	2	2	8	0.45	40.54	11.161	25.54	25.99	4.02	9.81

Conclusions: Maximum normal depth is 0.72 feet. Chute design depth is 2.0 feet; therefore, design is acceptable.

1. Calculations were performed using the HYDROCALC Hydraulics for Windows program developed by Dodson and Associates (Version 1.2a, 1996).

Prep By: JRP
Date: 3-31-17

Chkd By: NT
Date: 3-31-17

HARDIN COUNTY LANDFILL
0120-758-11-02
EROSION CONTROL STRUCTURE DESIGN
GABION, TURF REINFORCEMENT MAT, ROCK RIPRAP, OR GROUTED CONCRETE RIPRAP-LINED CHUTE

Chute flow design for the gabion and rock riprap-lined chutes on 4% slope.

Flow Rate (cfs)	Bottom Slope (ft/ft)	Manning's n	Side Slope (left)	Side Slope (right)	Bottom Width (ft)	Normal Depth (ft)	Flow Vel. (fps)	Froude Number	Velocity Head (ft)	Energy Head (ft)	Flow Area (sf)	Flow Top Width (ft)
40.8	0.04	0.04	2	2	8	0.77	5.54	1.198	0.48	1.25	7.36	11.09
81.5	0.04	0.04	2	2	8	1.15	6.91	1.259	0.74	1.89	11.79	12.58
122.3	0.04	0.04	2	2	12	1.17	7.29	1.282	0.83	2.00	16.77	16.68
163.0	0.04	0.04	2	2	14	1.27	7.76	1.303	0.93	2.21	21.02	19.08

Chute flow design for the gabion and rock riprap-lined chutes on 25% slope.

Flow Rate (cfs)	Bottom Slope (ft/ft)	Manning's n	Side Slope (left)	Side Slope (right)	Bottom Width (ft)	Normal Depth (ft)	Flow Vel. (fps)	Froude Number	Velocity Head (ft)	Energy Head (ft)	Flow Area (sf)	Flow Top Width (ft)
40.8	0.25	0.04	2	2	8	0.45	10.14	2.790	1.60	2.05	4.02	9.81
81.5	0.25	0.04	2	2	8	0.68	12.85	2.946	2.57	3.25	6.34	10.71
122.3	0.25	0.04	2	2	8	0.86	14.70	3.036	3.36	4.21	8.32	11.43
163.0	0.25	0.04	2	2	8	1.01	16.11	3.099	4.03	5.04	10.12	12.04

Conclusions: Maximum normal depth is 1.27 feet. Chute design depth is 2.0 feet; therefore, design is acceptable.
Maximum velocity is 16.11 fps. As noted in footnote No. 2 below, the lining material will be selected so that the permissible velocity is not exceeded for erosion control.

1. Calculations were performed using the HYDROCALC Hydraulics for Windows program developed by Dodson and Associates (Version 1.2a, 1996).
2. Permissible velocities are listed below, and lining material will be selected so that these are not exceeded.

Description	Permissible Velocity (fps)
Turf Reinforcement Mat (based on Pyramat or equivalent. Refer to Sheet IIIF-F-2-6.)	25
Rock Riprap (based on Sheet IIIF-F-2-7 and a D_{50} of 18 inches. If other riprap is used, it will meet the D_{50} requirements listed on Sheet IIIF-F-2-7.)	11
Gabion/Concrete Grouted Riprap (based on Sheet IIIF-F-2-8 and a D_{50} of 0.62 ft. If other gabion is used, it will meet the D_{50} requirements listed on Sheet IIIF-F-2-8. The permissible velocity for concrete grouted riprap will actually be greater than 21 fps because it is classified as a rigid channel lining material.)	21

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Date: 3-31-17

Chkd By: NT
Date: 3-31-17

HARDIN COUNTY LANDFILL
0120-758-11-02
EROSION CONTROL STRUCTURE DESIGN
GABION, TURF REINFORCEMENT MAT, ROCK RIPRAP, OR GROUTED CONCRETE RIPRAP-LINED CHUTE

Chute flow design for the grouted riprap and turf reinforcement-lined chutes on 4% slope.

Flow Rate (cfs)	Bottom Slope (ft/ft)	Manning's n	Side Slope (left)	Side Slope (right)	Bottom Width (ft)	Normal Depth (ft)	Flow Vel. (fps)	Froude Number	Velocity Head (ft)	Energy Head (ft)	Flow Area (sf)	Flow Top Width (ft)
40.8	0.04	0.03	2	2	8	0.65	6.71	1.563	0.70	1.35	6.08	10.61
81.5	0.04	0.03	2	2	8	0.97	8.42	1.646	1.10	2.08	9.68	11.89
122.3	0.04	0.03	2	2	12	0.99	8.84	1.673	1.21	2.20	13.84	15.96
163.0	0.04	0.03	2	2	14	1.07	9.39	1.699	1.37	2.44	17.36	18.30

Chute flow design for the grouted riprap and turf reinforcement-lined chutes on 25% slope.

Flow Rate (cfs)	Bottom Slope (ft/ft)	Manning's n	Side Slope (left)	Side Slope (right)	Bottom Width (ft)	Normal Depth (ft)	Flow Vel. (fps)	Froude Number	Velocity Head (ft)	Energy Head (ft)	Flow Area (sf)	Flow Top Width (ft)
40.8	0.25	0.03	2	2	8	0.38	12.19	3.624	2.31	2.69	3.35	9.53
81.5	0.25	0.03	2	2	8	0.57	15.55	3.842	3.76	4.33	5.24	10.29
122.3	0.25	0.03	2	2	8	0.73	17.83	3.963	4.94	5.67	6.86	10.90
163.0	0.25	0.03	2	2	8	0.86	19.59	4.048	5.97	6.82	8.32	11.43

Conclusions: Maximum normal depth is 1.07 feet. Chute design depth is 2.0 feet; therefore, design is acceptable. Maximum velocity is 19.59 fps. As noted in footnote No. 2 below, the lining material will be selected so that the permissible velocity is not exceeded for erosion control.

1. Calculations were performed using the HYDROCALC Hydraulics for Windows program developed by Dodson and Associates (Version 1.2a, 1996).
2. Permissible velocities are listed below, and lining material will be selected so that these are not exceeded.

Description	Permissible Velocity (fps)
Turf Reinforcement Mat (based on Pyramat or equivalent. Refer to Sheet IIIF-F-2-6.)	25
Rock Riprap (based on Sheet IIIF-F-2-7 and a D_{50} of 18 inches. If other riprap is used, it will meet the D_{50} requirements listed on Sheet IIIF-F-2-7.)	11
Gabion/Concrete Grouted Riprap (based on Sheet IIIF-F-2-8 and a D_{50} of 0.62 ft. If other gabion is used, it will meet the D_{50} requirements listed on Sheet IIIF-F-2-8. The permissible velocity for concrete grouted riprap will actually be greater than 21 fps because it is classified as a rigid channel lining material.)	21

PROPEX
GEOSYNTHETICS

Pyramat® Turf Reinforcement Mat

Technical Data Sheet

Roll Sizes - 8.5 ft x 90 ft, 85 sq yd (2.6m x 27.4m, 8.44 sq m)

PYRAMAT high performance turf reinforcement mat (HPTRM) is a three-dimensional, lofty, woven polypropylene geotextile that is available in green or tan which is specially designed for erosion control applications on steep slopes and vegetated waterways. The matrix is composed of polypropylene monofilament yarns featuring X3® technology woven into a uniform configuration of resilient pyramid-like projections. The material exhibits very high interlock and reinforcement capacity with both soil and root systems, demonstrates superior UV resistance, and enhances seedling emergence.

PYRAMAT conforms to the property values listed below[1] and is manufactured at a Propex facility having achieved ISO 9001:2000 certification. Propex performs internal Manufacturing Quality Control (MQC) tests that have been accredited by the Geosynthetic Accreditation Institute – Laboratory Accreditation Program (GAI-LAP).

PRODUCT TEST DATA

Property	Test Method	MARV[2]
Physical		
Mass Per Unit Area	ASTM D-6566	13.5 oz sq yd (455 g sq m)
Thickness	ASTM D-6525	.4 in (10.2 mm)
Light Penetration (% Passing)	ASTM D-6567	10% (10%)
Color	Visual	Green, Tan
Mechanical		
Tensile Strength (Grab)	ASTM D-6818	4000 x 3000 lbs/ft (58.4 x 43.8 kN/m)
Elongation	ASTM D-6818	65% max (65% max)
Resiliency	ASTM D-6524	80% (80%)
Flexibility	ASTM D-6575	.534 in/lbs (615000 mg-cm) avg
Endurance		
UV Resistance @ 6000 hrs	ASTM D-4355	90% (90%)
Performance		
Velocity[3] (Vegetated)	Large Scale	25 ft/sec (7.6 m/sec)
Shear Stress[3] (Vegetated)	Large Scale	15 lbs sq ft (718 Pa)
Manning's "n"[4] (Unvegetated)	Calculated	.028 (.028)
Seedling Emergence	ECTC Draft Method #4	296% (296%)

NOTES

1. The property values listed are effective 08/2006 and are subject to change without notice.
2. MARV indicates minimum average roll value calculated as the typical minus two standard deviations. Statistically, it yields a 97.7% degree of confidence that any sample taken during quality assurance testing will exceed the value reported.
3. Maximum permissible velocity and shear stress has been obtained through vegetated testing programs featuring specific soil types, vegetation classes, flow conditions, and failure criteria. These conditions may not be relevant to every project nor are they replicated by other manufacturers. Please contact Propex for further information.
4. Calculated as typical values from large-scale flexible channel lining test programs with a flow depth of 6 to 12 inches.

The information presented herein, while not guaranteed, is to the best of our knowledge true and accurate. Except when agreed to in writing for specific conditions of use, no warranty or guarantee expressed or implied is made regarding the performance of any product, since the manner of use and handling are beyond our control. Nothing contained herein is to be construed as permission or as a recommendation to infringe any patent.

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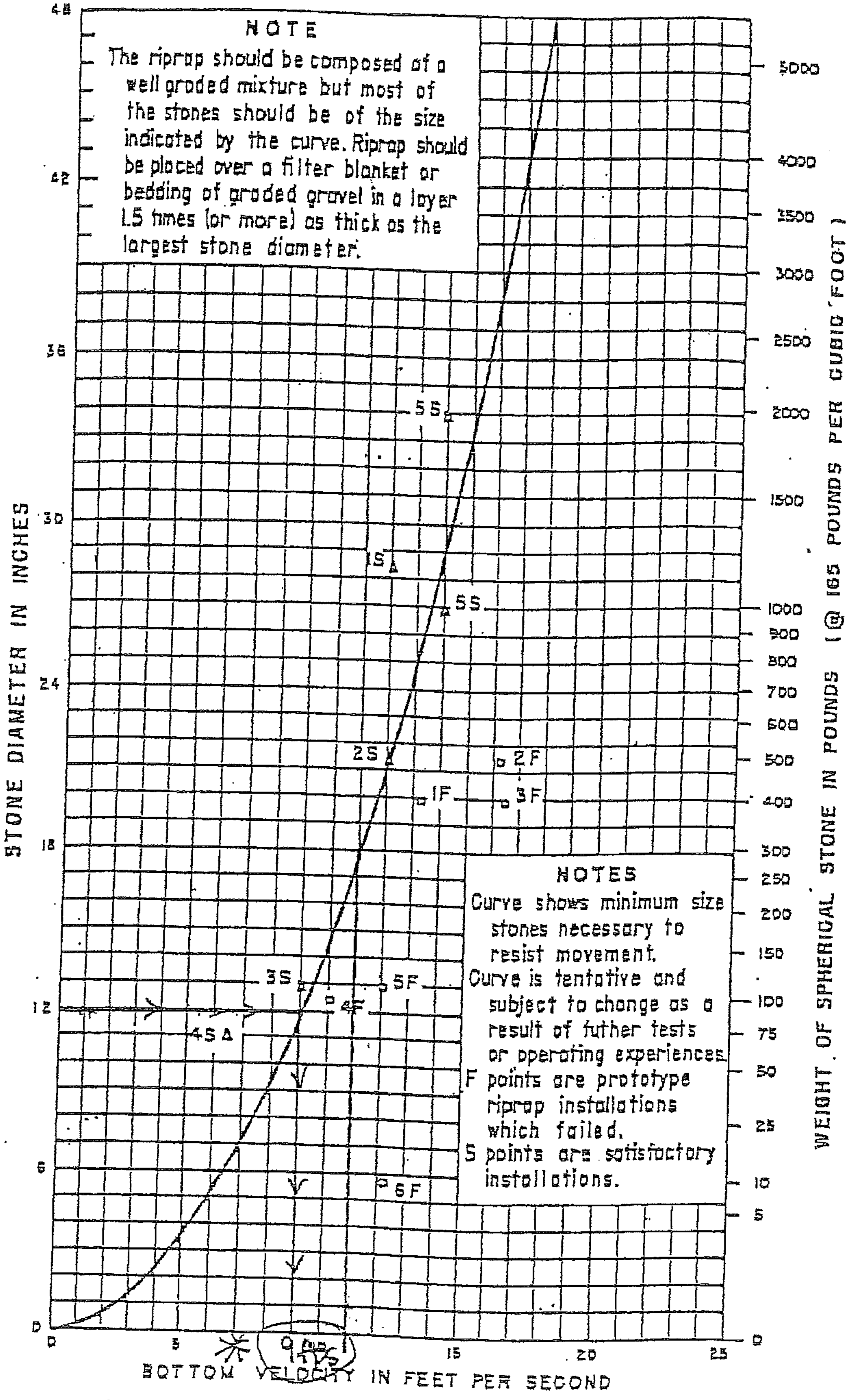
IVI-EC-2442-0407-C147

SIZE OF RIPRAP TO BE USED DOWNSTREAM FROM STILLING BASINS



NOTE

The riprap should be composed of a well graded mixture but most of the stones should be of the size indicated by the curve. Riprap should be placed over a filter blanket or bedding of graded gravel in a layer 1.5 times (or more) as thick as the largest stone diameter.

NOTES

Curve shows minimum size stones necessary to resist movement.

Curve is tentative and subject to change as a result of futher tests or operating experiences.

F points are prototype riprap installations which failed.

S points are satisfactory installations.

FIGURE 165.—Curve to determine maximum stone size in riprap mixture.

SOURCE: HYDRAULIC DESIGN OF STILLING BASINS AND ENERGY DISSIPATORS. US DEPT OF THE INTERIOR - BUREAU OF LAND RECLAMATION, [illegible].

Type	Thickness m ft	Rock Fill		Critical velocity m/s ft/sec	Limit velocity m/s ft/sec
		Size mm	d_{50} m ft		
Reno mattress	0.15 - 0.17	70 - 100	0.085	3.5	4.2
		70 - 150	0.110	4.2	4.5
	0.23 - 0.25	70 - 100	0.085	3.6	5.5
		70 - 150	0.120	4.5	6.1
	0.30	70 - 120	0.100	4.2	5.5
		100 - 150	0.125	5.0	6.4
Gabions	0.50 1.64	100 - 200	0.150 0.49	5.8 19	7.6 25
		120 - 250	0.190 0.62	6.4 21	8.0 26

Where the revetment has to be placed under water the thickness of the Reno mattress remains the same since it can be launched from a pontoon whereas rip rap has to be increased by 50% [12, 13, 49, 50, 51].

The big reduction in the revetment thickness, which is achieved using Reno mattress instead of rip rap, is of economic significance in protection projects in large rivers, given the same area of work, and, therefore, the quantity of material used.

2.2 Semi permeable and impermeable linings with sand asphalt mastic.

a) General characteristics of sand asphalt mastic grouted Reno mattress.

The combination of the stone filled Reno mattress and sand asphalt mastic has the characteristics of both gabion work and asphalt concrete. The addition of bituminous mastic to the Reno mattress produces a structure which combines the properties and performance of both materials. The mattress retains its flexibility, while the density of the filling is increased and therefore the efficiency of the protection. If all the voids between the stones in the layer are filled and the surface of the ress covered, the lining will be completely impervious. mastic also protects the wire mesh against corrosion and from abrasion by transported material.

The wire mesh reinforces the grouted stone layer and gives it strength in tension. Hence, the thickness of the combined structure can be considerably less than that of ordinary mastic grouted stone to withstand the same stresses. The resulting saving in bitumen and aggregate, and the increased flexibility due to the reduced thickness, have given rise to extensive use of this type of lining for protection in a variety of waterways.

b) Mix design of sand asphalt mastic.

To avoid excessive detail, only the fundamental data on mix design is given here. For fuller information, reference should be made to the specific publications listed in the bibliography [5, 6].

HARDIN COUNTY LANDFILL
0120-758-11-02
PIPE LETDOWN DESIGN

Required: Determine the maximum drainage area for 18-inch and 24-inch diameter letdown pipes using the BCAP computer program.

Method:

1. Determine the maximum flow for 18-inch and 24-inch diameter letdown pipes on the 25% side slope.
2. Determine the maximum drainage areas for the flows calculated in Step 1.

Reference:

1. State of Texas, Department of Transportation, Bridge Division, Hydraulic Manual, March 2004.

Solution:

1. Determine the maximum flow for 18-inch and 24-inch diameter letdown pipes on the 25% side slope.

The pipe design was analyzed using 10-feet of runout with a flat slope at the toe of the 25% sideslope (See Hydrocalc program output on pages IIIF-F-2-10 and IIIF-F-2-11).

Results:

Q18 =	13.0	cfs	maximum allowable flow in 18-in-dia pipe
Q24 =	22.0	cfs	maximum allowable flow in 24-in-dia pipe

2. Determine the maximum drainage areas for the flows calculated in Step 1.

$Q = CIA$

Where:
C= 0.8 (runoff coefficient, Ref 1.)
I = intensity, in/hr
A= drainage area, ac

$$I = \frac{b}{(t_c + d)^e}$$

b =	80	From Ref. 1, for Hardin County
d =	7.5	25-year storm event
e =	0.720	

t_c is assumed to be 10 min.

I = 10.19 in/hr

$A = Q / (CI)$

Pipe Diameter (in)	Flow (cfs)	Area (ac)
18	13.0	1.6
24	22.0	2.7

Conclusion: The maximum allowable drainage area for a 18-inch diameter letdown pipe is 1.6 acres for each inlet and for a 24-inch diameter letdown pipe is 2.7 acres for each inlet. The minimum berm height is 3 feet for both pipes. (Figure 3 details).

```
                        PIPE CULVERT ANALYSIS
                COMPUTATION OF CULVERT PERFORMANCE CURVE

                             April 4, 2017

==============================================================================
                          PROGRAM INPUT DATA
DESCRIPTION                                                          VALUE
------------------------------------------------------------------------------
Culvert Diameter (ft)...................................          1.5
FHWA Chart Number.......................................          2
FHWA Scale Number (Type of Culvert Entrance)............          1
Manning's Roughness Coefficient (n-value)...............          0.012
Entrance Loss Coefficient of Culvert Opening............          0.5
Culvert Length (ft).....................................        200.0
Invert Elevation at Downstream end of Culvert (ft)......         80.0
Invert Elevation at Upstream end of Culvert (ft)........        130.0
Culvert Slope (ft/ft)...................................          0.25

Starting Flow Rate (cfs)................................         10.0
Incremental Flow Rate (cfs).............................          1.0
Ending Flow Rate (cfs)..................................         20.0

Starting Tailwater Depth (ft)...........................          0.0
Incremental Tailwater Depth (ft)........................          0.1
Ending Tailwater Depth (ft).............................          1.0

==============================================================================

                         COMPUTATION RESULTS

    Flow  Tailwater  Headwater (ft)      Normal  Critical  Depth at   Outlet
    Rate      Depth  Inlet    Outlet      Depth     Depth    Outlet Velocity
   (cfs)       (ft)  Control  Control      (ft)      (ft)      (ft)    (fps)
------------------------------------------------------------------------------
    10.0        0.0     2.06      0.0      0.43      1.22      0.43    24.26
    11.0        0.1     2.32      0.0      0.45      1.27      0.45    24.88
    12.0        0.2     2.6       0.0      0.47      1.31      0.47    25.5
    13.0        0.3     2.9       0.0      0.49      1.35      0.49    26.13
    14.0        0.4     3.23      0.0      0.51      1.38      0.51    26.69
    15.0        0.5     3.58      0.0      0.53      1.41      0.53    27.17
    16.0        0.6     3.95      0.0      0.54      1.42      0.54    27.6
    17.0        0.7     4.35      0.0      0.56      1.44      0.56    28.08
    18.0        0.8     4.78      0.0      0.58      1.45      0.58    28.59
    19.0        0.9     5.23      0.0      0.6       1.46      0.6     28.98
    20.0        1.0     5.7       0.0      0.61      1.47      0.61    29.41

==============================================================================
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                    PIPE CULVERT ANALYSIS
           COMPUTATION OF CULVERT PERFORMANCE CURVE

                        April 4, 2017

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                         PROGRAM INPUT DATA
DESCRIPTION                                                              VALUE
-------------------------------------------------------------------------------
Culvert Diameter (ft).....................................          2.0
FHWA Chart Number.........................................          2
FHWA Scale Number (Type of Culvert Entrance)..............          1
Manning's Roughness Coefficient (n-value).................          0.012
Entrance Loss Coefficient of Culvert Opening..............          0.5
Culvert Length (ft).......................................        200.0
Invert Elevation at Downstream end of Culvert (ft)........         80.0
Invert Elevation at Upstream end of Culvert (ft)..........        130.0
Culvert Slope (ft/ft).....................................          0.25

Starting Flow Rate (cfs)..................................         15.0
Incremental Flow Rate (cfs)...............................          1.0
Ending Flow Rate (cfs)....................................         25.0

Starting Tailwater Depth (ft).............................          0.0
Incremental Tailwater Depth (ft)..........................          0.1
Ending Tailwater Depth (ft)...............................          1.0

===============================================================================

                        COMPUTATION RESULTS

    Flow  Tailwater   Headwater (ft)      Normal  Critical  Depth at   Outlet
    Rate      Depth   Inlet    Outlet      Depth     Depth    Outlet  Velocity
   (cfs)       (ft)   Control  Control      (ft)      (ft)      (ft)     (fps)
-------------------------------------------------------------------------------
    15.0       0.0       1.96     0.0       0.47      1.4       0.47     26.51
    16.0       0.1       2.12     0.0       0.49      1.44      0.49     26.94
    17.0       0.2       2.27     0.0       0.5       1.49      0.5      27.45
    18.0       0.3       2.37     0.0       0.52      1.53      0.52     27.92
    19.0       0.4       2.52     0.0       0.53      1.57      0.53     28.34
    20.0       0.5       2.67     0.0       0.55      1.61      0.55     28.71
    21.0       0.6       2.82     0.0       0.56      1.64      0.56     29.19
    22.0       0.7       2.99     0.0       0.57      1.68      0.57     29.49
    23.0       0.8       3.16     0.0       0.59      1.71      0.59     29.9
    24.0       0.9       3.34     0.0       0.6       1.74      0.6      30.28
    25.0       1.0       3.53     0.0       0.61      1.76      0.61     30.63

===============================================================================
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HARDIN COUNTY LANDFILL
0120-758-11-02
PIPE LETDOWN RIPRAP DESIGN

Required: Determine the Riprap size and Dimensions for 18-inch and 24-inch diameter letdown pipes using Riprap Apron Design provided by the Reference 1.

Method:

1. Determine the hydraulic conditions at the outlet of 18-inch and 24-inch diameter letdown pipes using the hydraulic design HydroCalc.
2. Determine the riprap size and apron dimensions for each pipe letdown

Reference:

1. U.S. Department of Transportation - Federal Highway Administration. Hydraulic Engineering Circular No. 14, Third Edition. *Hydraulic Design of Energy Dissipators for Culverts and Channels*. Publication No. FHWA-NHI-06-086, July 2006.

Solution:

1. Determine the hydraulic parameters from pages IIIF-F-2-10 (pipe diameter 18-inches) and IIIF-F-2-11 (pipe diameter 24-inches):

Parameter	Symbol	18-inch Dia. Culvert	24-inch Dia. Culvert
Design flow rates, cfs	Q=	13	22
Pipe Diameters, ft	D=	1.5	2
Depth at the pipe outlet, ft	y_n=	0.49	0.57
Adjusted culvert rise, ft	D'=	0.995	1.29
Tailwater Depth[1], ft	TW=	0.3	0.7

[1] Tailwater depth is the pipe diameter when the calculated tailwater depth is higher per Reference 1.

$$D_{50} = 0.2 \times D \left[\frac{Q}{\sqrt{g} \times D^{2.5}} \right]^{4/3} \times \left[\frac{D}{TW} \right]$$ Eq. 10.4 (page 10-17 of Ref. 1)

$$D' = \frac{D + y_n}{2}$$ Eq. 10.5 (page 10-17 of Ref. 1)

D_{50} = Riprap size in feet

Riprap Classes and Apron Dimensions[1]

Class	D_{50} (in)	Apron Length[2] (ft)	Apron Depth (ft)
1	5	4xD'	$3.5xD_{50}$
2	6	4xD'	$3.3xD_{50}$
3	10	5xD'	$2.4xD_{50}$
4	14	6xD'	$2.2xD_{50}$
5	20	7xD'	$2.0xD_{50}$
6	22	8xD'	$2.0xD_{50}$

[1] This table has been reproduced from Table 10.1 included on page 10-18 of Reference 1.

[2] D' is the culvert rise.

Design Parameter	18-inch Dia. Culvert	24-inch Dia. Culvert
D_{50}, calculated, inches =	24.3	14.9
D_{50}, selected, inches =	5	5
Apron Length, calculated, feet =	3.98	5.14
Apron Length, selected, feet =	10	10
Apron Depth, calculated, inches =	17.5	17.5
Apron Depth, selected, inches =	18	18

Conclusion:

Riprap sizes for pipe diameters of 18-inches and 24-inches are selected conservatively. The calculated apron length is increased to 10 feet in the design. The apron depth used is higher than the calculated apron depth. Therefore, the design of the pipe letdown outlet energy dissipater calculations are acceptable and channels at the pipe outlets will be stable.

HARDIN COUNTY LANDFILL
0120-758-11-02
PIPE LETDOWN DESIGN

Required: Determine the maximum drainage area for 18-inch and 24-inch diameter letdown pipes using the HY-8 computer program.

Method:

1. Determine the maximum flow for 18-inch and 24-inch diameter letdown pipes on the 25% side slope.
2. Determine the maximum drainage areas for the flows calculated in Step 1.

Reference:

1. State of Texas, Department of Transportation, Bridge Division, Hydraulic Manual, March 2004.

Note: The pipe letdown analysis has been performed using the HY-8 Culvert Hydraulic Analysis Program which is available from the Federal Highway Administration Web Page:
https://www.fhwa.dot.gov/engineering/hydraulics/software/hy8/ [follow link to downloadable files and info]

The program was developed to analyze culverts with changing slopes.

Solution:

1. Determine the maximum flow for 18-inch and 24-inch diameter letdown pipes on the 25% side slope.

The following pages include the program outputs for the 18-in diameter culvert and 24-in diameter culvert. Pages IIIF-F-2-16 through IIIF-F-2-33 include rating tables that show if the hydraulic jump occurs within the pipe or not. The results also include pipe outlet velocity for each flow rate as well as the tailwater depth and velocity in the channel ("Tailwater Velocity"). The analysis indicated that a hydraulic jump does not occur in the letdown pipe.

The flow ratings are used to calculate the maximum allowable top dome drainage area for each pipe size analyzed (Step 2). The maximum flow rate has been identified as the upper limit for the culvert performance curve. The corrugated polyethylene (PE) pipe option has been used with a Manning's Coefficient of 0.024.

The pipe design was analyzed using 10-feet of runout with a flat slope at the toe of the 25 percent sideslope

Results:

Q18=	15.0	cfs	maximum allowable flow in 18-in-dia pipe
Q24 =	24.0	cfs	maximum allowable flow in 24-in-dia pipe

2. Determine the maximum drainage areas for the flows calculated in Step 1.

$$Q = CIA$$

Where:

C= 0.8 (runoff coefficient, Ref 1.)
I = intensity, in/hr
A= drainage area, ac

$$I = \frac{b}{(t_c + d)^e}$$

b = 80 From Ref. 1, for Hardin County
d = 7.5 25-year storm event
e = 0.720
t_c is assumed to be 10 min.

I = 10.19 in/hr

$$A = Q / (CI)$$

Pipe Diameter (in)	Flow (cfs)	Area (ac)
18	15.0	1.8
24	24.0	2.9

Conclusion: The maximum allowable drainage area for a 18-inch diameter letdown pipe is 1.8 acres for each inlet and for a 24-inch diameter letdown pipe is 2.9 acres for each inlet. The minimum berm height is 2.5 feet for a 18-inch diameter pipe and 3 feet for 24-inch diameter pipe. (Figure 3 in Appendix IIIF-F details indicate 1 foot berm above the pipe).

HY-8 Culvert Analysis Report

For 18 inch Diameter Letdown Pipe

Crossing Discharge Data

Discharge Selection Method: Specify Minimum, Design, and Maximum Flow

Minimum Flow: 8 cfs

Design Flow: 13 cfs

Maximum Flow: 15 cfs

Table 1 - Summary of Culvert Flows at Crossing: SideSlope Pipe Letdown

Headwater Elevation (ft)	Total Discharge (cfs)	Culvert 2 Discharge (cfs)	Roadway Discharge (cfs)	Iterations
124.71	8.00	8.00	0.00	1
124.90	8.70	8.70	0.00	1
125.11	9.40	9.40	0.00	1
125.33	10.10	10.10	0.00	1
125.57	10.80	10.80	0.00	1
125.83	11.50	11.50	0.00	1
126.03	12.20	12.04	0.14	16
126.09	12.90	12.19	0.68	6
126.10	13.00	12.21	0.77	4
126.17	14.30	12.41	1.87	5
126.21	15.00	12.50	2.48	4
126.00	11.96	11.96	0.00	Overtopping

Rating Curve Plot for Crossing: SideSlope Pipe Letdown

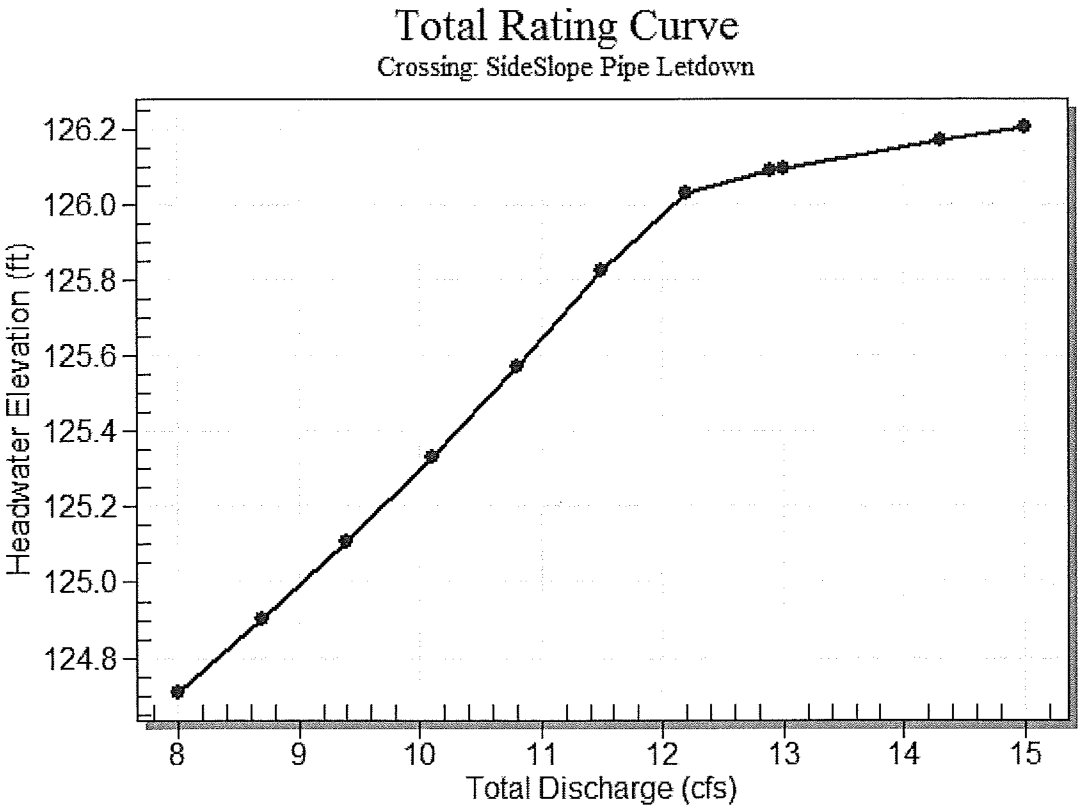

Table 2 - Culvert Summary Table: Culvert 2

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
8.00	8.00	124.71	2.210	0.790	5-S2n	0.000	0.352	0.927	0.306	10.340	5.678
8.70	8.70	124.90	2.402	0.903	5-S2n	0.000	0.352	0.982	0.321	10.689	5.842
9.40	9.40	125.11	2.610	1.026	5-S2n	0.000	0.352	1.040	0.336	11.041	5.994
10.10	10.10	125.33	2.834	1.158	5-S2n	0.000	0.352	1.099	0.350	11.362	6.142
10.80	10.80	125.57	3.074	1.300	5-S2n	0.000	0.352	1.169	0.364	11.656	6.282
11.50	11.50	125.83	3.327	1.451	5-S2n	0.000	0.352	1.247	0.377	11.926	6.415
12.20	12.04	126.03	3.530	1.573	5-S2n	0.000	0.352	1.315	0.390	12.118	6.540
12.90	12.19	126.09	3.589	1.610	5-S2n	0.000	0.352	1.322	0.403	12.171	6.662
13.00	12.21	126.10	3.597	1.614	5-S2n	0.000	0.352	1.322	0.405	12.178	6.678
14.30	12.41	126.17	3.673	1.661	5-S2n	0.000	0.352	1.329	0.427	12.243	6.891
15.00	12.50	126.21	3.709	1.683	5-S2n	0.000	0.352	1.333	0.439	12.273	6.998

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Double Broken-back Culvert

Inlet Elevation (invert): 122.50 ft,

Upper Break Elevation (invert): 122.20 ft,

Lower Break Elevation (invert): 78.00 ft,

Culvert Length: 183.58 ft,

Upper Culvert Section Slope: 0.0750

Steep Culvert Section Slope: 0.2695

Runout Culvert Section Slope: 0.0400

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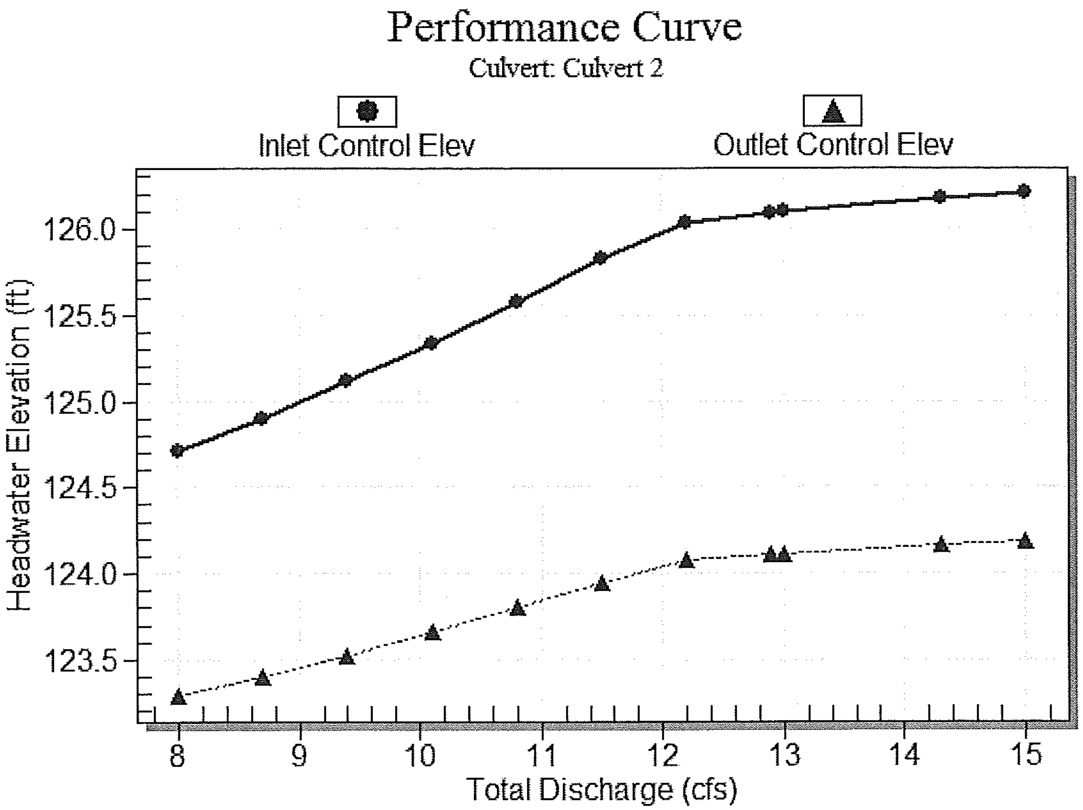
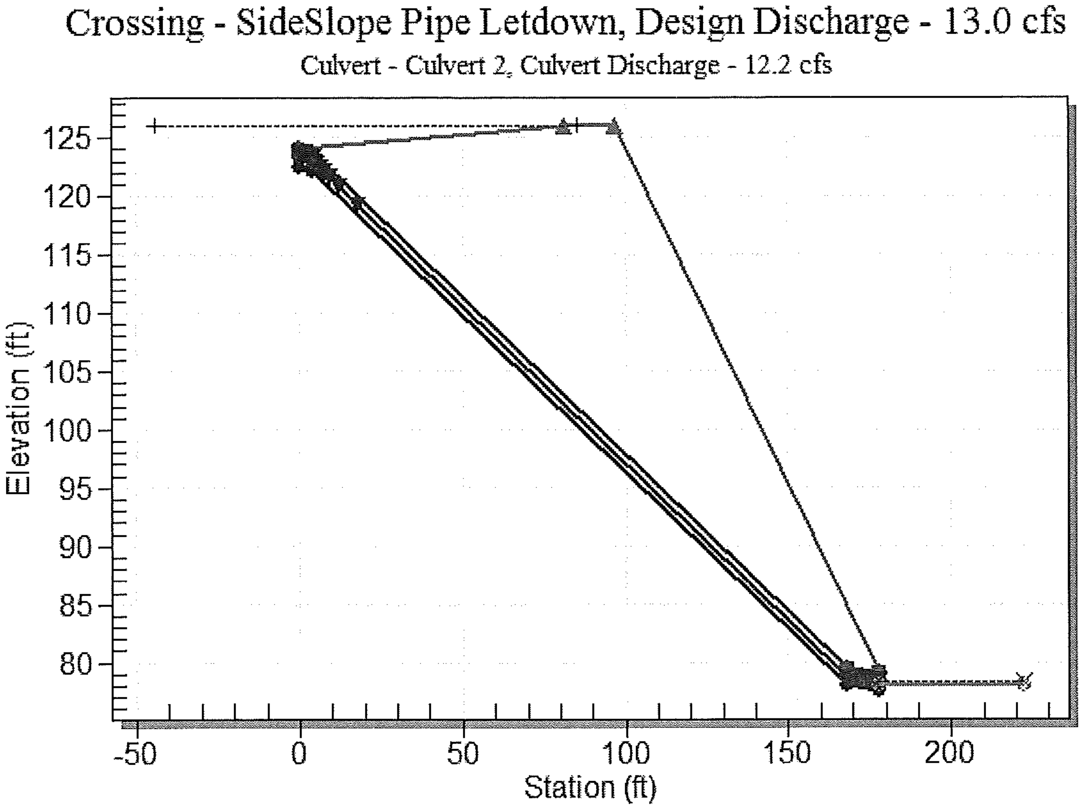
Culvert Performance Curve Plot: Culvert 2



Water Surface Profile Plot for Culvert: Culvert 2



Site Data - Culvert 2

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: 122.50 ft

Upper Break Station: 4.00 ft

Upper Break Elevation: 122.20 ft

Lower Break Station: 168.00 ft

Lower Break Elevation: 78.00 ft

Outlet Station: 178.00 ft

Outlet Elevation: 77.60 ft

Number of Barrels: 1

Culvert Data Summary - Culvert 2

Barrel Shape: Circular

Barrel Diameter: 1.50 ft

Upper & Middle Section Material: Corrugated PE

Lower Section Material: Corrugated PE

Embedment: 0.00 in

Upper & Middle Section Manning's n: 0.0240

Lower Section Manning's n: 0.0240

Culvert Type: Double Broken-back

Inlet Configuration: Mitered to Conform to Slope

Inlet Depression: NONE

Table 3 - Downstream Channel Rating Curve (Crossing: SideSlope Pipe Letdown)

Flow (cfs)	Water Surface Elev (ft)	Depth (ft)	Velocity (ft/s)	Shear (psf)	Froude Number
8.00	78.31	0.31	5.68	0.95	1.93
8.70	78.32	0.32	5.84	1.00	1.94
9.40	78.34	0.34	5.99	1.05	1.95
10.10	78.35	0.35	6.14	1.09	1.96
10.80	78.36	0.36	6.28	1.13	1.97
11.50	78.38	0.38	6.41	1.18	1.98
12.20	78.39	0.39	6.54	1.22	1.99
12.90	78.40	0.40	6.66	1.26	2.00
13.00	78.40	0.40	6.68	1.26	2.00
14.30	78.43	0.43	6.89	1.33	2.01
15.00	78.44	0.44	7.00	1.37	2.02

Tailwater Channel Data - SideSlope Pipe Letdown

Tailwater Channel Option: Trapezoidal Channel

Bottom Width: 4.00 ft

Side Slope (H:V): 2.00 (_:1)

Channel Slope: 0.0500

Channel Manning's n: 0.0240

Channel Invert Elevation: 78.00 ft

HY-8 Culvert Analysis Report

For 24 inch Diameter Letdown Pipe

Crossing Discharge Data

Discharge Selection Method: Specify Minimum, Design, and Maximum Flow

Minimum Flow: 16 cfs

Design Flow: 22 cfs

Maximum Flow: 24 cfs

Table 1 - Summary of Culvert Flows at Crossing: SideSlope Pipe Letdown

Headwater Elevation (ft)	Total Discharge (cfs)	Culvert 2 Discharge (cfs)	Roadway Discharge (cfs)	Iterations
125.38	16.00	16.00	0.00	1
125.51	16.80	16.80	0.00	1
125.65	17.60	17.60	0.00	1
125.80	18.40	18.40	0.00	1
125.96	19.20	19.20	0.00	1
126.05	20.00	19.66	0.31	9
126.10	20.80	19.91	0.87	6
126.15	21.60	20.11	1.46	5
126.17	22.00	20.20	1.76	4
126.22	23.20	20.46	2.72	5
126.25	24.00	20.62	3.36	4
126.00	19.39	19.39	0.00	Overtopping

Rating Curve Plot for Crossing: SideSlope Pipe Letdown

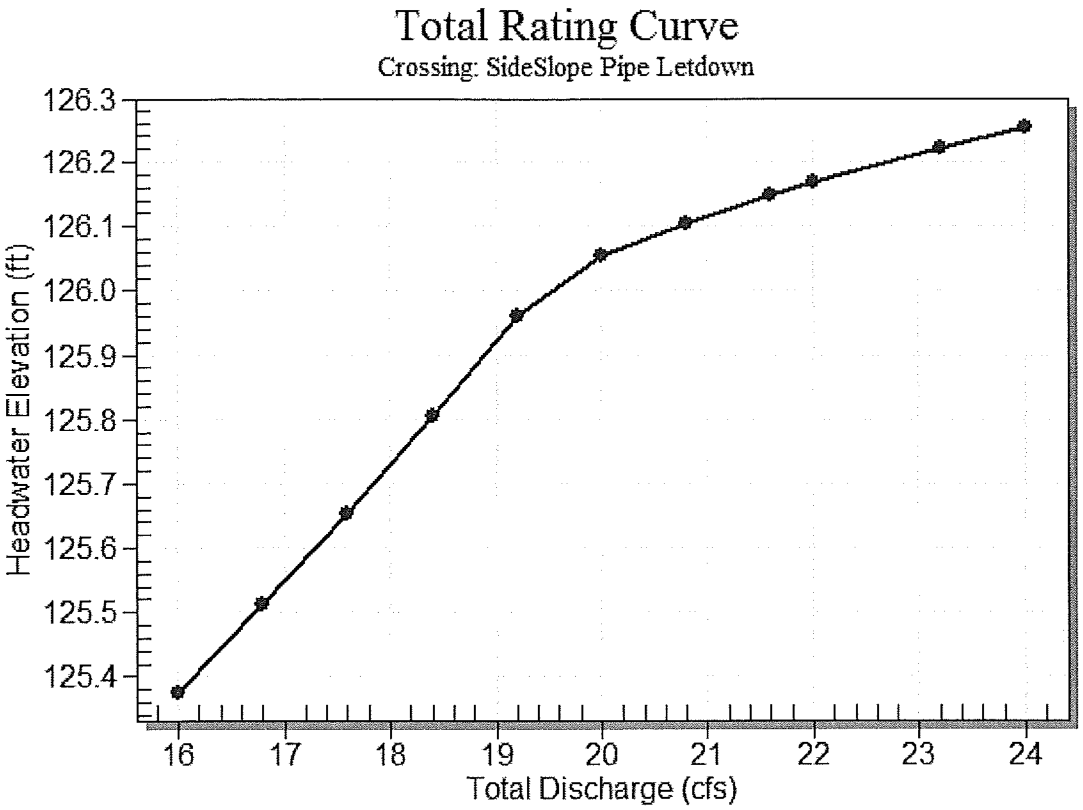


Table 2 - Culvert Summary Table: Culvert 2

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
16.00	16.00	125.38	2.876	1.084	5-S2n	0.000	0.473	1.177	0.456	13.368	7.145
16.80	16.80	125.51	3.012	1.161	5-S2n	0.000	0.473	1.215	0.469	13.597	7.259
17.60	17.60	125.65	3.154	1.242	5-S2n	0.000	0.473	1.253	0.481	13.811	7.367
18.40	18.40	125.80	3.304	1.326	5-S2n	0.000	0.473	1.292	0.494	14.038	7.475
19.20	19.20	125.96	3.460	1.415	5-S2n	0.000	0.473	1.333	0.506	14.281	7.574
20.00	19.66	126.05	3.553	1.467	5-S2n	0.000	0.473	1.356	0.518	14.415	7.675
20.80	19.91	126.10	3.604	1.496	5-S2n	0.000	0.473	1.369	0.529	14.486	7.771
21.60	20.11	126.15	3.647	1.520	5-S2n	0.000	0.473	1.380	0.541	14.543	7.864
22.00	20.20	126.17	3.666	1.531	5-S2n	0.000	0.473	1.384	0.546	14.569	7.908
23.20	20.46	126.22	3.721	1.562	5-S2n	0.000	0.473	1.397	0.563	14.639	8.043
24.00	20.62	126.25	3.754	1.580	5-S2n	0.000	0.473	1.405	0.574	14.681	8.127

****

Double Broken-back Culvert

Inlet Elevation (invert): 122.50 ft,

Upper Break Elevation (invert): 122.20 ft,

Lower Break Elevation (invert): 78.00 ft,

Culvert Length: 183.58 ft,

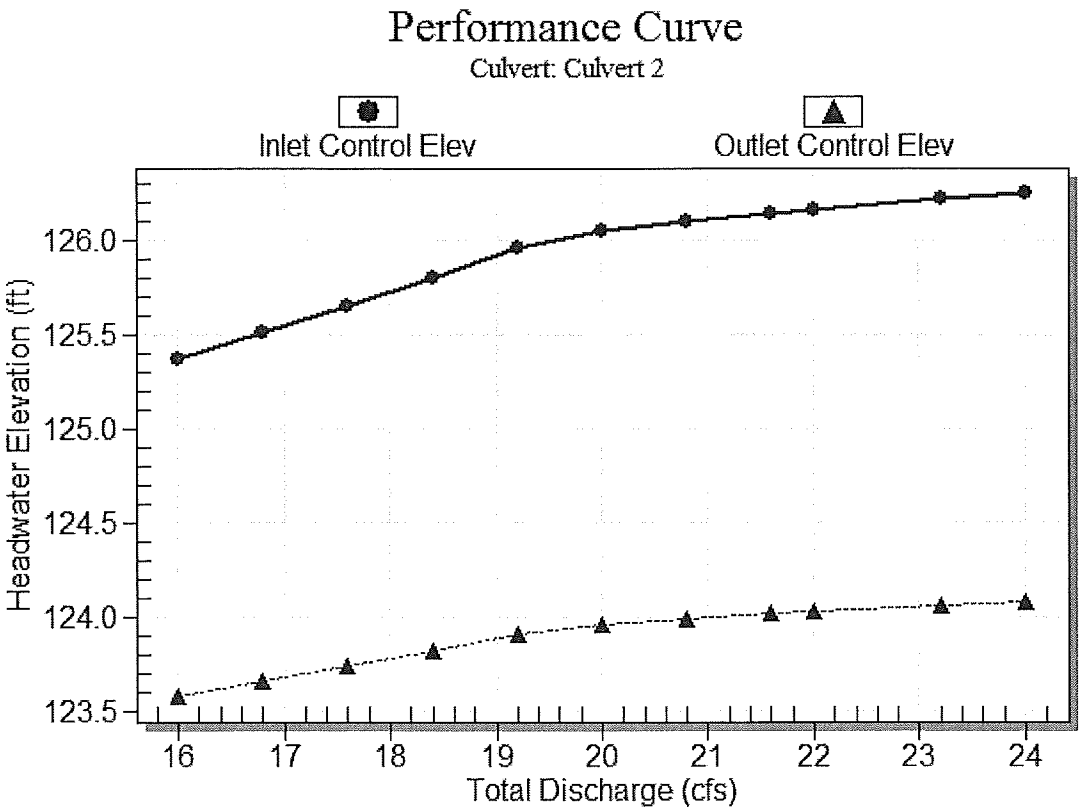
Upper Culvert Section Slope: 0.0750

Steep Culvert Section Slope: 0.2695

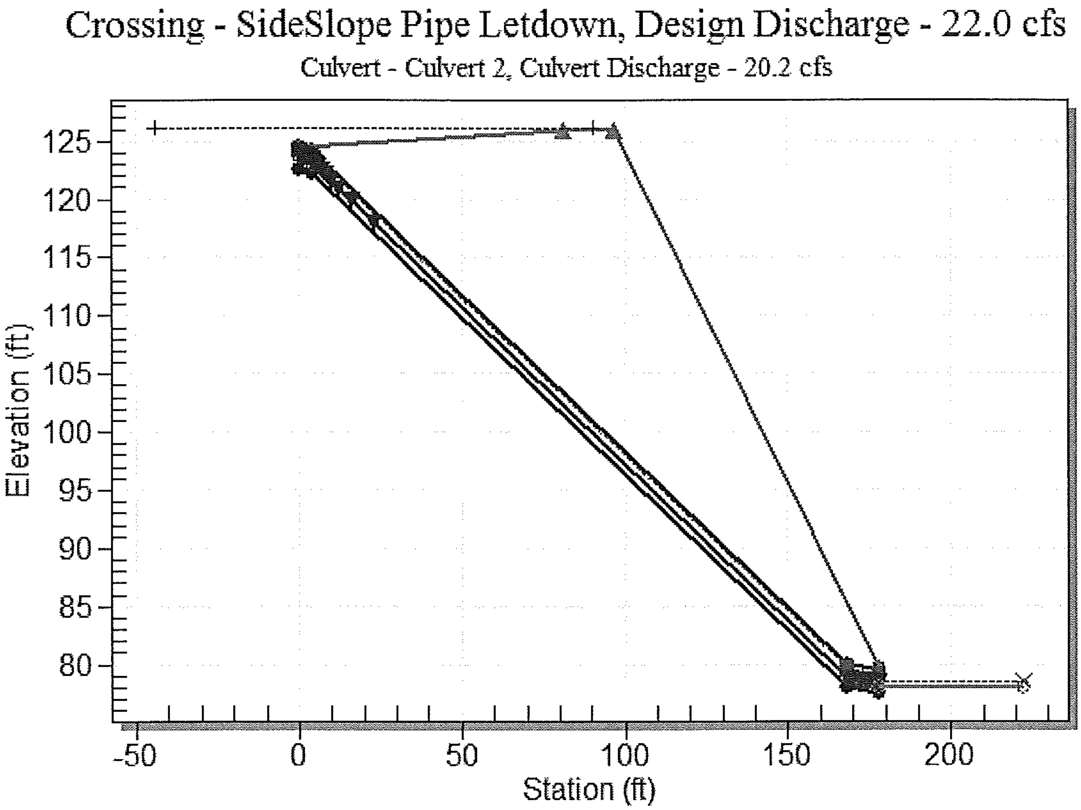
Runout Culvert Section Slope: 0.0400

****

Culvert Performance Curve Plot: Culvert 2



Water Surface Profile Plot for Culvert: Culvert 2



Site Data - Culvert 2

Site Data Option: Culvert Invert Data

Inlet Station: 0.00 ft

Inlet Elevation: 122.50 ft

Upper Break Station: 4.00 ft

Upper Break Elevation: 122.20 ft

Lower Break Station: 168.00 ft

Lower Break Elevation: 78.00 ft

Outlet Station: 178.00 ft

Outlet Elevation: 77.60 ft

Number of Barrels: 1

Culvert Data Summary - Culvert 2

Barrel Shape: Circular

Barrel Diameter: 2.00 ft

Upper & Middle Section Material: Corrugated PE

Lower Section Material: Corrugated PE

Embedment: 0.00 in

Upper & Middle Section Manning's n: 0.0240

Lower Section Manning's n: 0.0240
Culvert Type: Double Broken-back
Inlet Configuration: Mitered to Conform to Slope
Inlet Depression: NONE

Table 3 - Downstream Channel Rating Curve (Crossing: SideSlope Pipe Letdown)

Flow (cfs)	Water Surface Elev (ft)	Depth (ft)	Velocity (ft/s)	Shear (psf)	Froude Number
16.00	78.46	0.46	7.15	1.42	2.03
16.80	78.47	0.47	7.26	1.46	2.04
17.60	78.48	0.48	7.37	1.50	2.04
18.40	78.49	0.49	7.47	1.54	2.05
19.20	78.51	0.51	7.57	1.58	2.06
20.00	78.52	0.52	7.67	1.61	2.06
20.80	78.53	0.53	7.77	1.65	2.07
21.60	78.54	0.54	7.86	1.69	2.08
22.00	78.55	0.55	7.91	1.70	2.08
23.20	78.56	0.56	8.04	1.76	2.09
24.00	78.57	0.57	8.13	1.79	2.09

Tailwater Channel Data - SideSlope Pipe Letdown

Tailwater Channel Option: Trapezoidal Channel

Bottom Width: 4.00 ft

Side Slope (H:V): 2.00 (_:1)

Channel Slope: 0.0500

Channel Manning's n: 0.0240

Channel Invert Elevation: 78.00 ft

APPENDIX IIIF-F-3

SEDIMENT CONTROL POND DESIGN

STATE OF TEXAS
NEVZAT TURAN
84059
LICENSED
PROFESSIONAL ENGINEER

12-5-2017

Includes pages IIIF-F-3-1 through IIIF-F-3-7

SEDIMENT CONTROL POND DESIGN

This appendix includes supporting information for the sedimentation pond sizing procedure presented on Figure 4 (refer to Section 2.2 of the Erosion Control Plan for All Phases of Development). In the event that certain percent ground cover that limits the soil loss to 50 tons/acre/year is not achieved and soil loss is temporarily greater than 50 tons/acre/year, a sedimentation pond will be used along with other structural and non-structural BMPs approved as part of this plan to limit the discharge or eroded soil. The sedimentation pond option is a secondary erosion control option, similar to mulch, wood chips, compost, or straw/hay, and will only be used if the 60 percent vegetation specification is not met. If the sedimentation pond option is implemented, the swales and letdowns specified will remain in-place. The sedimentation pond option simply allows for the control of sediment while vegetation is being established. The pond design procedure has been developed for reducing soil loss to less than the allowable amount for external side slopes (i.e., 50 tons/acre/year or less) if 60 percent vegetation coverage is not obtained. The stormwater sedimentation pond design provided is for a 25-year frequency storm event. This provides for a conservative design because the efficiency of the pond will be higher for more frequent storms (e.g., one year frequency). The example calculation included on pages IIIF-F-3-2 through IIIF-F-3-6 demonstrates that a 0.7-acre detention pond is capable of reducing 60 tons/acre/year of soil loss to 50 tons/acre/year or less of soil loss from the external slopes for a 20-acre area. A factor has been calculated that will be used to determine the required pond size for a specified external slope area. For a summary of the efficiencies of ponds for various required soil loss reduction amounts, refer to Figure 4 – Sediment Control Pond Plan as well as the table on page IIIF-F-3-7.

HARDIN COUNTY LANDFILL
0120-758-11-02
SEDIMENT CONTROL POND DESIGN

Required:

Develop a procedure to size a sedimentation pond to reduce sediment discharge from the external embankment area to 50 tons/acre/year or less.

Method:

1. Determine the 25-year frequency peak flow rate upstream of the sediment control pond using the Rational Method.
2. Calculate the settling velocity of sediment particles using Stokes equation.
3. Calculate the fraction of sediment trapped under dynamic conditions.
4. Calculate the fraction of sediment trapped under quiescent conditions.
5. Calculate the total fraction of sediment trapped under combined conditions.
6. Verify that pond design is adequate to reduce given soil loss to 50 tons/acre/year or less.

Reference:

1. State of Texas, Department of Transportation, Bridge Division, Hydraulic Manual, 3rd Edition, December 1985.
2. Chin, David. A. Water-Resources Engineering. Prentice Hall, Inc., 2000.
3. Haan, C.T., et al. Design Hydrology and Sedimentology for Small Catchments, 1994.
4. Cooperative Studies Section, Hydrologic Serices Division. U.S. Department of Commerce. *Technical Paper No. 40.*

Solution:

1. Determine the 25-year frequency peak flow rate upstream of the sediment control pond.

$$Q = CIA$$

Where:

C= 0.8 (runoff coefficient, Ref. 1)
I = intensity (in/hr)
A= upstream drainage area (ac)

Note: A runoff coefficient of 0.8 is used for all areas regardless of slope.

$$I = \frac{b}{(t_c + d)^e}$$

b =	80	From Ref. 1, for Hardin County
d =	7.5	25-year frequency storm event
e =	0.720	

t_c is assumed to be 10 min.

$I =$ 10.19 in/hr

$A =$ 20.0 acres

$Q =$ 163.01 cfs

2. Calculate the settling velocity, V_s (ft/hr), of sediment particles using Stokes equation.

$$V_s = \frac{\alpha (\rho_s/\rho_w - 1) g\phi^2}{18v_w}$$ (Ref. 2)

Where:

α = factor that measures the effect of particle shape (assume spherical, $\alpha = 1$)
ρ_s = density of sediment particle (pcf)
ρ_w = density of ambient water (62.4 pcf)
g = gravity (32.2 ft/s^2)
ϕ = particle diameter (ft)
v_w = kinematic viscosity of the ambient water (ft^2/s)

$\alpha =$ 1
$\rho_s =$ 165 pcf
$v_w =$ 1.08E-05 ft^2/s

Particle Class[1]	Percent in Class	Particle Diameter[2] (ft)	Settling Velocity, V_s (ft/hr)
1	10	1.31E-05	0.17
2	20	1.97E-05	0.38
3	30	2.62E-05	0.68
4	20	3.28E-05	1.06
5	20	3.94E-05	1.52
Total	**100**		

[1] Particle class corresponds to particle diameter.

[2] Particle diameter ranges from 4μm to 12μm, which is typical for clay and silt particles.

3. Calculate the fraction of sediment trapped under dynamic conditions.

a. Determine the overflow rate.

$V_c = Q/A_p$ (EPA Pond Performance Model from Ref. 3)

Where:

V_c = overflow rate
A_p = area of sediment control pond (ac)

Q = 163.01 cfs (from Step 2)
A_p = 0.70 acre

V_c = 19.25 ft/hr

b. Determine the fraction of sediment removed.

$F = 1 - (1 + 1/\beta * V_s/V_c)^{-\beta}$ (Ref. 3)

Where:

F = single-storm trapping of sediment
β = turbulence or short-circuiting parameter reflecting non-ideal performance of pond (assume good performance, $\beta = 3$)

β = 3

$D_R = L_F[(1/CV_Q^2) / (1/CV_Q^2 - \ln (E_m/L_F))]^{(1/CVQ\char`^2)+1}$ (Ref. 3)

Where:

D_R = long-term dynamic removal fraction for stormwater
L_F = removal ratio for very low flow rates
E_m = mean storm removal fraction
CV_Q = coefficient of variation of flows

L_F = 1
E_m = assume equals single-storm trapping, F
CV_Q = 1.46 (from Table 9B.1, p. 570, Ref. 3)

Table 1 - Summary for Dynamic Conditions

Particle Class	Percent in Class	Particle Diameter (ft)	Settling Velocity, V_s (ft/hr)	Single-storm Trapping, F	Fraction Removed Over All Storms, D_R	Fraction Captured Under Dynamic Conditions, E_D[1]
1	10	1.31E-05	0.17	0.009	0.029	0.29
2	20	1.97E-05	0.38	0.019	0.037	0.74
3	30	2.62E-05	0.68	0.034	0.046	1.37
4	20	3.28E-05	1.06	0.053	0.054	1.09
5	20	3.94E-05	1.52	0.075	0.064	1.27
Total	**100**					**4.76**

[1] E_D is the product of percent in class and D_R.

4. Calculate the fraction of sediment trapped under quiescent conditions.

$$RR = \frac{T_{IA} V_s A_Q}{V_R} \qquad \text{(Ref. 3)}$$

$$V_R = RA$$

Where:

RR = removal ratio
T_{IA} = average time interval between storms (hr)
V_s = settling velocity (ft/hr) from Step 2
A_Q = average surface area under quiescent conditions (ft^2)
V_R = mean runoff volume (ft^3)
R = runoff depth for 25-year frequency storm (ft)
A = upstream drainage area (ac)

A_Q = 30,492 ft^2 (assume equal to A_p)
T_{IA} = 99 hrs (from Table 9B.1, p. 570 of Ref. 3)
R = 0.58 ft (Ref. 4)
A = 20.0 ac (from Step 1)

V_R = 505,296 ft^3

Table 2 - Summary for Quiescent Conditions

Particle Class	Percent in Class	Settling Velocity, V_s (ft/hr)	Removal Ratio, RR (ft^3/hr)	Effective Volume Ratio, V_E/V_R[1]	Fraction Removed Under Quiescent Conditions[2]	Fraction Captured Under Quiescent Conditions, E_Q
1	10	0.17	1.01	0.190	0.150	1.50
2	20	0.38	2.27	0.210	0.170	3.40
3	30	0.68	4.04	0.220	0.171	5.13
4	20	1.06	6.30	0.230	0.180	3.60
5	20	1.52	9.08	0.240	0.190	3.80
Total	**100**					**17.43**

[1] Based on Figure 9.29 from Ref. 3, using RR and V_B/V_R.

V_B = reservoir volume = 121,968 ft^3, assuming a 0.7-acre pond with an average depth of 4 feet.

V_B/V_R = 0.241

[2] Based on Figure 9.30 from Ref. 3 with $CV_R = 1.46$.

5. Calculate the total fraction of sediment trapped under combined conditions, E_T.

$$E_T = 1 - (1 - E_D) * (1 - E_Q) \qquad \text{(Ref. 3)}$$

E_T = 21.36 %

Refer to page IIIF-F-3-7 for the total efficiency of ponds for different soil loss reduction amounts.

6. Verify that pond design is adequate to reduce given soil loss to 50 tons/acre/year or less.

   a. Calculate net soil loss (i.e., sediment not captured by pond).

Total Soil Loss = 60.0 tons/ac/yr
E_T = 21.4 % (from Step 5)

Net Soil Loss = Total Soil Loss x (1 - E_T/100)
Net Soil Loss = 47.2 tons/ac/yr

Refer to page IIIF-F-3-7 for the net soil loss for different soil loss reduction amounts.

   b. Calculate the required pond size per unit drainage area factor.

Drainge Area = 20.0 acres (from Step 1)

Pond Area = 0.7 acres (from Step 3)

Required Pond Size / Unit Drainage Area Factor = 0.035

This factor was calculated using a drainage area of 20 acres and a pond area of .7 acre. If a 40-acre drainage area drains to the pond, then a 0.6-acre pond will be required to achieve the above efficiency and net soil loss estimate (40 acres x 0.035 = 1.4 acres). Refer to page IIIF-F-3-7 for the required pond size/unit drainage area factor for different soil loss reduction amounts.

Conclusion:

A 1-acre pond will sufficiently capture enough sediment from a 20-acre drainage area so that no more than 50 tons/acre/year of net soil loss occurs on external embankment slopes. If the size of the drainage area changes, this procedure will need to be updated. Refer to the table on page IIIF-F-3-7 for a summary of the pond efficiencies and net soil loss estimates for different soil loss reduction amounts.

Prep By: JRP
Date: 3-31-17

Chkd By: NT
Date: 3-31-17

HARDIN COUNTY LANDFILL
0120-758-11-02
SEDIMENT CONTROL POND DESIGN

SEDIMENT CONTROL POND SUMMARY

External Slope Area Soil Loss (Tons/Acre/Year)	Percent Efficiency of Pond (Dynamic Conditions)	Percent Efficiency of Pond (Quiescent Conditions)	Total Efficiency of Pond (%)	Net Soil Loss (Tons/Acre/Year)	Pond Area Required Per Unit Drainage Area[1]	50 Tons/Acre/Year or Less?
60	4.8	17.4	21.4	48.3	0.035	YES
70	6.2	25.9	30.5	48.7	0.060	YES
80	7.3	34.5	39.3	48.6	0.085	YES
90	8.4	41.3	46.2	48.4	0.110	YES
100	9.2	46.5	51.4	48.6	0.130	YES
200	15.7	70.9	75.5	49.1	0.300	YES

[1] This factor multiplied by a given drainage area will give the required pond size to achieve the efficiencies shown in the table.

HARDIN COUTY LANDFILL
HARDIN COUNTY, TEXAS
TCEQ PERMIT NO. MSW-2214B

PART III – SITE DEVELOPMENT PLAN
APPENDIX IIIG
GEOLOGY REPORT

Prepared for

BFI Waste Systems of North America, LLC

March 2017
Revised August 2017

Revised December 2017

STATE OF TEXAS
AARON K. EVANS
GEOLOGY
11143
LICENSED
PROFESSIONAL GEOSCIENTIST

12-05-17

Prepared by

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WCG Project No. 0120-758-11-02

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CONTENTS

CONTENTS (CONTINUED)

STATE OF TEXAS
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12-05-17

TABLES

STATE OF TEXAS
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12-05-17

GEOLOGY REPORT CERTIFICATION

Site Information

Site: Hardin County Landfill

Site Location: Hardin County, Texas

MSW Permit No.: 2214B

Qualified Groundwater Scientist Statement

I, Aaron K. Evans, am a Texas-licensed professional geoscientist and a qualified groundwater scientist as defined in §330.3(120). I have prepared the Geology Report which constitutes Appendix IIIG of this permit application. In my professional opinion, the geology report is in compliance with the requirements specified in 30 TAC §§330.63(e). This report has been completed specifically for the Hardin County Landfill. The only warranty made by me in connection with this report is that I have used that degree of care and skill ordinarily exercised under similar conditions by reputable members of my profession, practicing in the same or similar locality. No other warranty, expressed or implied, is intended.

Firm/Address:
Weaver Consultants Group, LLC
6420 Southwest Blvd., Suite 206
Fort Worth, Texas 76109

AARON K. EVANS
GEOLOGY
11143

12-05-17

Signature:

Aaron Evans, P.G., Texas License No. 11143

Date: 12-05-17

1 REGIONAL GEOLOGIC/HYDROGEOLOGIC INFORMATION

1.1 Regional Physiography and Site Topography

According to the USGS 7.5-minute topographic maps of the landfill area (reference Figure IIIG-A.10 in Appendix IIIG-A), the topography in the landfill vicinity generally slopes from south/southwest to north/northeast towards Cypress Creek and Longston Branch. Based on these USGS topographic maps and the facility's site specific topographic map (reference Figure IIIG-B.1 in Appendix IIIG-B), the natural ground surface elevations within the existing permit boundary area range from a high of approximately 82 ft-msl along the northern and western permit boundary to a low of approximately 73 ft-msl along the southern and eastern permit boundary. Locally the ground surface slopes to the east at approximately 30 feet per mile. Due to the relatively flat topography, soil erosion is not expected to pose significant concerns.

According to the Bureau of Economic Geology (BEG, 1996), the landfill is located in the West Gulf Coast Plain regional physiographic province. The provincial topography is relatively flat with gently rolling hills and pine forests. In the immediate vicinity of the landfill, topography drops considerably to the east toward Longston Branch and to the north toward Cypress Creek. Longston Branch, which drains the majority of the site, is an intermittent tributary of Cypress Creek located approximately 1,000 feet east of the permit boundary. Cypress Creek is located west and north of the permit boundary, approximately 1,000 feet from the landfill and drains into Village Creek approximately six miles northeast of the site. Village Creek is a major tributary to the Neches River at their junction approximately 16 miles southeast of the landfill property.

1.2 Regional Geology

1.2.1 Geologic History

During the late Cretaceous period marine deposition dominated with most of Texas covered in shallow seas. Toward the end of the Cretaceous period, this marine deposition largely ceased following a regional uplift to the west. This resulted in a retreat of the seas gulf ward. Subsequent erosion of Cretaceous deposits continued from the early Cenozoic Era to the present. During this time, thick fluviodeltaic progradational sequences of Cretaceous-sourced sediments were deposited seaward under influence of glacio-eustatic sea-level fluctuations that drove erosional downcutting. During the Quaternary period, erosion produced limited areas of Quaternary alluvium and terrace deposits along area stream courses.

Geologic formations that outcrop in the site vicinity are largely Cenozoic aged alluvial sediments and include the Deweyville, Beaumont, Lissie, Willis, Fleming, and Catahoula Formations, as well as Holocene age Alluvium and late Pleistocene Fluviatile Terrace Deposits (BEG, Beaumont Sheet, 1992). Stratigraphic positions of these groups, along with lithologic characteristics and approximate depths to the formations, are presented in Table 1-1 (modified from Young, et al., 2012).

1.2.2 Regional Structural Geology

Figure IIIG-A.2 – Regional Structural Features Map presents the major regional structural geologic features. A wide range of structural features affected the deposition of the Gulf Coast Aquifer system.

During the formation of Pangea in the late Paleozoic era, tectonic collisions uplifted and deformed the southern margin of the Laurasian paleocontinent. With the early Mesozoic breakup of Pangea, continental rifting occurred to the southeast of the Ouachita Fold Belt which created low-lying areas that were subsequently flooded during the Jurassic Period to form the ancestral Gulf of Mexico (Stearn et al., 1979). During this time, relatively restricted intermittent marine flooding formed extensive evaporate deposits in the ancestral Gulf of Mexico. During the Cretaceous, shelf margin carbonate platforms provided a foundation for subsequent Cenozoic terrigenous clastic sedimentation processes. The primary structural features affecting Cenozoic sediment deposition were sediment loading and gravity tectonics. As Cenozoic sediments accumulated, syndepositional normal growth faults formed by gravitational failure due to rapid sediment loading. Cenozoic depositional episodes are marked by growth faults zones running parallel to the gulf coast which represent ancient shelf-margin positions. The underlying Jurassic-aged evaporate deposits provide the salt source layer for subsequent diapiric upwelling through the thick sequence of overlying younger sediments.

Regional structural features include mostly salt domes and related normal growth faults. According to Young, et al. (2012), active faults in the Gulf Coast Aquifer system typically display a mappable surface expression that locally coincides with differential subsidence zone boundaries. Maximum fault displacements in the Gulf Coast Aquifer system are generally a few hundred feet with only a few feet of surface displacement. Unstable areas and fault investigations are discussed in detail in the location restriction demonstration located in Appendix I/IIC of Parts I/II.

1.2.3 Regional Stratigraphy

Regional stratigraphy consists of geologic units of the Gulf Coastal Plain. Geologic formations that outcrop in the site vicinity are largely Cenozoic aged alluvial sediments and include the Deweyville, Beaumont, Lissie, Willis, Fleming, and Catahoula Formations, as well as Holocene age Alluvium and late Pleistocene Fluviatile Terrace Deposits (BEG, Beaumont Sheet, 1992). Stratigraphic positions of these groups, along with lithologic characteristics and approximate depths to the formations, are presented in Table 1-1 (modified from Young et al., 2012).

According to the Texas Bureau of Economic Geology Geologic Atlas of Texas (1992), the site is located upon the Quaternary Lissie Formation as shown on the Figure IIIG-A.1 – Regional Geologic Map, and Figure IIIG-A.3 – Regional Geologic Cross Section Map. As indicated on Figure IIIG-A.3, the Cenozoic strata form a southward-thickening wedge extending toward the Gulf of Mexico. Outcrops of Cenozoic geologic formations generally strike parallel to the gulf coastline in a monoclinal structure that dips gulfward at rates ranging from 5 to 20 feet per mile (Young, et al., 2012). The Lissie Formation is described as a sequence of interbedded fluviatile channel sands separated by interchannel muds consisting of clay, silt, sand, and minor quantities of gravel. The Lissie formation is approximately 400 feet thick beneath the facility and overlies the Willis Formation.

The Pliocene-age Willis Formation, and Miocene-age Goliad Sand Formation, Fleming Formation, and Catahoula Formation constitute the Tertiary aged stratigraphic units that underlay the Lissie Formation. The approximate depths and thicknesses of these units and their predominant lithologic characteristics are summarized in Table 1-1.

Table 1-1
Regional Stratigraphy in the Vicinity of the Hardin County Landfill

System	Series	Stratigraphic Unit	Hydrogeologic Units	Approximate Depth to Top of Formation and (Formation Thickness) at the Site in Feet	Lithologic Characteristics
Quaternary	Holocene	Alluvium	Chicot Aquifer	Not present beneath site Outcrops 3 miles northeast of site (0-50 ft. thick regionally)	Clay, silt, sand, and gravel
	Pleistocene	Deweyville Formation		Not present beneath site Outcrops 7 miles east of site (0-100 ft. thick regionally)	Sand, silt, clay , and gravel
		Fluviatile Terrace Deposits (Undivided)		Not present beneath site Outcrops 4 miles northeast of site (0-50 ft. thick regionally)	Gravel, sand, and silt
		Beaumont Formation		Not present at the site Outcrops 6 miles south of site (0-800 ft. thick regionally)	Clay, silt, and sand
		Lissie Formation		At surface (400 ft.)	Clay, silt, sand, and gravel
Tertiary	Pliocene	Willis Formation		400 ft. (400 ft.)	Clay, silt, sand, and gravel
	Miocene	Goliad Sand	Evangeline Aquifer	800 ft. (1,400 ft.)	Sand and gravel
		Fleming/Lagarto Formation	Burkville Confining System	2,200 ft. (400 ft.)	Clay, silt, and sand
		Fleming/Oakville Formation	Jasper Aquifer	2,600 ft. (1,400 ft.)	Sand, silt, and clay
		Catahoula Formation	Catahoula Confining System	0-500 ft. thick regionally	Mudstone, sand, and tuff

Modified from Young, et al., 2012.

1.3 Geologic Processes

1.3.1 Fault and Seismic Data

Seismic impact zone and fault investigations are discussed in the location restrictions in Parts I/II. As discussed in these sections, no active faults or seismic impact zones are located within one mile of the site. The geologic processes that could potentially cause unstable areas are discussed in detail in the unstable areas location restriction demonstration located in Appendix I/IIC of Parts I/II.

1.3.2 Erosional Processes

Erosional processes in the landfill area are limited to those produced by the Longston Branch and Cypress Creek drainage systems located to the East and North of the landfill unit; respectively. These include rill and channel erosion and sheet flow. Natural topographic relief across the site is low and erosion from these processes is minimal. No adverse effects on the site are anticipated. No mass wasting was observed.

1.3.3 Wetlands Identification

Details regarding jurisdictional wetland areas are provided in the location restriction demonstrations in Appendix I/IIC of Parts I/II.

1.4 Regional Aquifers

Regional aquifers beneath the landfill include the Chicot, Evangeline, and Jasper hydrogeologic units of the larger Gulf Coast Aquifer system. The Gulf Coast Aquifer system is classified by the Texas Water Development Board (TWDB) (Ashworth and Hopkins, 1995) as a major Texas aquifer. The Gulf Coast Aquifer is comprised of, from youngest to oldest, five hydrogeologic units: the Chicot Aquifer, the Evangeline Aquifer, the Burkville Confining System, the Jasper Aquifer, and the Catahoula confining system (Baker, 1979). These hydrogeologic units consist of complex interbedded clays, silts, sands, and gravels which are regionally hydraulically connected and form a large leaky artesian aquifer system. Regional Chicot, Evangeline, and Jasper Aquifer potentiometric surface contour maps were reproduced from USGS (Kasmarek, 2013) and are presented on Figures IIIG-A.4 through IIIG-A.6 in Appendix IIIG-A. Gauged groundwater elevation data from the landfills' previous four semiannual groundwater monitoring events were used to construct site-specific groundwater contour maps illustrated in Figures IIIG-D.1 through IIIG-D.4 in Appendix IIIG-D. These regional and site specific groundwater contour figures indicate an approximate correlation between the regional Chicot and Evangeline Aquifer potentiometric surfaces in the landfill area and groundwater elevations gauged in landfill groundwater monitor wells screened in the uppermost aquifer. It is noted that delineation of the boundary between the Chicot and Evangeline is based on chrono-stratigraphic rather than litho-stratigraphic characteristics and the regional data indicate some degree of hydraulic interconnectivity between the aquifers (Baker, 1986).

Based on water well water level and simulation data obtained from the USGS (Kasmarek, 2013), the potentiometric surface of the Chicot and Evangeline Aquifers are heavily influenced by historical and ongoing groundwater pumping around the Houston area. However, the groundwater pumping and subsequent water table drop in the Houston area appears to have had nominal influence on groundwater elevations in Hardin County and the vicinity of the landfill property.

Aquifer hydraulic properties and water quality data are summarized in Table 2-1. These data were reproduced from the facility's approved Site Development Plan (MSW Permit No. 2214A, Attachment 4 – Geology Report)(Hydrex, 2001) and were originally sourced from TWDB groundwater resource reports for Hardin, Chambers, Jefferson, Polk, and San Jacinto Counties. The site specific uppermost aquifer is further discussed in Sections 2 and 3.

1.4.1 Chicot Aquifer

The Chicot Aquifer is described as an unconfined to leaky-confined hydrogeologic unit comprised of clays, silts, sands and gravels and ranging in thickness from less than 100 feet at its northernmost extent to over 3,000 feet in the southeast down-dip areas of the Gulf of Mexico (Young, et al., 2012). Stratigraphic units which comprise the Chicot Aquifer include, from youngest to oldest, the Holocene-age Alluvium and Deweyville Formation, and the Pleistocene-age Fluviatile Terrace deposits (undivided), Beaumont Formation, Lissie Formation, and the Willis Formation (Baker, 1979). The landfill unit is founded in the Lissie Formation of the Chicot Aquifer which outcrops throughout most of Hardin County. The primary source of recharge to the aquifer is precipitation infiltration directly on the outcrop and through alluvial deposits overlying the outcrop (Baker, 1964). As illustrated in Figures IIIG-A.4 , the regional Chicot Aquifer groundwater flow direction follows the regional dip of its formation constituents to the south.

The Chicot Aquifer hydraulic properties and water quality data are summarized in Table 2-1. The average rate of groundwater movement in the Chicot Aquifer was calculated to be about 178.9 feet per year (Hydrex, 2001). The Chicot Aquifer produces fresh, good quality water from regional water wells with TDS concentrations generally below 100 mg/L.

1.4.2 Evangeline Aquifer

The Evangeline Aquifer is described as an unconfined hydrogeologic unit comprised of mostly sands and gravels and ranging in thickness from less than 100 feet at its northernmost extent to over 6,000 feet in the southeast down-dip areas of the Gulf of Mexico (Young, et al., 2012). The Miocene-age Goliad Sand lithologic unit comprises the Evangeline Aquifer (Baker, 1979). The depth to the top of the aquifer is approximately 800 feet below the landfill unit. The primary source of recharge to the aquifer is precipitation infiltration directly on the outcrop, through alluvial deposits overlying the outcrop, and through the overlying Chicot Aquifer sediments to a lesser degree (Baker, 1964). As illustrated in Figure IIIG-A.5, the regional Evangeline Aquifer groundwater flow direction follows the regional dip of its formation constituents to the south. The Evangeline Aquifer hydraulic properties and water quality data are summarized in Table 2-1. The average rate of groundwater movement in the Evangeline Aquifer was calculated to be about 62.05 feet per year (Hydrex, 2001). The Evangeline Aquifer screened regional water wells groundwater data indicate TDS concentrations generally below 3,000 mg/L (Baker, 1979).

1.4.3 Jasper Aquifer

The Jasper Aquifer is described as a confined hydrogeologic unit comprised of mostly sands, silts, and clays and ranging in thickness from less than 100 feet at its northernmost extent to over 6,000 feet in the southeast down-dip areas of the Gulf of Mexico (Young, et al., 2012). The Miocene-age lower Fleming and Oakville Formations comprise the Jasper Aquifer which are separated from the overlying Goliad Sand and Evangeline Aquifer by the Burkville confining system (Baker, 1979). The depth to the top of the aquifer is approximately 2,600 feet below the landfill unit. The primary source of recharge to the aquifer is precipitation infiltration directly on the outcrop, through alluvial deposits overlying the outcrop, and through the overlying Evangeline Aquifer sediments to a lesser degree. (Baker, 1964). As illustrated in Figure IIIG-A.6, the regional Evangeline Aquifer groundwater flow direction follows the regional dip of its formation constituents to the south. The Jasper Aquifer hydraulic properties and water quality data are summarized in Table 2-1. The average rate of groundwater movement in the Jasper Aquifer was calculated to be about 98.55 feet per year (Hydrex, 2001). The Evangeline Aquifer screened regional water wells groundwater data indicate TDS concentrations generally below 3,000 mg/L (Baker, 1979).

Table 1-2
Regional Hydraulic Properties and Water Quality Parameters of the Chicot, Evangeline, and Jasper Aquifers

Properties	Chicot Aquifer	Evangeline Aquifer	Jasper Aquifer
Composition	Clay, Silt, Sand, and Gravel	Sand and Gravel	Sand, Silt, and Clay
Transmissivity	Range 1,900 – 14,800 gal/day/ft	Range 3,000-14,800 gal/day/ft	Range 10,800-29,800 gal/day/ft
Hydraulic Conductivity	Range 9-11x10^{-3} cm/s	Range 3-8x10^{-3} cm/s	Range 8-28x10^{-3} cm/s
Flow Rate	178.9 ft/year	62.05 ft/year	98.55 ft/year
Potentiometric Surface	See Figure IIIG-A.4	See Figure IIIG-A.5	See Figure IIIG-A.6
Elevation & Thickness	See Figure IIIG-A.7	See Figure IIIG-A.8	See Figure IIIG-A.9
Present Water Use	Public supply, industrial, irrigation, domestic, livestock	Public supply, industrial, irrigation, domestic, livestock	Public supply, industrial, irrigation, domestic, livestock
Total Dissolved Solids	<100 mg/L	<3,000 mg/L	<3,000 mg/L

Composition data obtained from Baker (1979) and Barnes (1992).

Hydraulic Properties modified from Hydrex (2001).

1.5 Water Well Search

A search to identify Texas registered water wells within a one-mile radius of the landfill permit boundary included a water well search performed by Geosearch on October 31, 2016 for records and maps on file in the USGS National Water Information System (NWIS), Texas Submitted Drillers Report Database (SSDRD), Texas Commission on Environmental Quality (TCEQ) database, Texas Water Development Board (TWDB) database, and state Water Utility Database (WUD) records. The provided water well information is shown in Table 1-3. The well locations are shown on the Figure IIIG-A.10 – Water Well Location Map. Corresponding individual water well reports are provided in Appendix IIIG-A on pages IIIG-A.12 through IIIG-A.69. The water well locations listed on Figure IIIG-A.10 were obtained from GeoSearch and modified based on review of water well report descriptions, Google Earth aerial map images, and site reconnaissance performed by WCG in January 2017. A total of 44 water wells were identified by 33 SSDRD reports, five TCEQ reports, and six TWDW reports. No water wells were identified in NWIS or WUD records for the referenced search area. Of the identified water wells, 39 are completed in the Chicot Aquifer at less than 400 feet in total depth and one well is completed in the Evangeline Aquifer at a depth of 485 feet.

Water well 232600 is the only water well identified within the landfill permit boundary and provided potable water to the landfill prior to being plugged in December 2016. At this time the landfill utilizes water piped from nearby water well 211340 located on the adjacent property to the east which is also owned by BFI. Water well 211340 is the only water well identified within 500 feet of the landfill permit boundary. The reported coordinates of one identified water well (232601) are identical to those of former landfill water well 232600. However, there is no other onsite water well and the 232601 water well report describes the well's physical location as "Hwy 326". Texas State Highway 326 is located approximately 1,000 feet east of the landfill permit boundary at its nearest extent. Consistent with the water well's report location description, water well 232601 was moved to the east adjacent highway 326 on Figure IIIG-A.10; although the well's exact location is unknown.

It is noted that limited information is available for four of the identified water wells which were completed to depths of over 8,000 feet (61-46-104, 61-46-105, 61-46-106, and 61-46-107). The completion depths of these wells coincide with those of nearby oil and gas wells completed in the Eocene-age Yegua and Cook Mountain Formations. These four water wells have been included in Table 3-1 but are interpreted to be misclassified oil and/or gas wells.

In January 2017, WCG completed a water well reconnaissance from area roadways. The purpose of the reconnaissance was to identify potential unregistered water wells within a one mile radius of the landfill permit boundary. WCG also searched for the presence of springs and faulting within the immediate accessible area within a one mile radius of the permit boundary. This visual survey was limited by viewing obstructions including vegetation and structures, and a common private property access restriction.

Indications of potential water wells used for the reconnaissance included elevated water tanks, wellhead equipment, pressure balance tanks, small outlying structures having electrical power drops, and windmills. Based on field observations conducted by WCG, no unregistered water wells, springs, or surface expressions of faulting were identified. As part of WCG's water well reconnaissance, the municipal water provider nearest to the landfill property was interviewed. The City of Kountze, Texas municipal water department provides water within the jurisdictional Kountze city limits and to select businesses and residents within the immediate area outside of the city limits. According to City of Kountze water department, there is no water utility provider for those properties located within a mile of the landfill property because quality potable groundwater is readily accessible via private water well.

WCG noted no flowing springs during the site and area reconnaissance. According to Brune (2002), there are no large springs within Hardin County due to the lack of significant topographic relief. Several small springs are noted throughout the county including one located approximately 1.5 miles northeast of the landfill property near the Old Hardin Cemetery.

Table 1-3
Registered Water Wells Within One Miles of the Landfill[1]

Well No.	Total Depth (ft)	Aquifer[2]	Use
232601	245	Chicot	Domestic
232600	225	Chicot	Industrial
211340	160	Chicot	Industrial
211311	160	Chicot	Domestic
284555	257	Chicot	Rig Supply
155687	360	Chicot	Domestic
127906	374	Chicot	Domestic
375993	160	Chicot	Domestic
211350	485	Evangeline	Domestic
228904	350	Chicot	Domestic
242617	200	Chicot	Domestic
211303	160	Chicot	Domestic
228798	220	Chicot	Domestic
228812	160	Chicot	Domestic
232598	120	Chicot	Domestic
228887	257	Chicot	Domestic
392245	360	Chicot	Domestic
61-46-103	127	Chicot	Domestic
61-46-102	227	Chicot	Domestic
79360	180	Chicot	Domestic
149219	180	Chicot	Domestic
330449	392	Chicot	Domestic
131319	180	Chicot	Domestic
308946	500	Chicot	Domestic
232597	120	Chicot	Domestic
197065	440	Chicot	Domestic
232599	130	Chicot	Industrial
213620	240	Chicot	Domestic
338046	190	Chicot	Domestic
246137	140	Chicot	Domestic
335190	160	Chicot	Domestic
128253	250	Chicot	Domestic
228891	185	Chicot	Rig Supply
211390	150	Chicot	Domestic
252466	250	Chicot	Domestic
133825	130	Chicot	Irrigation
335292	160	Chicot	Domestic

Notes: [1] Water well number, depth and use information obtained from SSDRD, TCEQ, and TWDB records.
[2] Water well aquifer designation was estimated based on the total depth of the well.
[3] Water well location and total depth corresponds to that of nearby oil and gas wells.

Table 1-3
Registered Water Wells Within One Miles of the Landfill[1](Continued)

Well No.	Total Depth (ft)	Aquifer[2]	Use
61-46-105[3]	8400	Yegua/Cook Mountain	Unknown
61-46-104[3]	8518	Yegua/Cook Mountain	Unknown
149212	350	Chicot	Domestic
42480	200	Chicot	Domestic
131320	383	Chicot	Domestic
61-46-106[3]	8616	Yegua/Cook Mountain	Unknown
61-46-107[3]	8506	Yegua/Cook Mountain	Unknown

Notes: [1] Water well number, depth and use information obtained from SSDRD, TCEQ, and TWDB records.
[2] Water well aquifer designation was estimated based on the total depth of the well.
[3] Water well location and total depth corresponds to that of nearby oil and gas wells.

1.6 Petroleum Well Search

On November 8, 2016 WCG conducted an online review of the Texas Railroad Commission (TRC) geographic information system petroleum well database (http://wwwgisp.rrc.texas.gov/GISViewer2/) to identify oil and natural gas wells in the landfill vicinity. All active and plugged oil and gas wells identified by the TRC GIS database are shown on Figure IIIG-A.11. The TRC database indicated 15 total oil and gas wells within one mile radius of the landfill permit boundary. Nine active oil wells, one active gas well, four active combined oil/gas wells, and one plugged gas well were identified. No oil or gas wells were identified within the landfill permit boundary. As indicated on Figure IIIG-A.11, all but one of these wells is located north of the landfill and one well is located to the south of the landfill. Eight dry wells were also identified by the TRC search and their locations have also been plotted on Figure IIIG-A.11. According to the TRC well reports, all the oil and gas wells within the one-mile radius of the landfill property are drilled to depths between 8,300 and 8,600 feet within the Nona Mills Field. According to the American Association of Petroleum Geologists (AAPG), oil and gas wells in the Nona Mills Field terminate in the oil rich sands of the Eocene-age Yegua and Cook Mountain Formations. The Nona Mills field coincides with a strike-faulted anticline trending southwest to northeast (Penn, 1962). Fault investigations are discussed in detail in the location restriction demonstration in Appendix I/IIC of Parts I/II. Nona Mills Field oil and gas production began in 1950.

2 SUBSURFACE INVESTIGATION REPORT

2.1 Site Stratigraphy

2.1.1 Existing Borehole Data

The subsurface characterization of the existing 79-acre permit boundary area is supported by 45 soil borings. These include 17 borings by Southwestern Laboratories, Inc. (SWL) (1990 and 1991), 21 borings by Hydrex Environmental Inc. (Hydrex)(1998, 2005, and 2010), and six borings completed by WCG (January 2017). The site-specific lithology has previously been characterized by SWL in 1994 (Permit No. MSW-2214) and by Hydrex in 2001 (Permit No. MSW-2214A).

2.1.1.1 Site-Wide Subsurface Characterization

The permitted site-wide subsurface characterization describes six site-specific stratigraphic units. These units include from shallowest to deepest; a surficial silt, silty clay, clay (upper), clayey sand, clay (lower), and silty sand. To illustrate site-wide subsurface conditions, two geologic cross sections were produced by Hydrex in 2001 (Permit No. MSW-2214A, Figures A-8 and A-9). These two site-wide cross section figures are included as pages IIIG-C-7 and IIIG-C-8 in appendix IIIG-C. The previous subsurface characterizations provided adequate lithologic information and suitable subsurface description to satisfactorily define the permitted 49.6-acre Type I limits of waste footprint and 79-acre landfill property area, and obtain TCEQ approval of Permit No.s MSW-2214 and MSW-2214A.

2.1.1.2 Southern Expansion Area Subsurface Characterization

Since approval of Permit Number MSW-2214A, 15 additional onsite borings have been completed. These include nine borings by Hydrex (2005 and 2010) and six borings by WCG (January 2017). The lithologic, hydrogeologic, and geotechnical data from these additional borings were integrated into the facility's site-specific geologic framework for comprehensive review pursuant to the proposed 16.6-acre expansion area within the undeveloped southern waste footprint for permit application MSW-2214B. The former site-specific geologic units which were encountered during previous subsurface investigations have been incorporated into five site-specific geologic strata. These five strata include the Upper Clay Stratum, Upper Sand Stratum, Lower Clay Stratum, and Lower Sand Stratum. An additional new clay unit was encountered beneath the Lower Sand Stratum in five of the 2017 WCG geotechnical borings which had not been encountered in previous subsurface investigations. This new clay unit has been designated the Basal Clay Stratum. Four WCG geologic cross sections were constructed using stratigraphic and lithologic data from the borehole logs in Appendix IIIG-B to illustrate subsurface conditions in the proposed 16.6-acre southern vertical expansion area. These cross sections are presented in Appendix IIIG-C as Figures IIIG-C.2 through IIIG-C.5. A geologic cross section index map is included as Figures IIIG-C.1. The geologic cross sections illustrate the hydrostratigraphic and lithologic subdivision of the five site-specific strata.

2.1.2 Upper Clay Stratum

The uppermost site-specific stratigraphic unit is the Upper Clay Stratum. This unit is continuous beneath the landfill and characterized as predominantly dry to moist, low permeability clay and silty clay with minor amounts of sand. This uppermost unit also includes a minor subunit of discontinuous surficial sand and silt. The Upper Clay Stratum ranges in thickness from 8 to 39 feet site-wide. Discontinuous thin silt and sand filled partings and seams are common in this strata. Slickensides are noted within the Upper Clay Stratum at depths ranging from 12 to 20 feet. These slickensides are described in the lithologic descriptions of nine SWL borings (B-5A, P-2A, P-20, P-21, P-22, P-23, P-25, P-26, and P-27) and two WCG borings (WC-2 and WC-3). The slickensides appear as a single parted surface in geotechnical borings WC-2 and WC-3.

SWL screened one piezometer (P-2A) exclusively within the Upper Clay Stratum. SWL conducted a rising head slug test on P-2A in 1991. The SWL-computed hydraulic conductivity value from the slug test data was $8.2\text{x}10^{-5}$ cm/s as indicated in Table 3-2.

2.1.3 Upper Sand Stratum

Beneath the Upper Clay Stratum lies the Upper Sand Stratum. The Upper Sand Stratum constitutes the landfill's permitted groundwater monitoring zone (uppermost aquifer). This stratum is continuous beneath the landfill. The Upper Sand Stratum is characterized as a saturated sandy silt, silty sand, clayey sand, or sandy clay with a range in thickness from 1 foot to 30 feet site-wide. The Upper Sand sediments are generally much finer in the western borings often occurring as a sandy clay or as zones of thinly interbedded seams of wet sands and silts bound by moist silty clay. The Upper Sand Stratum sediments become coarser toward the east with more abrupt and well defined transitions between the Upper Sand and bounding Upper Clay and Lower Clay strata.

All facility groundwater monitoring wells are screened within the Upper Sand Stratum. As illustrated in groundwater contour map Figures IIIG-D.1 through IIIG-D.4, the groundwater within the uppermost aquifer flows from the west and the southwest toward the east or northeast. Rising head slug tests were conducted in seven Upper Sand Stratum screened wells. In 1991, SWL conducted rising head slug tests in six groundwater piezometers (P-1A, P-4A, P-21, P-22, P-24, and P-26) with calculated hydraulic conductivity values ranging from $7.52\text{x}10^{-6}$ cm/s to $3.00\text{x}10^{-4}$

cm/s. In January 2017, WCG conducted a rising head slug test in groundwater detection monitor well MW-7 with a calculated hydraulic conductivity of $3.03\text{x}10^{-4}$ cm/s. Hydraulic conductivity values are presented in Table 3-2.

2.1.4 Lower Clay Stratum

Beneath the Upper Sand Stratum lies the Lower Clay Stratum. The Lower Clay Stratum is the lower confining unit (aquiclude) to the Upper Sand Stratum (uppermost aquifer). The Lower Clay Stratum is characterized as predominantly dry to moist silty clay with minor amounts of sand present in matrix and interbedded horizontally in thin partings and laminations. The Lower Clay Stratum is continuous beneath the site. Five SWL borings (P-3A, P-22, P-23, P-24, and P-25) and the six WCG borings (WC-1, WC-2, WC-3, WC-4, WC-5, and WCP-5) penetrated the vertical extent of the Lower Clay Stratum. The lithologic details for these borings indicate the Lower Clay Stratum ranges in thickness from 9 to 26 feet. No wells or piezometers have been screened exclusively within the Lower Clay Stratum. The laboratory data indicate a lower Clay Stratum vertical hydraulic conductivity of $1.1\text{x}10^{-8}$ cm/s. The laboratory data is presented in Appendix IIIE – Geotechnical Report.

2.1.5 Lower Sand Stratum

The Lower Sand Stratum lies beneath the Lower Clay Stratum aquiclude. Three of the SWL borings (P-23, P-24, and P-25) encountered the top of the Lower Sand Stratum. All three of these borings are located along the eastern landfill permit boundary. All six of the deep borings completed by WCG in 2017 (WC-1, WC-2, WC-3, WC-4, WC-5, and WCP-5) penetrated the Lower Sand Stratum. Lithologic data from these borings indicate an abrupt increase in Lower Sand Stratum thickness and sediment coarseness within the eastern quarter of the southern undeveloped waste footprint in the cross section figures included in Appendix IIIG-C. The Lower Sand Stratum is characterized as a thick sequence of loose to unconsolidated saturated silty sand with a thickness of 44 to 46 feet on the east end of the undeveloped waste footprint. West of boring WC-3, deep borings WC-1, WC-2, and WC-3 encounter the Lower Sand Stratum as a saturated sandy silt with minor amounts of clay and a thickness of 2.0 to 3.5 feet. The characteristics and occurrence of the Lower Sand Stratum suggest the unit to be a channel sand body on the east end of the undeveloped waste footprint. This channel sand corresponds with present day topographic relief and the trend of Longston Branch east of the permit boundary.

As part of the 2017 site characterization by WCG, piezometer WCP-5 was installed directly adjacent to the existing Upper Sand Stratum screened groundwater monitor well MW-7. WCP-5 was screened through the full vertical extent of the Lower Sand Stratum. WCG gauged the static water levels in MW-7 and WCP-5 over the course of five days. The data obtained revealed a difference of approximately five feet in the separation of static hydraulic head between the Upper Sand Stratum and Lower Sand Stratum screened wells. This significant static head separation demonstrates a lack of hydraulically interconnectivity between the two saturated sand strata. Laboratory vertical hydraulic conductivity test data indicate very low values ($1.1\text{x}10^{-8}$ cm/s) for the Lower Clay Stratum, which separates the Upper Sand Stratum and Lower Sand Stratum. The static hydraulic head elevations gauged in WCP-5 indicate that the Lower Sand Stratum groundwater is under confined conditions with a static hydraulic head within the Lower Clay Stratum interval at 11 feet above the top of the Lower Sand Stratum. The laboratory data is presented in Appendix IIIE – Geotechnical Report.

2.1.6 Basal Clay Stratum

Beneath the Lower Sand Stratum lies the Basal Clay Stratum. The Basal Clay Stratum is characterized as a moist silty clay of low permeability. Five of the deep geotechnical borings completed by WCG in 2017 (WC-1, WC-2, WC-4, WC-5, and WCP-5) encountered the Basal Clay Stratum. The Basal Clay Stratum is the lower confining unit to the saturated Lower Sand Stratum. The laboratory test data for the Basal Clay Stratum indicate a vertical hydraulic conductivity of $2.8\text{x}10^{-8}$ cm/s.

2.2 Soil Boring Plan

The subsurface characterization for the current permit (TCEQ Permit No. MSW-2214A) utilized 39 soil borings to characterize the existing 49.6-acre permitted Type I waste footprint and 79-acre permit boundary area. The approved borehole data set for Permit No. MSW-2214A is based on an EDE of 45.23 ft-msl.

On October 5, 2016, the facility submitted a Soil Boring Plan to the TCEQ to facilitate the proposed TCEQ Permit No. MSW-2214B landfill expansion. The Soil Boring Plan utilized the permitted subsurface characterization (Permit No. MSW-2214A) supported by the facility's 39 existing soil borings to characterize subsurface conditions beneath the facility. In a Soil Boring Plan response letter (dated November 10, 2016), TCEQ requested that the 16.6-acre undeveloped southern vertical expansion area be characterized with a minimum of six geotechnical borings, five of which must be completed to a depth of at least 30 feet below the EDE. Excerpts from the 2016 Soil Boring Plan are presented in Appendix IIIG-E. The TCEQ Soil Boring Plan approval letter (dated December 27, 2016) is also presented in Appendix IIIG-E.

From January 3 through January 14, 2017, WCG completed site exploration by advancing six additional boreholes within the proposed 16.6-acre southern vertical expansion area at the locations shown on Figure IIIG-B.1. Geotechnical borings WC-1 through WC-5 were continuously sampled using hollow stem auger, Shelby tubes, or split spoon techniques. All recovered subsurface material samples were retained for laboratory testing. Piezometer boring WCP-5 was continuously sampled using hollow stem auger techniques. A 2-inch groundwater piezometer was installed in the borehole and screened throughout the Lower Sand Stratum. The logs of these boreholes are presented as Figures IIIG-B.84 through IIIG-B.98 in Appendix IIIG-B. As indicated on the boring logs, all six WCG borings were drilled deeper than 30 feet below the permitted EDE of 45.23 ft-msl. The subsurface samples obtained from the geotechnical borings indicated the subsurface conditions beneath the 16.6-acre undeveloped southern expansion area to be consistent with the conditions reported in previous site explorations. All soil borings completed by WCG were installed, abandoned, and plugged in accordance with the applicable rules in Title 16 TAC Chapter 76 (water well drillers and water well pump installers' rules, administered by the Texas Department of Licensing and Regulation). No unplugged borings were observed by WCG during the 2017 drilling activities.

2.3 Site Exploration Summary

Subsurface characterization of the site has been performed during six events at the landfill. Various geotechnical and geological subsurface explorations, and piezometer, monitor well, and landfill gas probe installations were completed by SWL (1990 and 1991), Hydrex (1998, 2005, and 2015), and WCG (January 2017). These investigations included a total of 45 exploratory borings across the site whose locations are shown on Figure IIIG-B.1. The site-wide geologic cross sections created by Hydrex for the approved 79-acre permit boundary and 49.6-acre Type I waste footprint are included as Figures IIIG-C-7 and IIIG-C-8 in Appendix IIIG-C. Geologic cross sections produced for the 16.6-acre undeveloped southern vertical expansion area are presented as Figures IIIG-C.2 through IIIG-C.5 in Appendix IIIG-C. These cross section locations are provided in Figure IIIG-C.1. The borehole specifications are summarized in Table 2-1 and in the following text:

- A 1990 subsurface characterization by SWL included at least nine geotechnical borings that were advanced to evaluate subsurface conditions for the proposed landfill facility. Attachment 4 of the facility's approved Site Development Plan (Permit No. MSW-2214A)(Hydrex, 2001) states that 10 borings within the 79-acre permit boundary (B-1, 2, 3, 4, 5, 6, 8, 9, and 12) and an additional unspecified number of off-site borings were drilled in May, 1990. With the exception of boring B-9, no lithologic information is available for these initial investigatory boreholes and they have not been included in the summation of existing facility exploration borings. Per the previously approved SDP for Permit No. MSW-2214, the Texas Department of Health (TDH) required SWL to re-drill the ten borings referenced above to greater depths and advance additional borings to characterize the site. In accordance with TDH's request, SWL redrilled several borings (within five feet of the former borehole locations) and drilled additional borings from December 1990 to January 1991.
- A 1990-1991 subsurface characterization continuation by SWL included an additional 16 geotechnical borings (P-1A, P-2A, P-3A, P-4A, B-5A, B-6A, B-8A, B-12A, and P-20 through P-27) that were drilled to further evaluate subsurface conditions for the 79-acre permit boundary. Twelve of these borings were completed as piezometers. These piezometers are indicated by a I.D. designation starting with "P". All of the 1990-1991 boreholes fully

penetrated the Upper Sand Stratum uppermost aquifer. All but five of the borings (B-5A, B-6A, B-8A, B-12A, and P-2A) encountered the underlying Lower Clay Stratum. Three of the borings/piezometers (P-23, P-24, and P-25) were advanced into the Lower Sand Stratum.

- A 1998 subsurface characterization by Hydrex included the advancement of 13 borings to depths ranging from 20 to 36 feet for the installation of six groundwater monitor wells (MW-1 through MW-6) and seven LFG monitoring probes (GMP-1 through GMP-7). Each of these borings/wells penetrated and were screened within the Upper Sand Stratum (uppermost aquifer). With the exception of four gas probe borings (GMP-1, GMP-2, GMP-4, and GMP-7), all 1998 borings encountered the top of the underlying Lower Clay Stratum.
- A 2005 subsurface characterization by Hydrex included the advancement of six borings to depths ranging from 34 to 40 feet for the installation of six groundwater monitor wells (MW-7 through MW-12). Each of these borings penetrated and were screened within the Upper Sand Stratum (uppermost aquifer) and encountered the underlying Lower Clay Stratum.
- A 2010 subsurface characterization by Hydrex included the advancement of three borings to depths ranging from 26 to 30.5 feet for the installation of three groundwater monitor wells (MW-5R, MW-6R, and MW-13). Each of these borings/wells penetrated and were screened within the Upper Sand Stratum uppermost aquifer and encountered the top of the underlying Lower Clay Stratum.
- A January 2017 subsurface characterization by WCG included the advancement of six geotechnical borings (WC-1, WC-2, WC-3, WC-4, WC-5, and WCP-5) that were drilled to depths ranging from 63 to 98 feet to further evaluate subsurface conditions for the 16.6-acre undeveloped southern vertical expansion area. Each of these borings penetrated the Upper Sand Stratum (uppermost aquifer), Lower Clay Stratum (aquiclude), and Lower Sand Stratum. Five of the borings (WC-1, WC-2, WC-4, WC-5, and WCP-5) encountered the Basal Clay Stratum and one boring (WCP-5) was completed as a groundwater piezometer. Piezometer WCP-5 was installed directly adjacent to existing groundwater monitor well MW-7 and screened throughout the full vertical extent of the Lower Sand Stratum.

Table 2-1
Summary of 45 Existing Borehole Depths and Elevations

Borehole Number	Borehole Surface Elevation (ft-msl)	Northing	Easting	Borehole Total Depth (ft)	Borehole Bottom Elevation (ft-msl)	Depth (Below) or Above EDE[1] (ft)	Top of Lower Clay Stratum Elevation (ft-msl)	Groundwater Elevation at Time of Drilling (ft-msl)	Static Groundwater Elevation (ft-msl)	Date of Static Groundwater Elevation Measurement
1990-1991 Geotechnical Borings by SWL										
B-5A	81.9	10137883.40	4180748.97	30.0	51.9	6.7	N.E.	N.R.	N.R.	N.R.
B-6A	81.4	10138409.76	4181249.12	30.0	51.4	6.2	N.E.	N.R.	N.R.	N.R.
B-8A	76.3	10138318.02	4181766.53	30.0	46.3	1.1	N.E.	N.R.	N.R.	N.R.
B-9	79.2	10137741.69	4181546.87	40.0	39.2	(6.0)	N.E.	N.R.	N.R.	N.R.
B-12A	78.5	10137024.34	4181855.76	30.0	48.5	3.3	N.E.	N.R.	N.R.	N.R.
P-1A	82.7	10138024.47	4180048.64	50.0	32.7	(12.6)	50.7	N.R.	62.81	5/20/1992
P-2A	81.2	10137411.92	4180365.31	30.0	51.2	6.0	55.2	N.R.	64.60	5/20/1992
P-3A	80.2	10136693.80	4180714.28	50.0	30.2	(15.0)	49.2	N.R.	64.20	5/20/1992
P-4A	77.9	10137183.48	4180994.58	50.0	27.9	(17.3)	35.9	N.R.	61.55	5/20/1992
P-20	79.2	10136277.33	4180918.70	50.0	29.2	(16.0)	50.2	N.R.	65.06	5/20/1992
P-21	77.2	10136384.08	4181265.05	50.0	27.2	(18.0)	43.2	N.R.	65.34	5/20/1992
P-22	79.4	10137255.04	4181522.65	50.0	29.4	(15.8)	47.4	N.R.	64.43	5/20/1992
P-23	78.3	10138628.90	4181692.68	50.0	28.3	(16.9)	47.3	N.R.	59.02	5/20/1992
P-24	73.5	10137894.73	4181909.60	50.0	23.5	(21.7)	46.5	N.R.	64.22	5/20/1992
P-25	75.8	10137125.23	4182145.28	50.0	25.8	(19.7)	47.8	N.R.	64.40	5/20/1992
P-26	75.5	10136583.56	4181930.94	50.0	25.5	(19.8)	51.5	N.R.	62.91	5/20/1992
P-27	82.2	10138238.90	4180606.04	50.0	32.2	(13.0)	48.2	N.R.	61.18	5/20/1992

[1] The permitted EDE for this facility is 45.23 ft-msl.
N.E. = Not Encountered; borehole not deep enough to penetrate the Lower Clay Stratum.
N.R. = Not Recorded.

Table 2-1 (Continued)
Summary of 45 Existing Borehole Depths and Elevations

Borehole Number	Borehole Surface Elevation (ft-msl)	Northing	Easting	Borehole Total Depth (ft)	Borehole Bottom Elevation (ft-msl)	Depth (Below) or Above EDE[1] (ft)	Top of Lower Clay Stratum Elevation (ft-msl)	Groundwater Elevation at Time of Drilling (ft-msl)	Static Groundwater Elevation (ft-msl)	Date of Static Groundwater Elevation Measurement
1998 Monitor Well and Gas Probes Borehole by Hydrex										
MW-1	82.9	10138068.44	4180066.94	33.0	47.9	2.7	48.9	55.9	55.94	7/12/2016
OW-2 (MW-2)	81.7	10138205.68	4180494.81	27.0	52.6	7.4	55.1	60.7	55.32	7/12/2016
OW-3 (MW-3)	83.4	10138298.61	4180822.58	36.0	47.2	2.0	51.2	59.4	54.77	7/12/2016
MW-4	81.0	10138518.51	4181298.20	34.0	44.7	(0.5)	45.7	54.0	53.31	7/12/2016
MW-5	77.7	10138423.81	4181786.96	29.0	48.7	3.4	50.7	53.7	50.14	1/11/2010
MW-6	73.5	10137761.48	4181987.88	27.0	46.5	1.2	47.5	54.5	50.05	1/11/2010
GMP-1	82.8	10138127.95	4180013.09	29.0	53.8	8.6	N.E.	56.8	67.79	4/24/1998
GMP-2	82.6	10138400.21	4180913.17	29.0	53.6	8.4	N.E.	58.6	60.44	4/24/1998
GMP-3	78.9	10138640.64	4181713.57	28.0	50.9	5.7	51.9	54.9	56.65	4/24/1998
GMP-4	73.5	10137777.42	4181977.83	20.0	53.5	8.3	N.E.	54.5	64.45	4/24/1998
GMP-5	81.1	10137163.04	4180489.51	27.0	54.1	8.9	60.1	63.1	65.53	4/24/1998
GMP-6	77.7	10136920.10	4182027.42	25.0	52.7	7.5	54.7	58.7	69.34	4/24/1998
GMP-7	78.9	10136619.23	4181024.00	23.0	55.9	10.7	N.E.	57.9	65.85	4/24/1998

[1] The permitted EDE for this facility is 45.23 ft-msl.
NE = Not Encountered; borehole not deep enough to penetrate the Lower Clay Stratum.
N.R. = Not Recorded.

Table 2-1 (Continued)
Summary of 45 Existing Borehole Depths and Elevations

Borehole Number	Borehole Surface Elevation (ft-msl)	Northing	Easting	Borehole Total Depth (ft)	Borehole Bottom Elevation (ft-msl)	Depth (Below) or Above EDE[1] (ft)	Top of Lower Clay Stratum Elevation (ft-msl)	Groundwater Elevation at Time of Drilling (ft-msl)	Static Groundwater Elevation (ft-msl)	Date of Static Groundwater Elevation Measurement
2005 Monitor Well Boreholes by Hydrex										
MW-7	74.7	10137174.54	4182161.64	34.0	39.7	(5.5)	46.7	51.7	53.75	7/12/2016
OW-8 (MW-8)	73.8	10136577.81	4182071.91	35.0	38.8	(6.4)	48.8	51.8	57.71	7/12/2016
OW-9 (MW-9)	78.2	10136752.90	4181509.87	45.0	31.4	(13.8)	34.4	48.5	55.85	7/12/2016
OW-10 (MW-10)	79.1	10136317.91	4181090.46	45.0	34.1	(11.1)	54.1	47.1	58.96	7/12/2016
MW-11	80.2	10136907.90	4180616.45	40.0	38.1	(7.1)	N.E.	48.2	56.8	7/12/2016
MW-12	82.0	10137449.83	4180347.78	40.0	40.7	(4.5)	N.E.	51.5	55.21	7/12/2016
2010 Monitor Well Boreholes by Hydrex										
MW-5R	75.3	10138168.36	4181861.37	30.5	43.2	(2.0)	46.7	51.3	57.11	7/12/2016
MW-6R	74.6	10137662.20	4182013.71	26.0	47.2	2.0	49.2	52.6	54.23	7/12/2016
MW-13	78.7	10138601.18	4181724.11	30.5	47.0	1.8	48.5	52.7	58.57	7/16/2012
2017 Geotechnical Boreholes by WCG										
WC-1	70.6	10137062.03	4180847.14	74	-3.4	(48.6)	32.6	45.1	N.R.	N.R.
WC-2	70.8	10136791.64	4181048.44	63	7.8	(37.4)	45.8	50.3	N.R.	N.R.
WC-3	68.9	10137260.46	4181768.94	65	3.9	(41.3)	46.4	49.9	N.R.	N.R.
WC-4	70.9	10136876.79	4181609.78	63	7.9	(37.3)	30.9	54.9	N.R.	N.R.
WC-5	80.0	10137147.28	4182051.22	98	-18	(63.2)	46.0	49.0	N.R.	N.R.
WCP-5	74.0	10137168.43	4182163.42	90	-16	(61.2)	46.5	49.0	45.20	1/14/2017

[1] The permitted EDE for this facility is 45.23 ft-msl.
N.E. = Not Encountered; borehole not deep enough to penetrate the Lower Clay Zone.
N.R. =- Not Recorded.

3 GROUNDWATER INVESTIGATION REPORT

3.1 Water Level Measurements

Groundwater at the facility has been evaluated using historical water level data from the facility's groundwater piezometers, monitoring wells, and observation wells. Historical piezometer groundwater elevation data was collected by SWL from February 2, 1991 through May 20, 1992 as part of the initial subsurface investigation for Permit No. MSW-2214. The SWL piezometer water level data were provided in Attachment 4, Appendix III-4-I, Appendix C of the approved Site Development Plan (Permit No. MSW-2214A) and have been included on pages IIIG-D-7 through IIIG-D-10 in Appendix IIIG-D. The historical groundwater elevation data from the facility's Subtitle D groundwater monitor wells are provided in Table 3-1. Groundwater potentiometric surface contour maps were prepared from the facilities most recent four semiannual groundwater monitoring event's well gauging data obtained from Hydrex. These contour maps are include as Figures IIIG-D.1 through IIIG-D.4. The highest recorded historical static groundwater elevation from each facility well and piezometer was plotted to create a highest measured groundwater surface. This highest measured groundwater surface contour is illustrated on Figure IIIG-D.5 in Appendix IIIG-D. It is noted that the groundwater contours depicted Figure IIIG-D.5 are the highest measures groundwater elevations recorded in each individual well and piezometer and do not represent a single groundwater monitoring event or actual groundwater flow.

Table 3-1
Subtitle D Monitoring Event Groundwater Elevations

Date of Gauging	MW-1	OW-2 (MW-2)	OW-3 (MW-3)	MW-4	MW-5	MW-5R	MW-6	MW-6R	MW-7	OW-8 (MW-8)	OW-9 (MW-9)	OW-10 (MW-10)	MW-11	MW-12	MW-13
11/12/1999	57.96	56.70	55.77	54.18	56.38	N.I.	57.23	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.
1/27/1999	63.02	61.24	60.08	57.93	60.51	N.I.	61.60	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.
4/15/1999	63.74	62.14	60.98	59.36	61.27	N.I.	61.08	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.
7/13/1999	61.42	60.02	58.98	57.47	59.04	N.I.	60.35	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.
10/13/1999	57.17	56.70	56.04	54.69	54.99	N.I.	52.54	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.
1/11/2000	55.37	54.88	54.23	52.85	53.57	N.I.	50.82	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.
4/12/2000	54.44	54.30	53.35	52.06	53.13	N.I.	50.20	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.
7/10/2000	54.76	54.30	53.39	51.95	53.04	N.I.	49.29	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.
1/29/2001	54.39	54.29	52.84	51.32	53.48	N.I.	49.28	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.
4/2/2001	55.85	55.14	54.14	52.43	54.75	N.I.	50.54	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.
7/9/2001	56.85	55.95	55.16	53.24	55.19	N.I.	54.03	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.
1/7/2002	58.09	56.83	55.94	53.88	56.23	N.I.	55.27	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.
4/11/2002	60.22	58.79	57.77	55.53	57.31	N.I.	57.90	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.
7/17/2002	57.31	56.63	55.89	54.16	54.67	N.I.	54.36	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.
2/9/2003	61.11	59.52	58.47	55.88	57.90	N.I.	58.46	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.
7/1/2003	58.40	57.71	57.03	55.41	56.03	N.I.	55.78	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.
10/6/2003	57.71	56.93	56.20	54.44	55.18	N.I.	54.12	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.
1/19/2004	60.73	59.29	58.22	55.76	57.43	N.I.	56.61	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.
7/6/2004	63.51	62.08	61.05	58.72	60.16	N.I.	60.82	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.
1/11/2005	58.60	57.47	56.66	54.79	55.37	N.I.	53.36	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.	N.I.
7/26/2005	57.03	56.54	55.89	54.23	54.24	N.I.	53.64	N.I.	53.46	54.90	53.96	56.38	57.55	56.37	N.I.

Notes: All water elevations listed above in feet above mean sea level.
Former well names indicated in parenthesis when applicable.
N.M. = Not Measured; water level not measured on this date.
DRY = no measurable groundwater present at time of gauging.
N.I. = Not Installed; monitor well had not yet been installed on this date.
P.&A. = monitor well was plugged and abandoned prior to this date.

Table 3-1 (continued)
Subtitle D Monitoring Event Groundwater Elevations

Date of Gauging	MW-1	OW-2 (MW-2)	OW-3 (MW-3)	MW-4	MW-5	MW-5R	MW-6	MW-6R	MW-7	OW-8 (MW-8)	OW-9 (MW-9)	OW-10 (MW-10)	MW-11	MW-12	MW-13
11/14/2005	55.21	54.72	54.08	52.92	52.88	N.I.	51.65	N.I.	51.49	50.21	51.94	54.30	55.68	54.34	N.I.
2/7/2006	54.70	54.29	53.21	51.61	51.70	N.I.	49.01	N.I.	50.11	52.96	52.10	54.43	55.33	53.91	N.I.
5/15/2006	55.58	54.80	54.06	52.20	50.97	N.I.	49.61	N.I.	49.49	53.40	52.85	55.30	56.33	54.67	N.I.
8/29/2006	56.73	55.87	55.14	53.11	52.74	N.I.	50.33	N.I.	51.66	54.78	54.05	56.89	58.10	56.12	N.I.
11/20/2006	57.51	56.45	55.58	53.45	53.21	N.I.	51.15	N.I.	52.64	56.57	55.25	58.40	59.20	57.22	N.I.
2/12/2007	60.72	59.24	58.15	55.29	55.46	N.I.	52.95	N.I.	55.54	61.39	57.94	61.35	61.66	59.77	N.I.
5/23/2007	62.10	60.80	59.79	57.07	57.27	N.I.	55.90	N.I.	56.95	61.92	59.34	62.76	63.43	61.48	N.I.
8/14/2007	61.21	60.10	59.18	56.93	57.17	N.I.	55.90	N.I.	56.66	61.39	58.76	62.23	62.88	60.76	N.I.
10/9/2007	58.66	58.02	57.31	55.70	55.90	N.I.	54.20	N.I.	54.69	57.88	56.66	59.65	60.43	58.59	N.I.
12/26/2007	58.75	56.57	56.05	54.60	54.69	N.I.	51.36	N.I.	53.29	55.57	54.41	56.83	57.76	56.20	N.I.
2/18/2008	57.15	56.40	55.72	54.20	55.05	N.I.	52.39	N.I.	54.32	56.30	54.98	57.53	58.18	56.64	N.I.
8/6/2008	57.57	56.93	56.17	54.39	54.36	N.I.	50.61	N.I.	51.81	54.93	54.39	57.11	58.29	56.81	N.I.
1/26/2009	57.94	56.90	56.09	54.06	53.97	N.I.	51	N.I.	52.73	54.73	54.35	57.13	58.24	56.76	N.I.
7/21/2009	58.04	57.44	56.75	54.90	54.58	N.I.	50.61	N.I.	53.26	55.03	54.84	57.65	58.64	57.44	N.I.
9/23/2009	N.M.	N.M.	55.19	N.M.	N.M.	N.I.	N.M.	N.I.	N.M.	N.M.	N.M.	N.M.	N.M.	N.M.	N.I.
1/11/2010	55.94	55.21	54.57	52.93	53.11	N.I.	50.05	N.I.	51.82	55.19	53.34	56.05	56.37	54.89	N.I.
3/29/2010	N.M.	N.M.	N.M.	N.M.	P.& A.	55.49	P.& A.	51.11	N.M.	N.M.	N.M.	N.M.	N.M.	N.M.	54.40
4/19/2010	58.88	57.71	56.81	54.71	P.& A.	55.50	P.& A.	51.06	54.20	59.06	56.17	59.40	59.30	57.75	54.71
7/13/2010	56.59	55.96	55.24	53.53	P.& A.	53.25	P.& A.	51.37	52.74	54.24	53.42	56.05	56.97	55.68	53.21
10/19/2010	54.89	54.29	53.62	52.16	P.& A.	51.59	P.& A.	49.93	50.73	52.12	51.39	54.03	55.33	53.78	51.81
1/12/2011	53.59	54.26	52.30	50.90	P.& A.	50.84	P.& A.	49.52	49.89	51.08	51.57	53.12	54.29	52.88	50.98

Notes: All water elevations listed above in feet above mean sea level.
Former well names indicated in parenthesis when applicable.
N.M. = Not Measured; water level not measured on this date.
DRY = no measurable groundwater present at time of gauging.
N.I. = Not Installed; monitor well had not yet been installed on this date.
P.&A. = monitor well was plugged and abandoned prior to this date.

Table 3-1 (continued)
Subtitle D Monitoring Event Groundwater Elevations

Date of Gauging	MW-1	OW-2 (MW-2)	OW-3 (MW-3)	MW-4	MW-5	MW-5R	MW-6	MW-6R	MW-7	OW-8 (MW-8)	OW-9 (MW-9)	OW-10 (MW-10)	MW-11	MW-12	MW-13
4/4/2011	53.26	54.31	52.26	50.90	P.& A.	51.00	P.& A.	49.35	47.94	50.67	50.30	52.72	53.89	52.44	51.02
7/11/2011	52.58	54.28	51.75	50.13	P.& A.	50.30	P.& A.	49.03	45.96	47.95	49.04	51.74	53.03	51.47	50.49
7/19/2011	N.M.	N.M.	N.M.	N.M.	P.& A.	N.M.	P.& A.	49.29	N.M.	N.M.	N.M.	N.M.	N.M.	N.M.	N.M.
10/3/2011	52.01	54.30	51.41	49.53	P.& A.	49.76	P.& A.	48.76	44.7	48.58	48.18	51.04	52.45	50.81	50.14
10/10/2011	N.M.	N.M.	N.M.	N.M.	P.& A.	49.82	P.& A.	DRY	N.M.	N.M.	N.M.	N.M.	N.M.	N.M.	50.17
1/11/2012	50.53	DRY	DRY	48.08	P.& A.	48.39	P.& A.	DRY	43.35	47.12	46.98	49.59	N.M.	49.38	48.69
1/19/2012	N.M.	N.M.	N.M.	N.M.	P.& A.	N.M.	P.& A.	DRY	N.M.	N.M.	N.M.	N.M.	N.M.	N.M.	48.62
3/21/2012	N.M.	N.M.	N.M.	N.M.	P.& A.	50.10	P.& A.	N.M.	N.M.	N.M.	N.M.	N.M.	N.M.	N.M.	N.M.
4/9/2012	52.00	53.85	51.27	49.46	P.& A.	50.16	P.& A.	DRY	46.54	49.49	50.04	51.19	N.M.	49.66	49.62
7/16/2012	51.79	54.17	51.26	49.33	P.& A.	49.76	P.& A.	DRY	46.2	49.28	49.14	51.22	51.92	50.68	49.57
10/3/2012	N.M.	N.M.	N.M.	N.M.	P.& A.	N.M.	P.& A.	DRY	N.M.	N.M.	N.M.	N.M.	N.M.	N.M.	N.M.
1/3/2013	DRY	53.59	51.12	48.73	P.& A.	47.11	P.& A.	DRY	44.54	48.94	48.35	50.67	49.28	48.93	DRY
4/1/2013	N.M.	N.M.	N.M.	N.M.	P.& A.	N.M.	P.& A.	DRY	N.M.	N.M.	N.M.	N.M.	N.M.	N.M.	N.M.
7/15/2013	51.27	DRY	DRY	48.80	P.& A.	49.26	P.& A.	DRY	45	48.27	48.30	50.62	51.11	49.75	DRY
1/6/2014	51.11	DRY	51.34	48.41	P.& A.	49.12	P.& A.	DRY	45.45	49.09	48.79	50.84	51.00	49.73	DRY
7/29/2014	51.91	54.25	51.25	49.73	P.& A.	51.16	P.& A.	49.42	49.45	50.82	50.13	52.26	51.95	50.54	DRY
1/6/2015	51.46	54.25	51.35	49.61	P.& A.	51.21	P.& A.	50.23	47.22	53.82	49.65	51.94	52.15	50.45	DRY
7/20/2015	53.36	54.15	51.66	45.39	P.& A.	54.41	P.& A.	52.00	51.50	54.70	52.95	55.21	53.71	52.66	DRY
1/25/2016	53.56	54.22	52.12	51.61	P.& A.	55.66	P.& A.	51.94	51.70	69.31	53.18	55.59	54.25	53.48	DRY
7/12/2016	55.94	55.32	54.77	53.31	P.& A.	57.11	P.& A.	54.23	53.75	57.71	55.85	58.96	56.80	55.21	DRY

Notes: All water elevations listed above in feet above mean sea level.
Former well names indicated in parenthesis when applicable.
N.M. = Not Measured; water level not measured on this date.
DRY = no measurable groundwater present at time of gauging.
N.I. = Not Installed; monitor well had not yet been installed on this date.
P.&A. = monitor well was plugged and abandoned prior to this date.

3.2 Permeability of the Uppermost Groundwater Zone

Vertical hydraulic conductivity (K_v) tests were performed by geotechnical laboratories on five subsurface samples collected by WCG in January 2017. The geotechnical laboratory reports are presented in Appendix IIIE-C. A summary of the results are listed in Table 3-2. The vertical hydraulic conductivity data indicate an Upper Clay Stratum arithmetic mean of $1.3x10^{-7}$ cm/s, Lower Clay Stratum arithmetic mean of $1.1x10^{-8}$ cm/s, and a Basal Clay Stratum vertical hydraulic conductivity of $2.8x10^{-8}$ cm/s. This range of hydraulic conductivities is reasonable for the low permeability sediments of these strata.

Table 3-2
Summary of Vertical Hydraulic Conductivity Results
(K_v from geotechnical laboratory tests)

Boring	Test Interval (ft-bgs)	K_v (cm/s)
Upper Clay Stratum		
WC-1	30-31	$1.7x10^{-7}$
WC-2	20-21	$9.7x10^{-8}$
Arithmetic Mean:		$1.3x10^{-7}$
Lower Clay Stratum		
WC-3	28-29	$1.1x10^{-8}$
WC-5	39-40	$1.1x10^{-8}$
Arithmetic Mean:		$1.1x10^{-8}$
Basal Clay Stratum		
WC-5	97-98	$2.8x10^{-8}$

Hydraulic conductivity (K) rates were computed by SWL as part of the initial permit application (Permit No. MSW-2214) in 1991 and by WCG in 2017. In 1991, SWL conducted rising head slug tests in eight piezometers (P-1A, P-2A, P-3A, P-4A, P-21, P-22, P-24, and P-26). These tests were performed to determine horizontal hydraulic conductivity values for the permitted 49.6-acre Type I waste footprint and 79-acre permit boundary. The SWL lithologic and well completion logs for the slug tested piezometers indicate that one piezometer was screened in the Upper Clay Stratum (P-2A) and seven piezometers were screened within the Upper Sand Stratum (P-1A, P-3A, P-4A, P-21, P-22, P-24, and P-26). In 2017, WCG conducted a rising head slug test in existing groundwater monitor well MW-7. The test was performed to provide a comparative and supplemental horizontal hydraulic conductivity value to those computed by SWL in 1991. The SWL and WCG slug test results are summarized in Table 3-3. Hydraulic conductivity for MW-7 was computed from the slug test data using Aqtesolv™. The graph and data summary sheet for the MW-7 slug test calculation is presented as page IIIG-D-6 in Appendix IIIG-D. The hydraulic conductivity computed by WCG in 2017 from the MW-7 slug test data indicated a Upper Sand Stratum hydraulic conductivity of $3.03x10^{-4}$ cm/s. This value is consistent with those computed by SWL in 1991 ($7.52x10^{-6}$ to $3.00x10^{-4}$ cm/s). The hydraulic conductivity computed by SWL from the piezometer P-2A slug test data indicated an Upper Clay Stratum hydraulic conductivity if $8.18x10^{-5}$ cm/s. The range of hydraulic conductivity values computed by SWL and WCG appear reasonable for the permeability of the screened sediments.

Table 3-3
Summary of Horizontal Hydraulic Conductivity Results
(K_h from Field Slug Test Measurements)

Location	Test Interval (ft-bgs)	Test Type	Solution Method	Hydraulic Conductivity (K in cm/s)
Upper Clay Stratum				
P-2A	19-30	Slug	Bouwer-Rice	$8.18x10^{-5}$
Upper Sand Stratum				
P-1A	25-36	Slug	Bouwer-Rice	$3.00x10^{-4}$
P-4A	33-42	Slug	Bouwer-Rice	$2.78x10^{-5}$
P-21	24-35	Slug	Bouwer-Rice	$1.49x10^{-5}$
P-22	24-35	Slug	Bouwer-Rice	$9.80x10^{-6}$
P-24	16-27	Slug	Bouwer-Rice	$1.09x10^{-4}$
P-26	19-30	Slug	Bouwer-Rice	$7.52x10^{-6}$
MW-7	21-34	Slug	KGS (with skin)	$3.03x10^{-4}$
Arithmetic Mean:				$1.10x10^{-4}$
Maximum:				$3.03x10^{-4}$
Minimum:				$7.52x10^{-6}$

Notes: Slug test results from MW-7 completed by WCG in 2017.
Slug test results from piezometers completed by SWL in 1991.
SWL slug test results were obtained from Table 3 in Attachment 5 of Permit No. MSW-2214A.

The site-wide hydraulic gradients were approximated from the IIIG-D.1 through IIIG-D.4 (Appendix IIIG-D) groundwater contour maps at 0.0038 ft/ft between OW-10 and MW-7 and 0.001ft/ft between MW-11 and MW-4. An effective porosity has been conservatively estimated at 20 percent in the Upper Sand Stratum (after Driscoll, 1989). The following groundwater linear velocity calculation uses the hydraulic gradients of 0.0012 and 0.003 ft/ft, a maximum Upper Sand Stratum hydraulic conductivity (K) value of $3.03x10^{-4}$ cm/s, and a silty sand effective porosity (n_e) of 0.20. The formula for the velocity calculation is:

$$V = K^* \, i \, {}^*1{,}034{,}646 / \, n_e$$

Where:

V = linear velocity
K = radial hydraulic conductivity (cm/s)
i = hydraulic gradient (ft/ft)
1,034,646 = scalar to convert from cm/s to ft/year
n_e = effective porosity

Using these conservative values and estimates, the maximum groundwater linear velocity in the uppermost aquifer is estimated at 1.9ft/yr (MW-11 to MW-4) to 5.9ft/yr (OW-10 to MW-7).

3.3 Hydrogeologic Interpretation

Appendix D presents historical groundwater potentiometric surface contour maps by Hydrex and Biggs and Mathews Environmental (B&M) taken from Attachment 5 of the facility's Site Development Plan (Permit No. MSW-2214A) and the facility's 2015 Annual Groundwater Monitoring Report submittal. As indicated by these figures, groundwater in the uppermost aquifer flows to the east and northeast toward tributaries of the Neches River (Cypress Creek and Longston Branch). B&M (2009) described this groundwater regime as closely mimicking surface topography, flowing from the west to topographic lows in the eastern and northeastern areas of the site.

Groundwater at the site occurs within the Upper Sand Stratum and Lower Sand Stratum. These two saturated strata are separated by the low permeability clay of the Lower Clay Stratum (aquiclude). WCG gauged static groundwater elevations in Upper Sand Stratum screened monitor well MW-7 and adjacent Lower Sand Stratum screened piezometer WCP-5 over a period of four consecutive days in January 2017 and again in July 2017. It is noted that MW-7 and WCP-5 are located less than seven feet away from one another. These groundwater elevation data are provided in Table 3-4 and reveal a static hydraulic head difference between the Upper Sand Stratum and Lower Sand Stratum ranging from 4.26 to 5.04 feet of separation in January 2017 and 4.52 feet of separation in July 2017. This consistent static head separation, coupled with the low vertical hydraulic conductivity of the Lower Clay Stratum, demonstrates a lack of hydraulic interconnectivity between the Upper Sand and Lower Sand strata.

Table 3-4
Summary of MW-7 and WCP-5 Static Groundwater Elevation Data

Date of Measurement	Static Groundwater Elevation (ft-msl)		Separation (feet)
	MW-7	WCP-5	
January 10, 2017	50.15	45.42	4.73
January 11, 2017	50.13	45.87	4.26
January 12, 2017	50.08	45.49	4.59
January 13, 2017	50.05	45.24	4.81
January 14, 2017	50.03	44.99	5.04
July 7, 2017	52.07	47.55	4.52

Notes: MW-7 top of casing elevation is 76.95 ft-msl.
WCP-5 top of casing elevation is 77.00 ft-msl.

The existing Permit No. MSW-2214A groundwater monitoring system design targets a single uppermost aquifer comprised of the Upper Sand Stratum sediments. All facility groundwater monitor wells are screened within the Upper Sand Stratum using ten foot screen intervals. Due to the limited vertical extent of the Upper Sand Stratum, the facility's groundwater monitoring wells are also screened partially within the bounding Upper Clay Stratum and/or Lower Clay Stratum to varying degrees. The Lower Clay Stratum comprises the lower confining unit (aquiclude) for groundwater present in unconfined (water table) conditions within the Upper Sand Stratum (uppermost aquifer). Groundwater elevations measured in the facility's perimeter wells indicate groundwater flow largely mimics the natural surface topography and flows across the top of the Lower Clay Zone towards to the east and northeast. Excavation into the Upper Sand Stratum is limited to the immediate vicinity of the landfill's existing northern sump and the proposed southeastern sump. Groundwater in the Upper Sand Stratum diverges around the liner in the immediate area of these two sumps.

As no changes are proposed to the permitted permit boundary, limits of waste, or elevation of deepest excavation, and based on the data and information provided in the facility's approved permit (Permit No. MSW-2214A) and the January 2017 subsurface investigation by WCG, no changes are proposed to the currently installed and permitted groundwater monitoring system.

3.4 Contaminant Pathways

In the unlikely occurrence of a release of leachate from the landfill unit, the most probable pathway for the migration of pollutants is vertically through the vadose zone and into the saturated Upper Sand Stratum or from sumps seated in Upper Sand Stratum (northern existing and southeastern proposed). Once within the Upper Sand Stratum, light non-aqueous phase liquid migration would occur along the upper surface of the Upper Sand Stratum (uppermost aquifer) and dense non-aqueous phase liquid migration would occur within the Upper Sand Stratum along the upper surface of the Lower Clay Stratum (aquiclude). In either case, the pollutants would be transported within the Upper Sand Stratum uppermost aquifer and down gradient in the direction of groundwater flow toward the permitted point of compliance and network of groundwater detection monitoring wells.

4 REFERENCES

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APPENDIX IIIG-A

REGIONAL GEOLOGIC/HYDROGEOLOGIC DATA

STATE OF TEXAS
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CONTENTS

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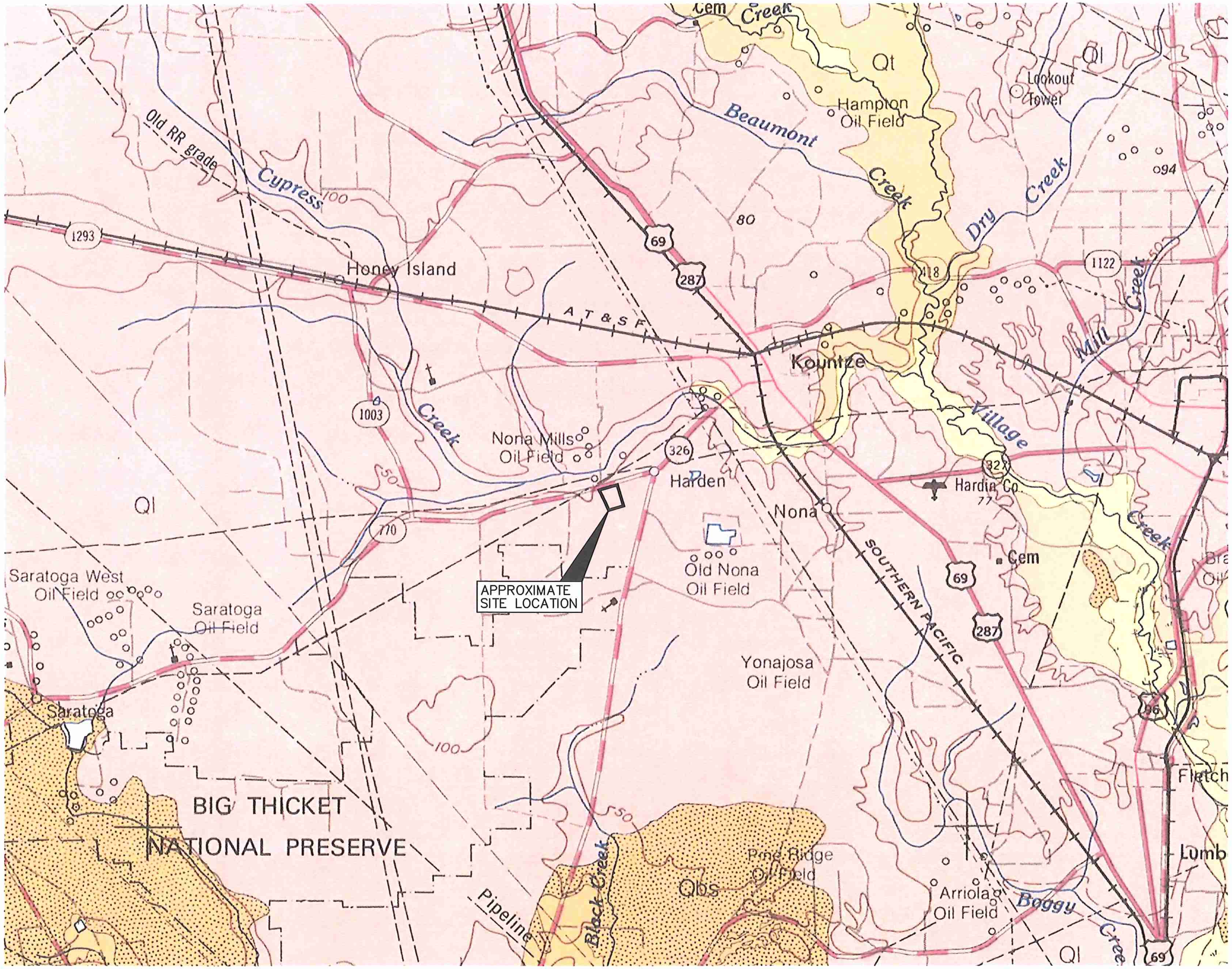

LEGEND

- Qal Alluvium
- Qd Qd? Deweyville Formation
- Qt Fluviatile terrace deposits undivided
- Qbc Qbs Qnb Beaumont Formation
- Ql Lissie Formation

NOTE:

1. REGIONAL GEOLOGIC MAP MODIFIED FROM GEOLOGIC ATLAS OF TEXAS MAP, BEAUMONT SHEET 1992, BUREAU OF ECONOMIC GEOLOGY.

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MAJOR PERMIT AMENDMENT
REGIONAL GEOLOGIC MAP

HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS

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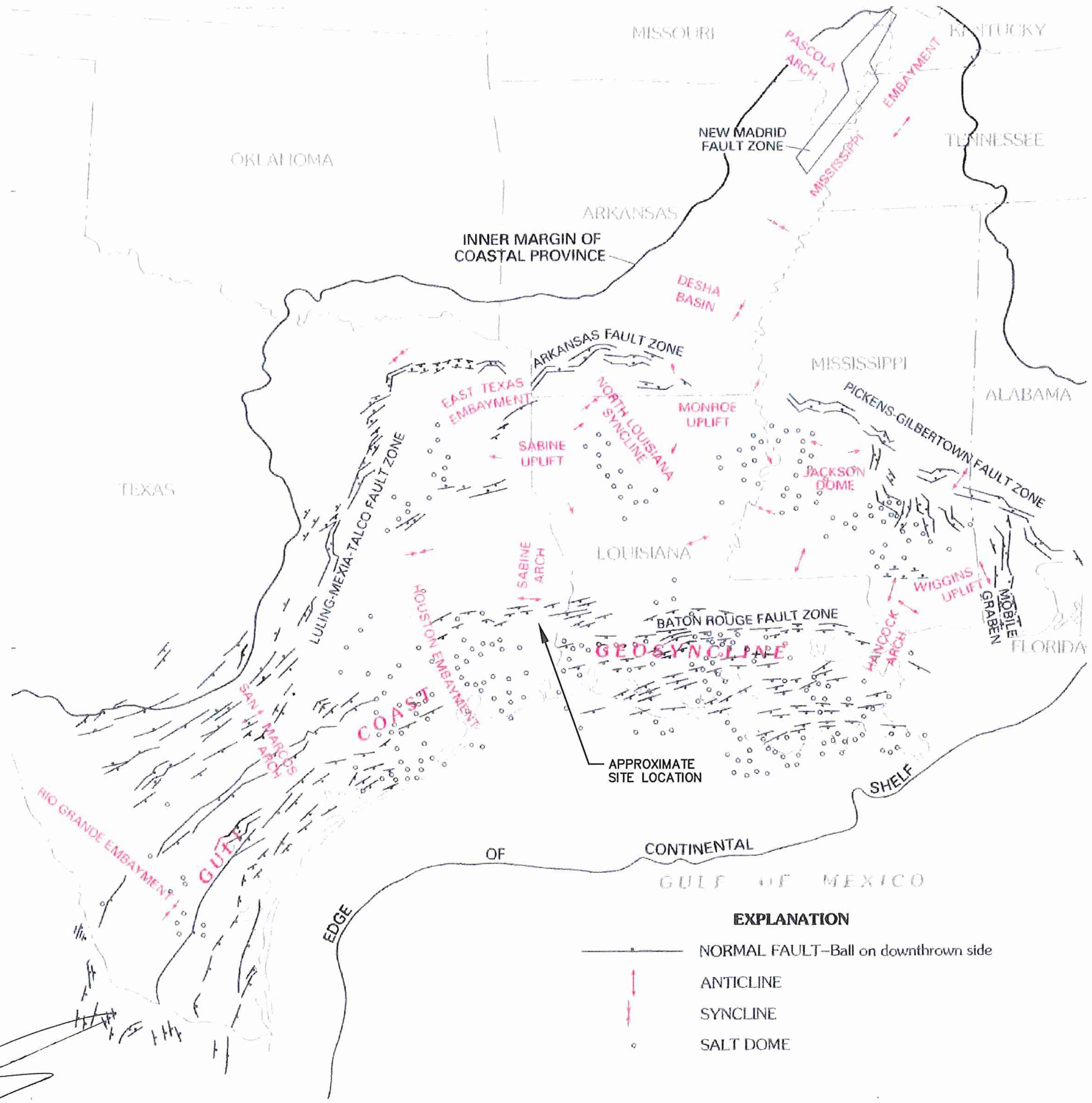
FIGURE IIIG-A.1

O:\0120\758\2214B EXPANSION\IIIG\IIIG-A.1_REGIONAL GEOLOGIC MAP.dwg, 11/15/2017 1:03:41 PM, rsellers, 1:2



EXPLANATION

NORMAL FAULT–Ball on downthrown side

ANTICLINE

SYNCLINE

SALT DOME

NOTES:

1. REGIONAL STRUCTURAL FEATURES ADAPTED FROM HOSMAN ET AL, 1996, REGIONAL STRATIGRAPHY AND SUBSURFACE GEOLOGY OF CENOZOIC DEPOSITS, GULF COAST PLAIN, SOUTH–CENTRAL UNITED STATES.

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REGIONAL STRUCTURAL FEATURES MAP

HARDIN COUNTY LANDFILL
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FIGURE IIIG-A.2

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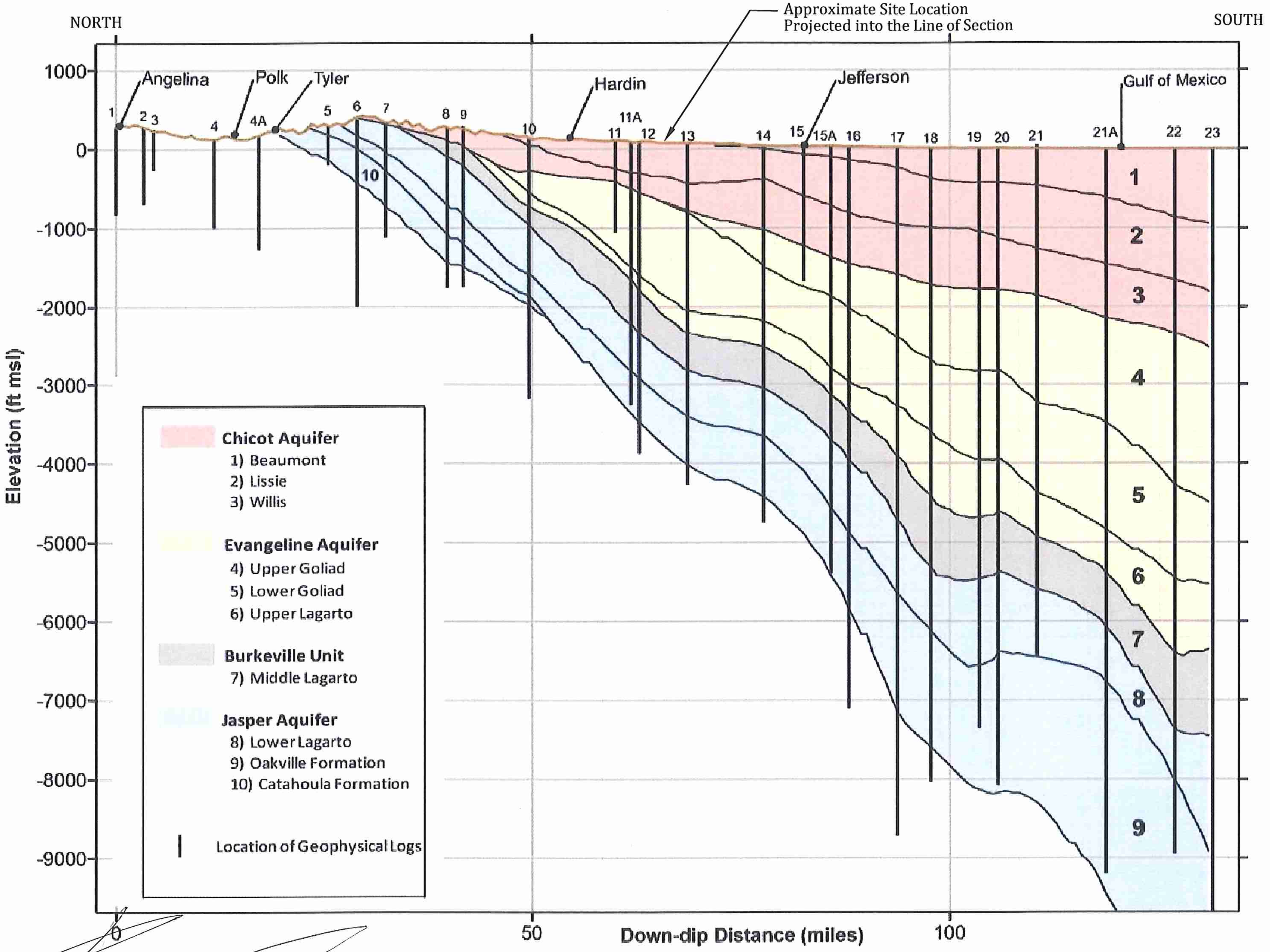
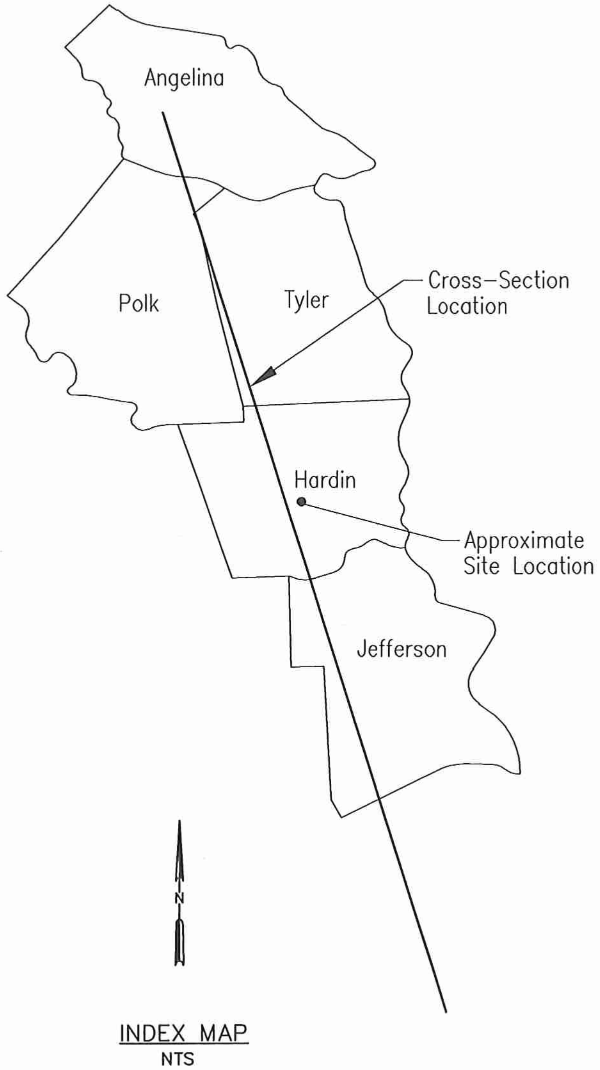


🟥	**Chicot Aquifer** 1) Beaumont 2) Lissie 3) Willis
🟨	**Evangeline Aquifer** 4) Upper Goliad 5) Lower Goliad 6) Upper Lagarto
▨	**Burkeville Unit** 7) Middle Lagarto
🟦	**Jasper Aquifer** 8) Lower Lagarto 9) Oakville Formation 10) Catahoula Formation
|	Location of Geophysical Logs



INDEX MAP
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NOTE:

1. REGIONAL GEOLOGIC CROSS-SECTION MODIFIED FROM YOUNG, et.Al., 2012, UPDATING THE HYDROGEOLOGIC FRAMEWORK FOR THE NORTHERN PORTION OF THE GULF COAST AQUIFER, TEXAS WATER DEVELOPMENT BOARD, FINAL REPORT.

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REGIONAL GEOLOGIC
CROSS-SECTION

HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS

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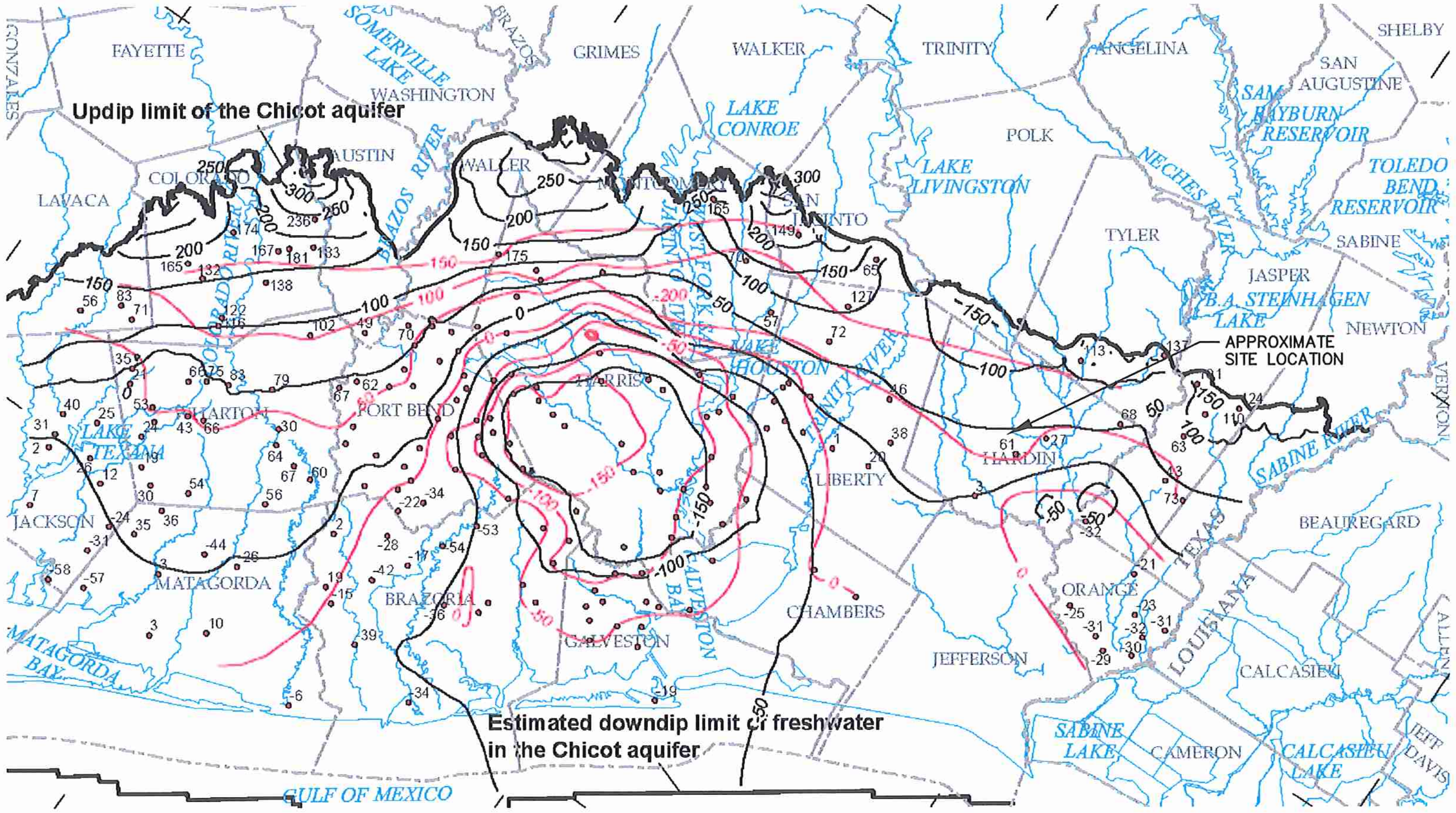
FIGURE IIIG-A.3



0 15 30
SCALE IN MILES

N

EXPLANATION

—-50— Simulated potentiometric contour – Shows altitude at which water would have stood in tightly cased well Interval 50 feet. Datum is NGVD 29

—-50— Measured potentiometric contour – Shows altitude at which water stands in tightly cased well Interval 50 feet. Datum is NGVD 29

•17 Data point – Well in which water-level measurement was made. Number is water-level altitude (shown in areas not having published water-level contours)

NOTE:

1. POTENTIOMETRIC SURFACE MAP ADAPTED FROM KASMAREK, 2013, USGS, SCIENTIFIC INVESTIGATION REPORT 2012–5154.

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MAJOR PERMIT AMENDMENT REGIONAL CHICOT AQUIFER POTENTIOMETRIC SURFACE MAP

HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS

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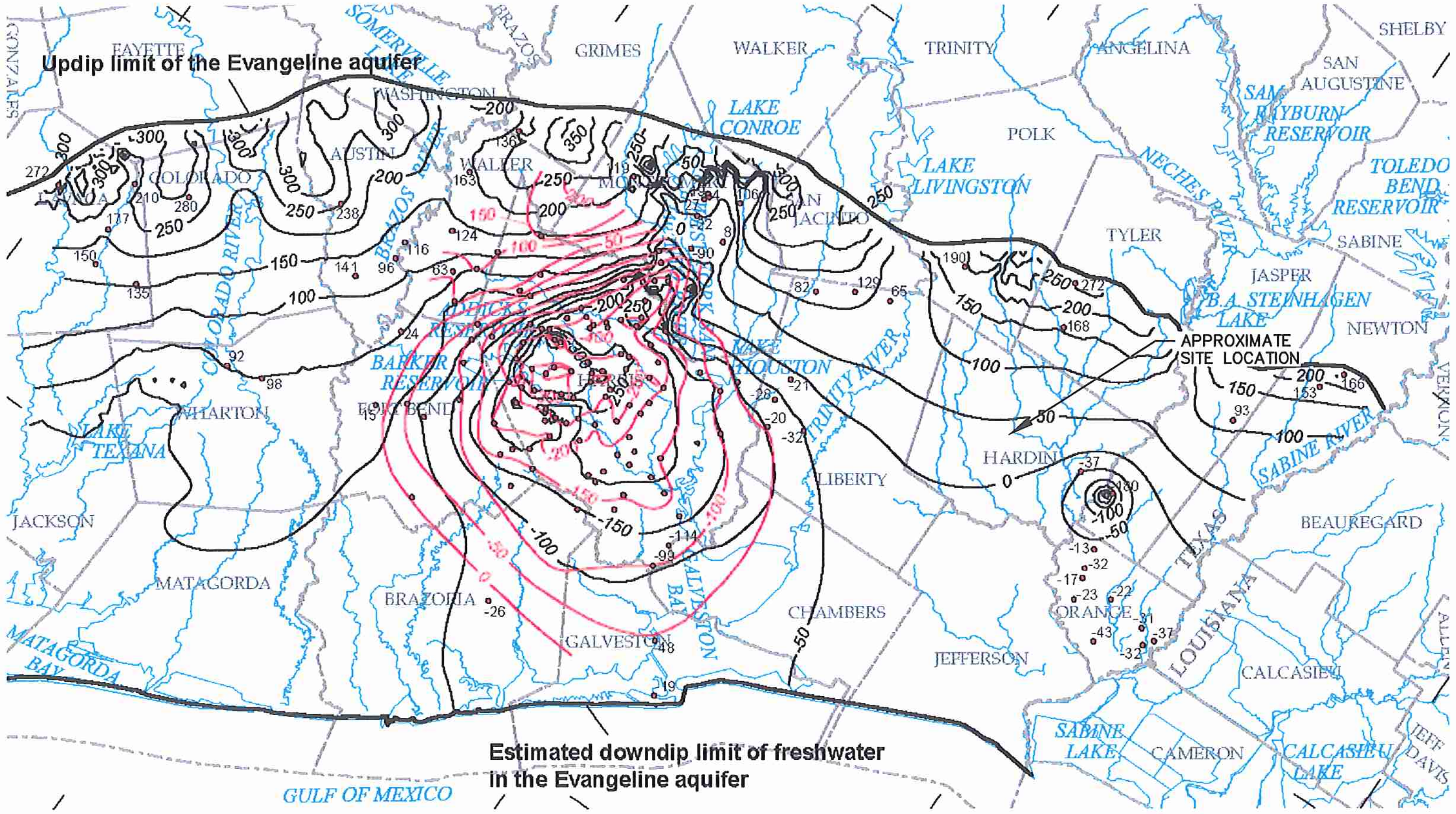
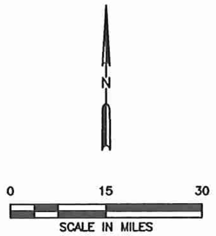
FIGURE IIIG–A.4

O:\0120\758\2214B EXPANSION\IIIG\IIIG-A.4_REGIONAL CHICOT AQUIFER.dwg, 11/15/2017 1:12:12 PM, rsellers, 1:2





EXPLANATION

-50 Simulated potentiometric contour — Shows altitude at which water would have stood in tightly cased well Interval 50 feet. Datum is NGVD 29

-50 Measured potentiometric contour — Shows altitude at which water stands in tightly cased well Interval 50 feet. Datum is NGVD 29

•-17 Data point — Well in which water-level measurement was made. Number is water-level altitude (shown in areas not having published water-level contours)

NOTE:

1. POTENTIOMETRIC SURFACE MAP ADAPTED FROM KASMAREK, 2013, USGS, SCIENTIFIC INVESTIGATION REPORT 2012-5154.

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MAJOR PERMIT AMENDMENT
REGIONAL EVANGELINE AQUIFER
POTENTIOMETRIC SURFACE MAP

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HARDIN COUNTY, TEXAS

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FIGURE IIIG-A.5

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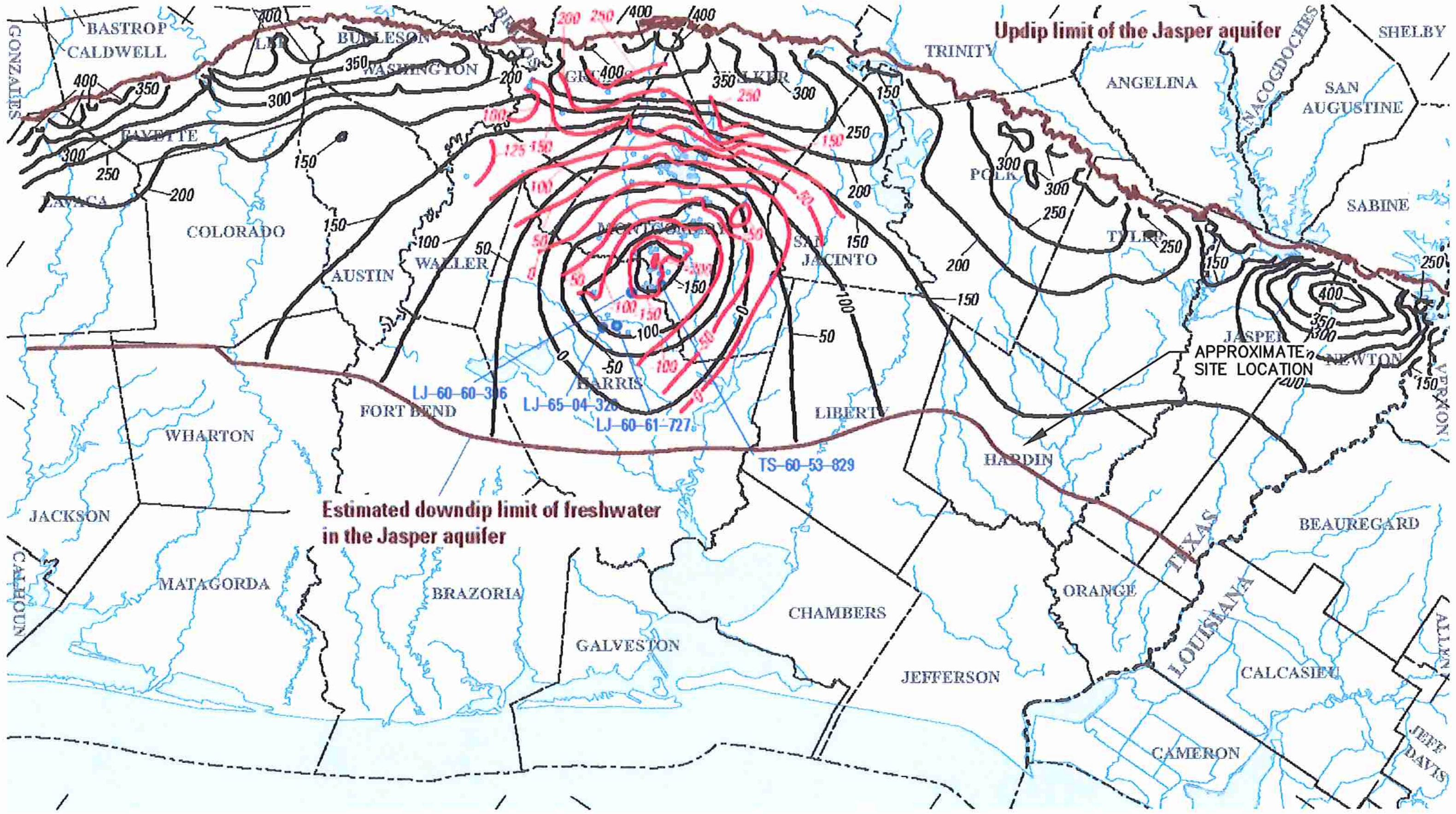

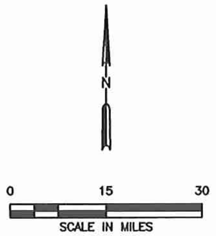

EXPLANATION

-50	Simulated potentiometric contour – Shows altitude at which water would have stood in tightly cased well Interval 50 feet. Datum is NAVD 88
50	Measured potentiometric contour – Shows altitude at which water stands in tightly cased well Interval 50 feet. Datum is NAVD 88
•	Data point – Well in which water–level measurement was made
● LJ-60-60-306	Data point and well number – Well in which water–level measurement was made and for which hydrograph is shown on figure 28

NAVD 88, North American Vertical Datum of 1988

NOTE:

1. POTENTIOMETRIC SURFACE MAP ADAPTED FROM KASMAREK, 2013, USGS, SCIENTIFIC INVESTIGATION REPORT 2012–5154.

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REGIONAL JASPER AQUIFER
POTENTIOMETRIC SURFACE MAP

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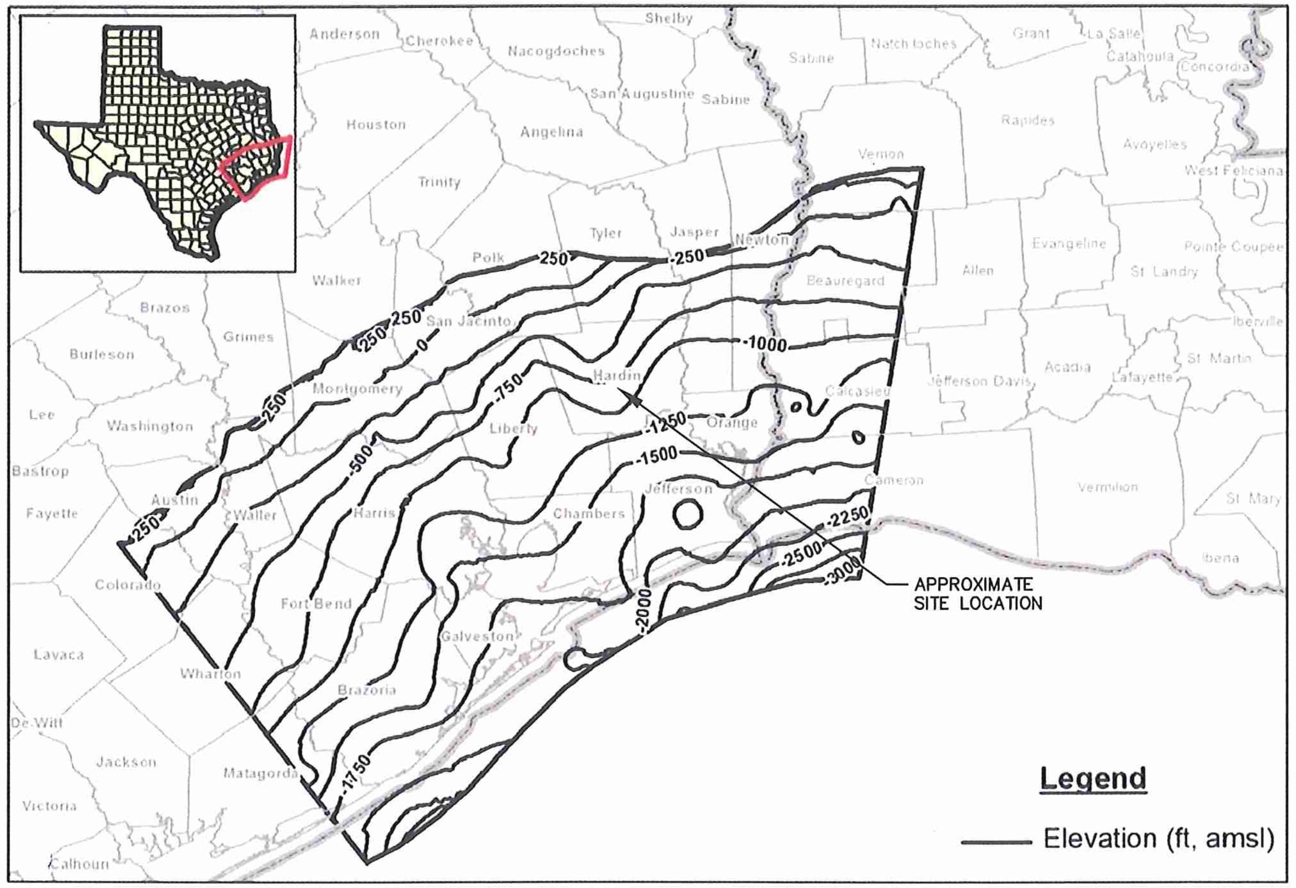
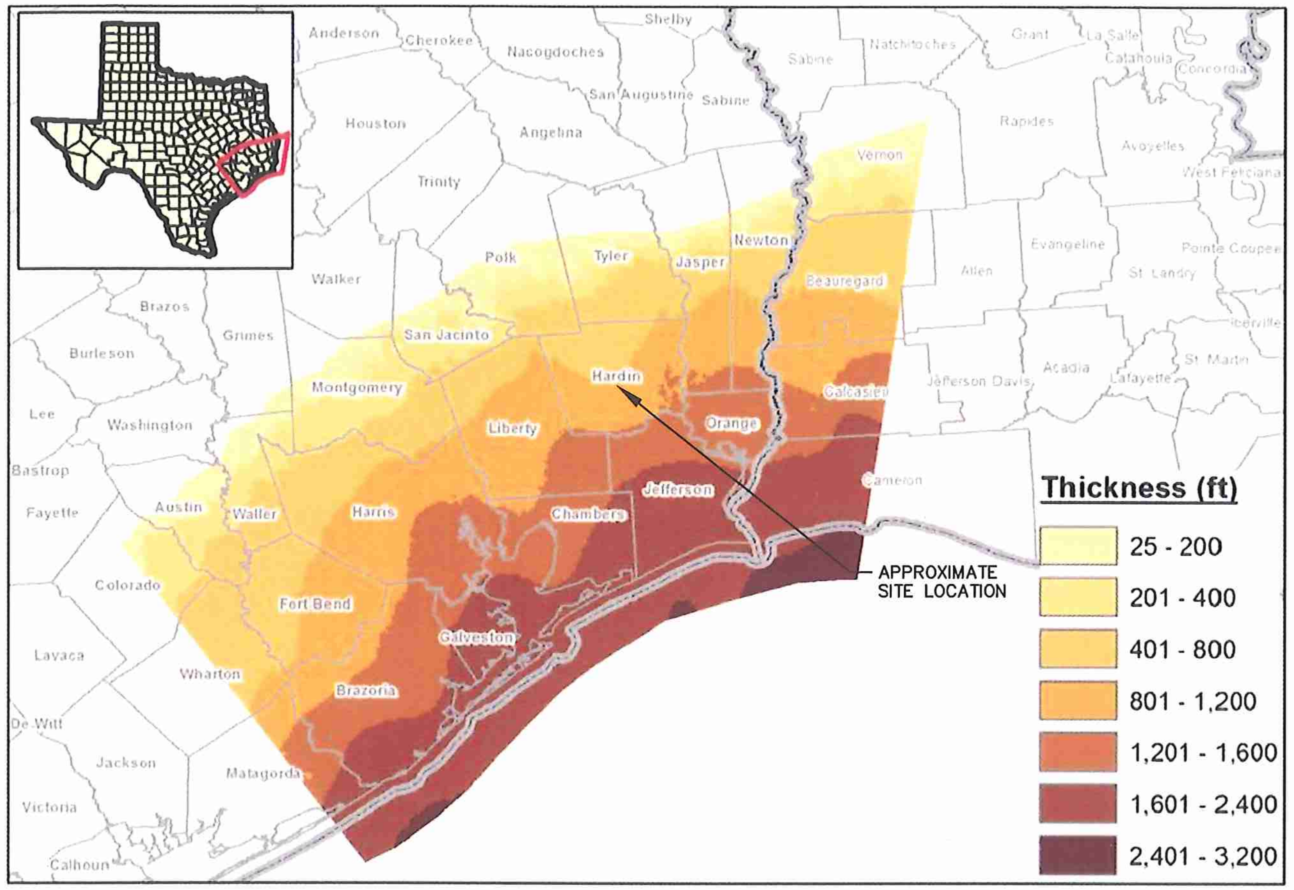
FIGURE IIIG–A.6

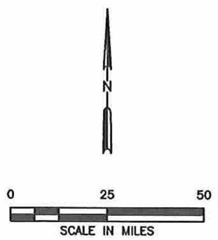
O:\0120\758\2214B EXPANSION\IIIG\IIIG-A.7_CHICOT AQUIFER THICKNESS MAP.dwg, 11/15/2017 1:15:44 PM, rsellers, 1:2

CHICOT AQUIFER BASE ELEVATION



CHICOT AQUIFER THICKNESS





NOTE:

1. CHICOT AQUIFER ELEVATION AND THICKNESS MAPS MODIFIED AFTER YOUNG, et.Al., 2012, UPDATING THE HYDROGEOLOGIC FRAMEWORK FOR THE NORTHERN PORTION OF THE GULF COAST AQUIFER, TEXAS WATER DEVELOPMENT BOARD, FINAL REPORT.

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REGIONAL CHICOT AQUIFER ELEVATION AND THICKNESS MAPS

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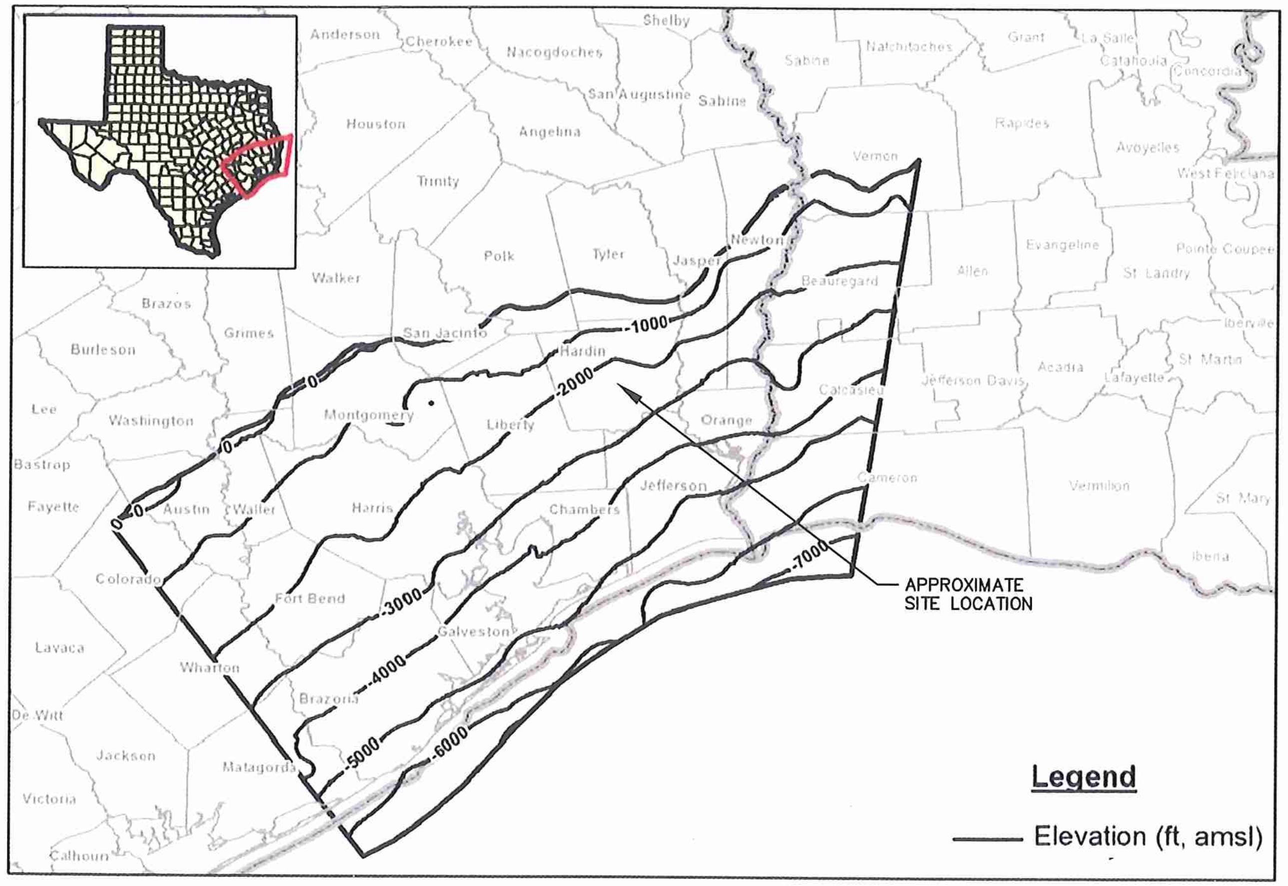
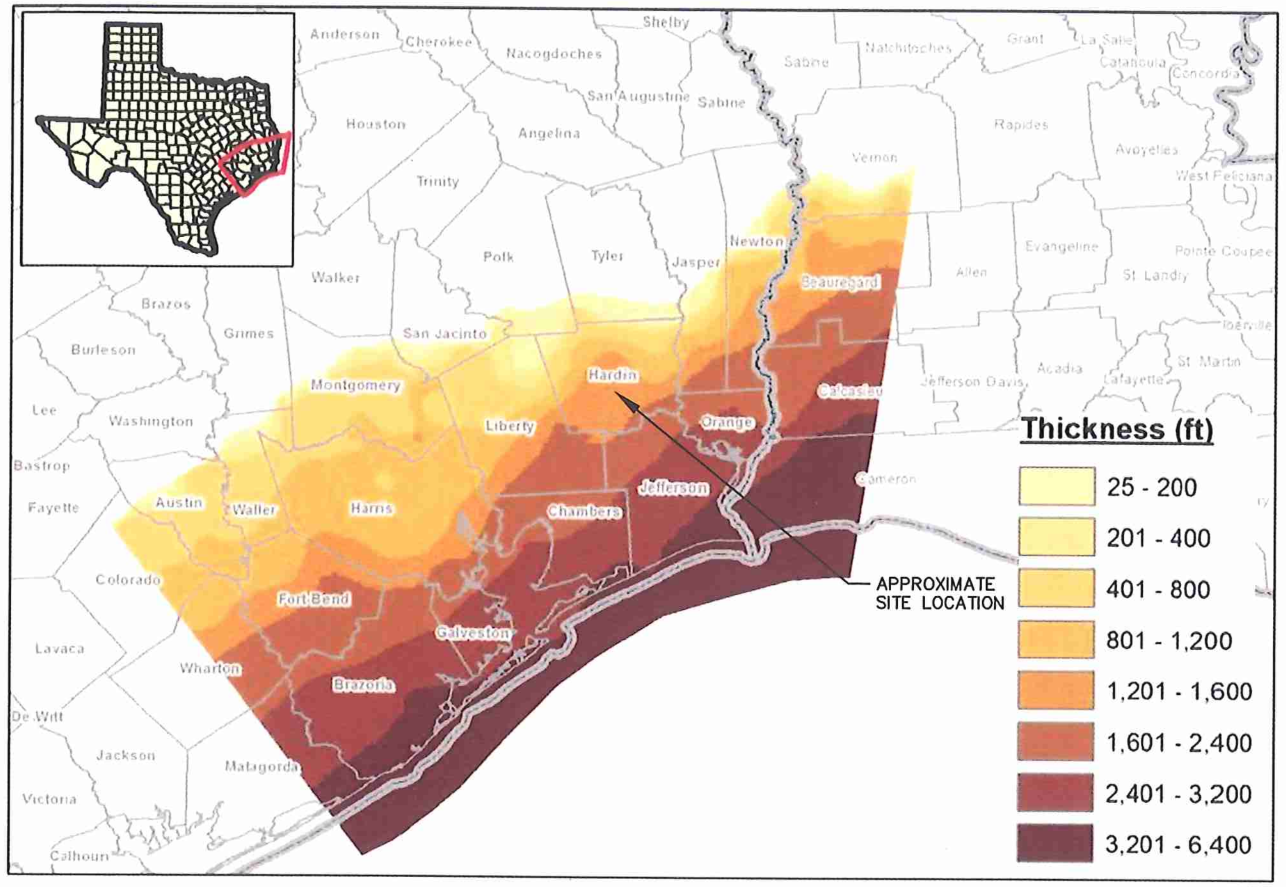
FIGURE IIIG-A.7

EVANGELINE AQUIFER BASE ELEVATION



EVANGELINE AQUIFER THICKNESS



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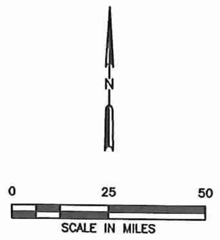

NOTE:

1. EVANGELINE AQUIFER ELEVATION AND THICKNESS MAPS MODIFIED AFTER YOUNG, et.Al., 2012, UPDATING THE HYDROGEOLOGIC FRAMEWORK FOR THE NORTHERN PORTION OF THE GULF COAST AQUIFER, TEXAS WATER DEVELOPMENT BOARD, FINAL REPORT.

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REGIONAL EVANGELINE AQUIFER
ELEVATION AND THICKNESS MAPS

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HARDIN COUNTY, TEXAS

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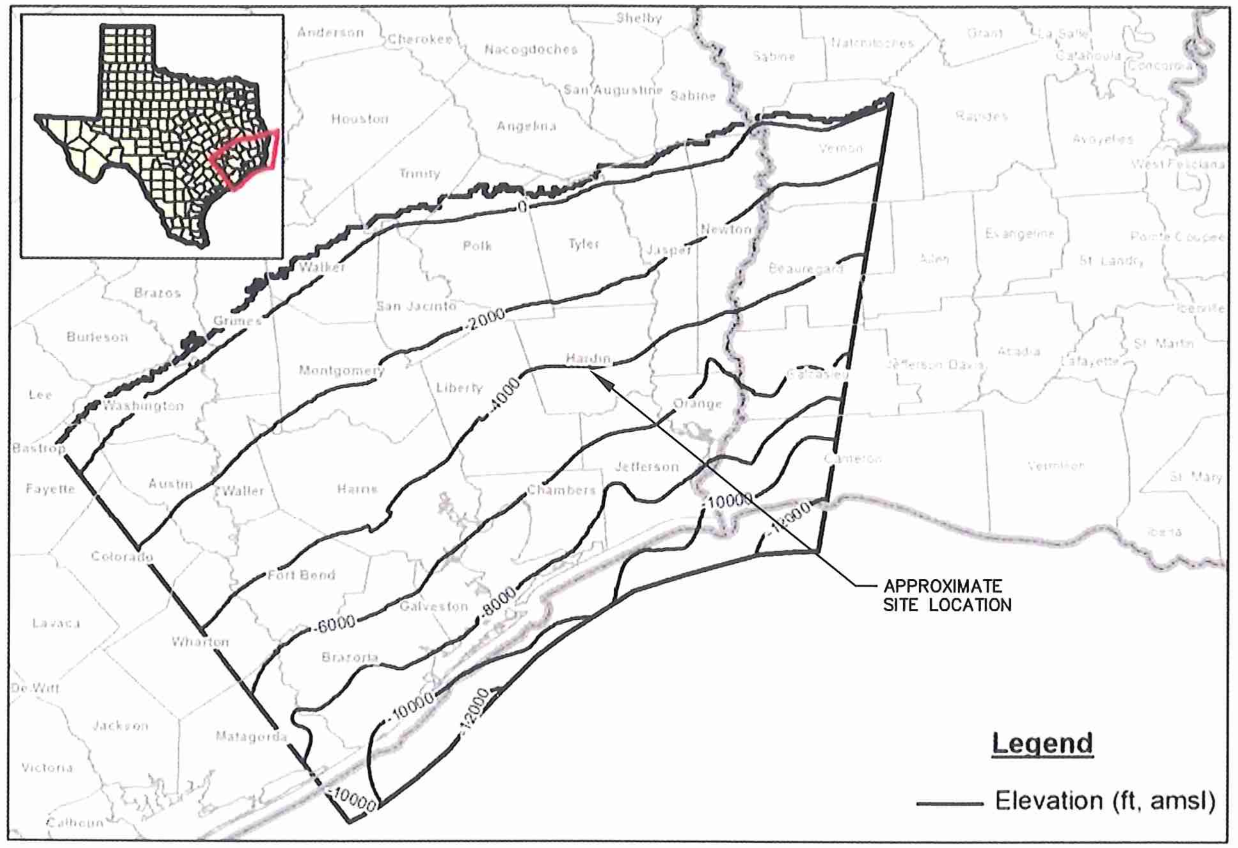
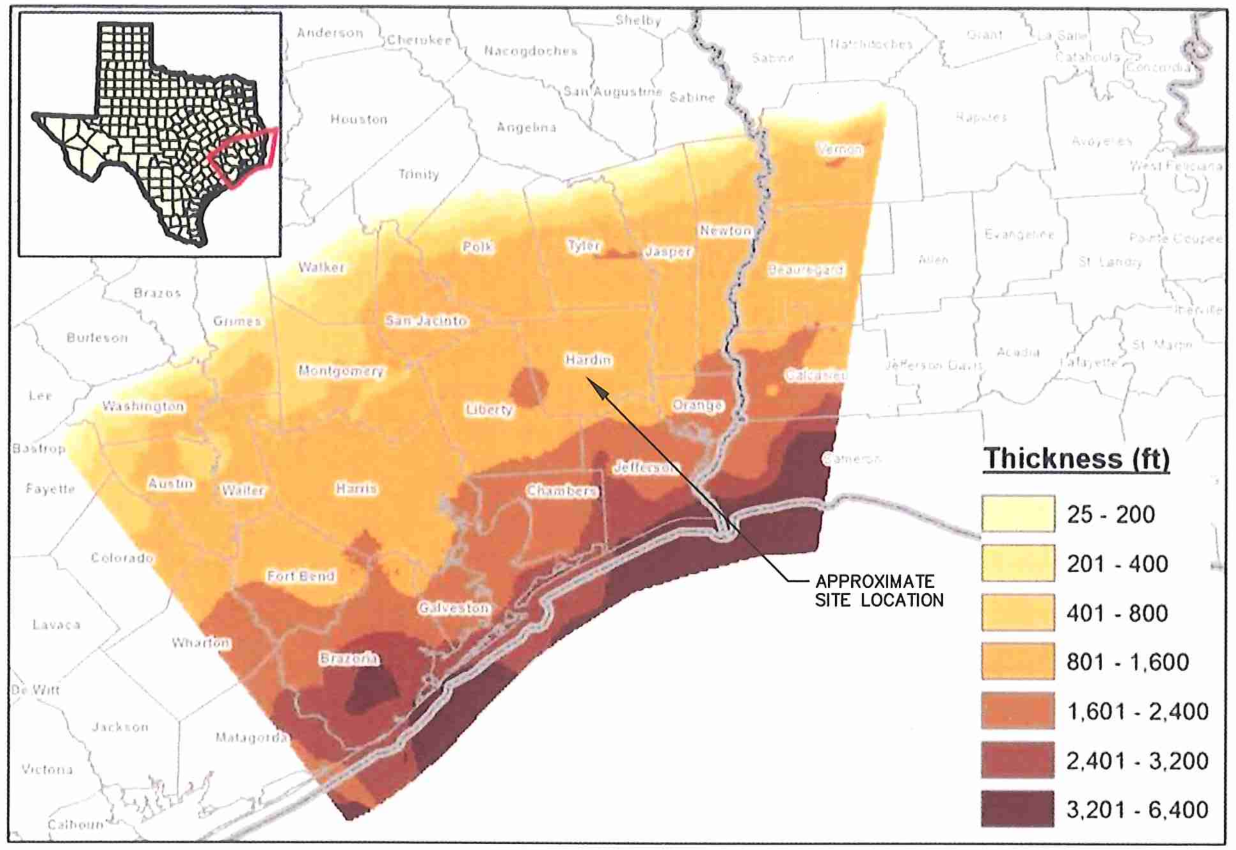
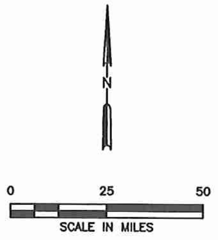
FIGURE IIIG-A.8

JASPER AQUIFER BASE ELEVATION



JASPER AQUIFER THICKNESS





NOTE:

1. JASPER AQUIFER ELEVATION AND THICKNESS MAPS MODIFIED AFTER YOUNG, et.Al., 2012, UPDATING THE HYDROGEOLOGIC FRAMEWORK FOR THE NORTHERN PORTION OF THE GULF COAST AQUIFER, TEXAS WATER DEVELOPMENT BOARD, FINAL REPORT.

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MAJOR PERMIT AMENDMENT REGIONAL JASPER AQUIFER ELEVATION AND THICKNESS MAPS

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FIGURE IIIG-A.9

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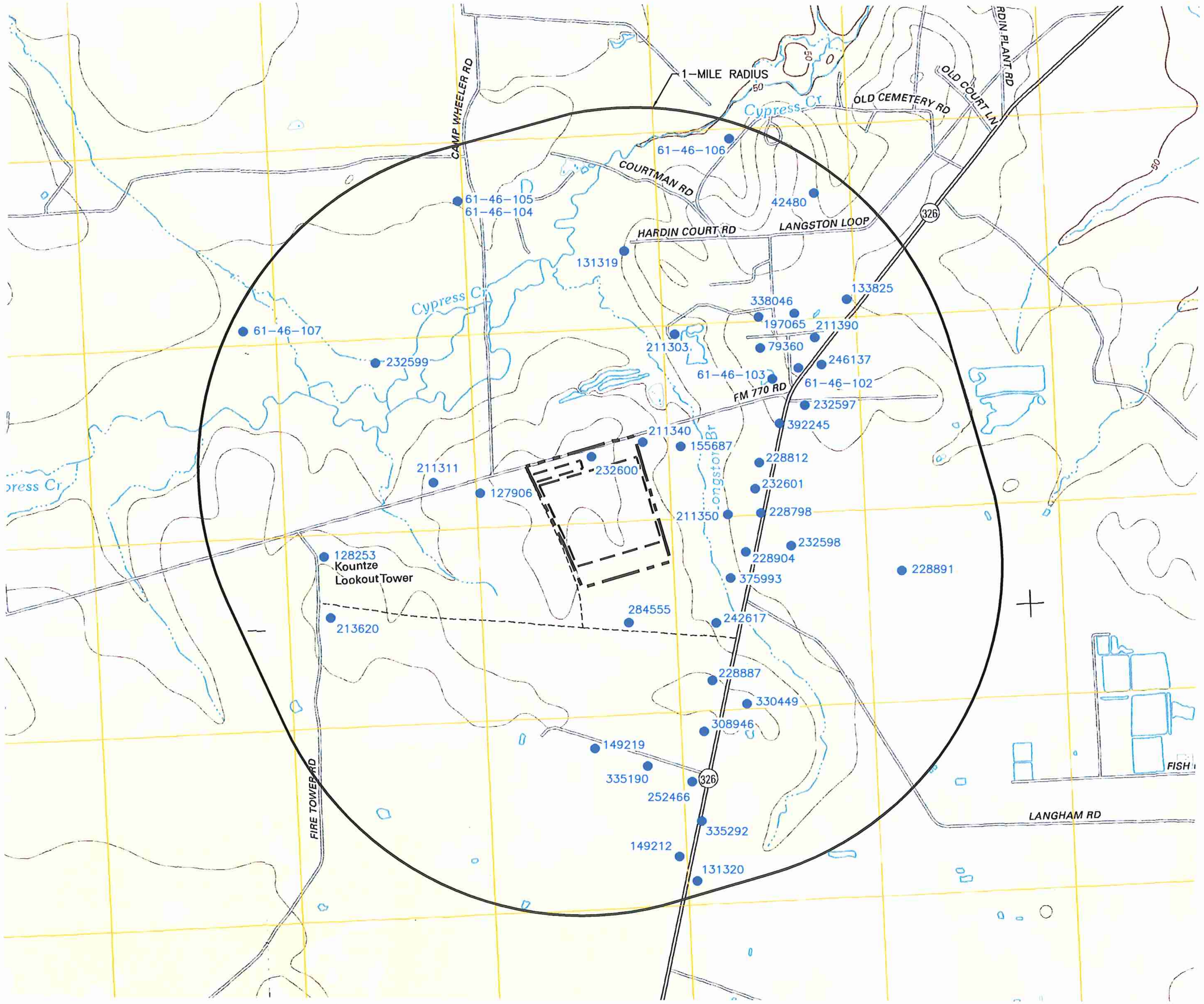

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0 1000 2000
SCALE IN FEET

LEGEND

EXISTING PERMIT BOUNDARY

PERMITTED LIMITS OF WASTE

● 232601 WATER WELL IDENTIFIED BY GEOSEARCH FOR A ONE MILE RADIUS AREA (SEE NOTE 2)

ROAD CLASSIFICATION

Interstate Route
US Route
Ramp
State Route
Local Road
4WD
Interstate Route
US Route
State Route

KOUNTZE SW, TX
2013

KOUNTZE SOUTH, TX
2013

NOTES:

1. SITE LOCATION BASE MAP ADAPTED FROM USGS 7.5 MINUTE TOPOGRAPHIC QUADRANGLE MAPS REFERENCED ABOVE.
2. WATER WELLS IDENTIFIED IN TEXAS WATER DEVELOPMENT BOARD, TCEQ RECORDS, AND SELECT SUBMITTED DRILLERS REPORT DATABASE BY GEOSEARCH IN OCTOBER, 2016.
3. WATER WELL LOCATIONS PROVIDED BY GEOSEARCH AND MODIFIED BASED ON REVIEW OF WATER WELL REPORT DESCRIPTIONS, GOOGLE EARTH AERIAL IMAGES, AND SITE RECONNAISSANCE PERFORMED BY WCG IN JANUARY 2017.

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WATER WELL LOCATION MAP

HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS

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FIGURE IIIG-A.10

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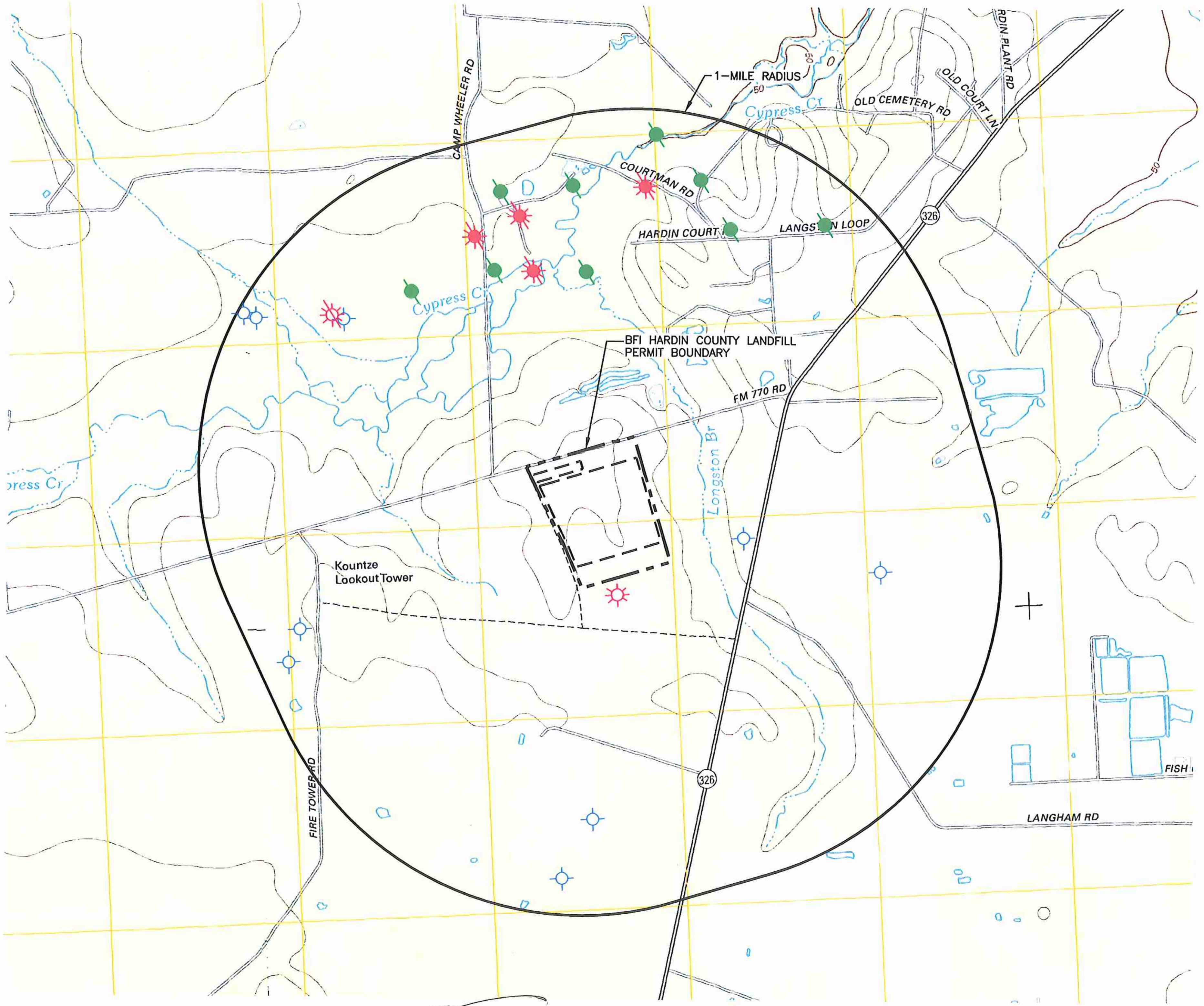

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0 1000 2000
SCALE IN FEET

LEGEND

▬ ▪ ▪ ▬	EXISTING PERMIT BOUNDARY
– – – – – –	PERMITTED LIMITS OF WASTE
⌖	DRY HOLE
●	OIL WELL
☼	GAS WELL
✹	OIL/GAS WELL
☼	PLUGGED GAS WELL

ROAD CLASSIFICATION

Interstate Route	▬	State Route	═
US Route	═	Local Road	═
Ramp	▬	4WD	- - - - - - - -

Interstate Route   US Route   ○ State Route

KOUNTZE SW, TX
2013

KOUNTZE SOUTH, TX
2013

NOTES:

1. ADAPTED FROM USGS 7.5 MINUTE QUADRANGLE TOPOGRAPHIC MAPS (KOUNTZE SOUTH, TX 2013 AND KOUNTZE SW, TX 2013).
2. PERMITTED OIL AND GAS WELL LOCATIONS IN LANDFILL VICINITY OBTAINED FROM TEXAS RAILROAD COMMISSION ONLINE GIS AT http://www.gisp.rrc.texas.gov/gisviewer2/ ON NOVEMBER 8, 2016.

STATE OF TEXAS
AARON K. EVANS
GEOLOGY
11143
LICENSED
PROFESSIONAL GEOSCIENTIST

12-05-17

☐ DRAFT
☒ FOR PERMITTING PURPOSES ONLY
☐ ISSUED FOR CONSTRUCTION

DATE: 03/2017
FILE: 0120-758-11
CAD: IIIG-A.11_OIL/GAS WELL MAP.DWG

DRAWN BY: SRF
DESIGN BY: AE
REVIEWED BY: NT

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

PREPARED FOR
BFI WASTE SYSTEMS OF NORTH AMERICA, LLC

REVISIONS

NO.	DATE	DESCRIPTION
1	11/2017	OWNERSHIP CHANGE

MAJOR PERMIT AMENDMENT
OIL AND GAS WELLS WITHIN ONE-MILE RADIUS

HARDIN COUNTY LANDFILL
HARDIN COUNTY, TEXAS

WWW.WCGRP.COM

FIGURE IIIG-A.11

O:\0120\758\2214B EXPANSION\IIIG\IIIG-A.11_OIL AND GAS WELL MAP.dwg, 11/15/2017 1:40:32 PM, rsellers, 1:2

GEOSEARCH WATER WELL REPORT

STATE OF TEXAS
AARON K. EVANS
GEOLOGY
11143
LICENSED
PROFESSIONAL GEOSCIENTIST

12-05-17

GeoSearch

On time. On target. In touch.™

Texas Water Well Report (Extended Radius)

---

http://www.geo-search.net/QuickMap/index.htm?DataID=Standard0000166196

Click on link above to access the map and satellite view of current property

Target Property:

Hardin County Landfill
Kountze, Hardin County, Texas 77625

Prepared For:

Weaver Consultants Group-Benbrook

Order #: 77143
Job #: 166196
Date: 10/31/2016

TARGET PROPERTY SUMMARY

Hardin County Landfill
Kountze, Hardin County, Texas 77625

USGS Quadrangle: ***Kountze South, TX***
Target Property Geometry: ***Area***

Target Property Longitude(s)/Latitude(s):
(-94.359770, 30.340473), (-94.357538, 30.335028), (-94.352732, 30.336102), (-94.354320, 30.341658), (-94.359770, 30.340473)

County/Parish Covered:
Hardin (TX)

Zipcode(s) Covered:
Kountze TX: 77625

State(s) Covered:
TX

****Target property is located in Radon Zone 3.***
Zone 3 areas have a predicted average indoor radon screening level less than 2 pCi/L (picocuries per liter).

DATABASE FINDINGS SUMMARY

DATABASE	ACRONYM	LOCA-TABLE	UNLOCA-TABLE	SEARCH RADIUS (miles)
FEDERAL				
UNITED STATES GEOLOGICAL SURVEY NATIONAL WATER INFORMATION SYSTEM	NWIS	0	0	1.0000
SUB-TOTAL		**0**	**0**	
STATE (TX)				
SELECT SUBMITTED DRILLERS REPORT DATABASE WELLS	SSDRD	33	0	1.0000
TEXAS COMMISSION ON ENVIRONMENTAL QUALITY WATER WELLS	TCEQ	5	0	1.0000
TEXAS WATER DEVELOPMENT BOARD GROUNDWATER DATABASE	TWDB	6	0	1.0000
WATER UTILITY DATABASE	WUD	0	0	1.0000
SUB-TOTAL		**44**	**0**	
TOTAL		**44**	**0**	

LOCATABLE DATABASE FINDINGS

ACRONYM	SEARCH RADIUS (miles)	TP/AP (0 - 0.02)	1/8 Mile (> TP/AP)	1/4 Mile (> 1/8)	1/2 Mile (> 1/4)	1 Mile (> 1/2)	> 1 Mile	Total
FEDERAL								
NWIS	1.000	0	0	0	0	0	NS	0
SUB-TOTAL		**0**	**0**	**0**	**0**	**0**	**0**	**0**
STATE (TX)								
SSDRD	1.000	0	3	6	8	16	NS	33
TCEQ	1.000	2	0	0	1	2	NS	5
TWDB	1.000	0	0	0	2	4	NS	6
WUD	1.000	0	0	0	0	0	NS	0
SUB-TOTAL		**2**	**3**	**6**	**11**	**22**	**0**	**44**

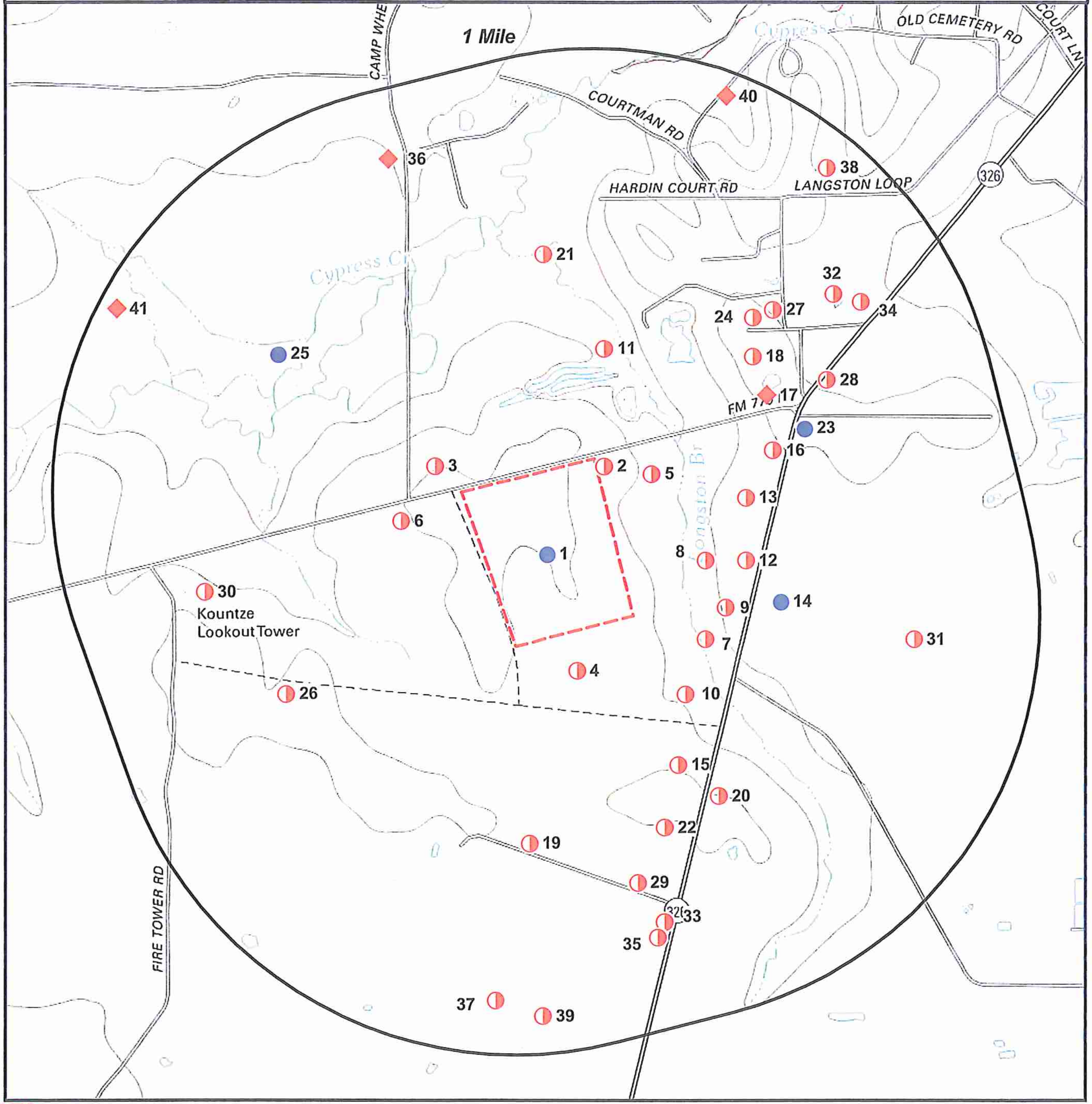
TOTAL		**2**	**3**	**6**	**11**	**22**	**0**	**44**

NOTES:

NS = NOT SEARCHED

TP/AP = TARGET PROPERTY/ADJACENT PROPERTY

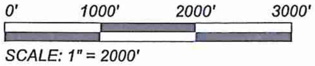
WATER WELL MAP



- ▭ Target Property (TP)
- ● TCEQ
- ◐ SSDRD
- ◆ TWDB

Hardin County Landfill
Kountze, Texas
77625

CONTOUR LINES REPRESENTED IN FEET



REPORT SUMMARY OF LOCATABLE SITES

MAP ID#	DATABASE NAME	SITE ID#	DISTANCE FROM SITE	SITE NAME	ADDRESS	CITY, ZIP CODE	PAGE #
1	TCEQ	TX232601	0.001 SE	JIMMY SONNIER	HWY 326	KOUNTZE, N/A	1
1	TCEQ	TX232600	0.001 SE	HARDIN COUNTY LANDFILL	HWY 326	KOUNTZE, 77625	3
2	SSDRD	TX211340	0.030 E	IESI CORP.	2339 FM 770	KOUNTZE, 77625	5
3	SSDRD	TX211311	0.100 NW	HAL SMITH	550 FM 770	KOUNTZE, 77625	6
4	SSDRD	TX284555	0.100 SE	FAIRWAYS EXPLORATION & PRODUCTION LLC	3.6 MI. SW KOUNTZE ON E SIDE HWY 326	3.6 MI. SW KOUNTZE	7
5	SSDRD	TX155687	0.130 E	ROBERT JOHNSON	4000 HIGHWAY 326 N	KOUNTZE, 77625	8
6	SSDRD	TX127906	0.170 SW	HILTON, STEVEN	5369 FM 770	KOUNTZE, 77625	9
7	SSDRD	TX375993	0.190 SE	BRUCE HOFFER	4324 HIGHWAY 326 NORTH	KOUNTZE, 77625	10
8	SSDRD	TX211350	0.210 E	TODD SAVOY	4156 HWY 326 N.	KOUNTZE, 77625	11
9	SSDRD	TX228904	0.240 SE	ZACHARIAH SHELTON	4244 HWY 326	KOUNTZE, 77625	12
10	SSDRD	TX242617	0.240 SE	MIKE CARBAUGH	4500 HWY 326	KOUNTZE, 77625	13
11	SSDRD	TX211303	0.280 NE	MARION CROW	3068 FM 770	KOUNTZE, 77625	14
12	SSDRD	TX228798	0.310 E	JASON DAUGEREAU	4068 HWY 326	KOUNTZE, 77625	15
13	SSDRD	TX228812	0.350 E	MATT WOODYARD	3882 HWY 326 N.	KOUNTZE, 77625	16
14	TCEQ	TX232598	0.370 E	ERNEST BROWN	ADDRESS NOT LISTED	NOT REPORTED, N/A	17
15	SSDRD	TX228887	0.390 SE	ANGELA FREGIA	4641 HWY 326	KOUNTZE, 77625	19
16	SSDRD	TX392245	0.440 E	CONNIE L JACKSON	3770 HWY 326 N	KOUNTZE, 77625	20
17	TWDB	61-46-103	0.480 E	W.M. MOORE			21
17	TWDB	61-46-102	0.460 E	W.M. MOORE			23
18	SSDRD	TX79360	0.470 NE	BETHEL BAPTIST CHURCH	HIGHWAY 770 @ 326	KOUNTZE, 77625	27
19	SSDRD	TX149219	0.490 S	MARK CROWDER	5080 HIGHWAY 326	KOUNTZE, 77625	28
20	SSDRD	TX330449	0.500 SE	CHRIS BOLSER	LANGHAM LOOP	KOUNTZE	29
21	SSDRD	TX131319	0.520 N	LANGSTON, LEO	2052 LANGSTON LP.	KOUNTZE, 77656	30
22	SSDRD	TX308946	0.530 SE	TERRI LOMBARDO	4886 HWY 326	KOUNTZE, 77625	31
23	TCEQ	TX232597	0.530 E	DANIEL BROWN	ADDRESS NOT LISTED	NOT REPORTED, N/A	32
24	SSDRD	TX197065	0.530 NE	RANDEE WALDEN	2220 LANGSTON LOOP	KOUNTZE, 77625	34
25	TCEQ	TX232599	0.570 NW	BIG SIX DRILLING CO	ADDRESS NOT LISTED	KOUNTZE, N/A	35
26	SSDRD	TX213620	0.570 SW	JIM TUCKER	4089 F.M. 770	KOUNTZE, 77625	38

REPORT SUMMARY OF LOCATABLE SITES

MAP ID#	DATABASE NAME	SITE ID#	DISTANCE FROM SITE	SITE NAME	ADDRESS	CITY, ZIP CODE	PAGE #
27	SSDRD	TX338046	0.580 NE	WILLIS	2185 LANGSTON LOOP	KOUNTZE, 77625	39
28	SSDRD	TX246137	0.610 E	DUIE SUTTERFIELD	3858 HWY 326 N	KOUNTZE, 77625	40
29	SSDRD	TX335190	0.640 SE	JODY OVERTON	5008 HWY. 326 N.	KOUNTZE, 77625	41
30	SSDRD	TX128253	0.680 W	POOL, CARLA & DAN	1 MI E OF SARATOGA	HEMPHILL	42
31	SSDRD	TX228891	0.710 E	SYNERGY OIL + GAS	HWY 326 SOUTH	KOUNTZE, 77625	43
32	SSDRD	TX211390	0.720 NE	JACK CAMPBELL JR.	2415 OLD COURTHOUSE RD.	KOUNTZE, 77625	44
33	SSDRD	TX252466	0.750 SE	MARK CROWDER	5150 HWY 326	KOUNTZE, 77625	45
34	SSDRD	TX133825	0.770 NE	SHELTON	3250 HWY. 326	KOUNTZE, 77625	46
35	SSDRD	TX335292	0.780 SE	MARK CROWDER	5080 HWY. 326 N.	KOUNTZE, 77625	47
36	TWDB	61-46-105	0.830 N	J.J. OLUMS WELL 1			48
36	TWDB	61-46-104	0.840 N	KIRBY LUMBER CO WELL 1			50
37	SSDRD	TX149212	0.880 S	DAVID KNAPP	5062 HIGHWAY 326	KOUNTZE, 77625	52
38	SSDRD	TX42480	0.920 NE	CLEWW LEWIS	1746 OLD COURTHOUSE	KOUNTZE, 77625	53
39	SSDRD	TX131320	0.920 S	WILLIAMS, JEFF	4732 HWY 326 N	KOUNTZE, 77625	54
40	TWDB	61-46-106	0.950 NE	AUDIE TAYLOR WELL 2			55
41	TWDB	61-46-107	0.970 NW	FEE NO.1			57

TEXAS COMMISSION ON ENVIRONMENTAL QUALITY WATER WELLS (TCEQ)

MAP ID# 1 **Distance from Property: 0.00 mi. SE**

ID NUMBER:	**TX232601**
STATE ID :	**61-46-1**
OWNER NAME:	**JIMMY SONNIER**
DATE DRILLED:	**03/12/1996**
DEPTH DRILLED:	**245'**
STATIC LEVEL:	**35'**
WATER USAGE:	**DOMESTIC**
LONGITUDE:	**-94.356251000**
LATITUDE:	**30.338294000**

1 PAGE(S) OF DRILLERS' LOGS

Page # 1 out of 1

Water Well ID: 232601

(60)

Send original copy by certified mail to: TNRCC, P.O. Box 13087, Austin, TX 78711-3087

Please use black ink.

ATTENTION OWNER: *Confidentiality Privilege Notice on Reverse Side*

Stat of Texas
WELL REPORT

Texas Water Well Drillers Advisory Council
P.O. Box 13087
Austin, TX 78711-3087
512-239-0530

1) OWNER Jimmy Sonnier (Name) ADDRESS 140 Stratton (Street or RFD) Beaumont (City) TX (State) (Zip)

2) ADDRESS OF WELL: County Hardin Hwy 326 (Street, RFD or other) Kountze (City) TX (State) (Zip) GRID # 6146-1

3) TYPE OF WORK (Check): ☑ New Well ☐ Deepening ☐ Reconditioning ☐ Plugging

4) PROPOSED USE (Check): ☐ Monitor ☐ Environmental Soil Boring ☑ Domestic ☐ Industrial ☐ Irrigation ☐ Injection ☐ Public Supply ☐ De-watering ☐ Testwell

If Public Supply well, were plans submitted to the TNRCC? ☐ Yes ☐ No

5) N

6) WELL LOG:
Date Drilling:
Started 3-12 1996
Completed 3-12 1996

DIAMETER OF HOLE

Dia. (in.)	From (ft.)	To (ft.)
4 1/2	Surface	245

7) DRILLING METHOD (Check): ☐ Driven ☐ Air Rotary ☑ Mud Rotary ☐ Bored ☐ Air Hammer ☐ Cable Tool ☐ Jetted ☐ Other ______

From (ft.)	To (ft.)	Description and color of formation material
0	3	
3	25	Brown clay
25	54	White sand
54	125	Blue clay
125	245	Blue sand

(Use reverse side if necessary)

8) Borehole Completion (Check): ☐ Open Hole ☑ Straight Wall ☐ Underreamed ☐ Gravel Packed ☐ Other ______
If Gravel Packed give interval ... from ______ ft. to ______ ft.

CASING, BLANK PIPE, AND WELL SCREEN DATA:

Dia. (in.)	[✓ New] or Used	Steel, Plastic, etc. Perf., Slotted, etc. Screen Mfg., if commercial	Setting (ft.) From	Setting (ft.) To	Gage Casting Screen
2	N		225	245	006

9) CEMENTING DATA [Rule 338.44(1)]
Cemented from 0 ft. to 15 ft. No. of sacks used 3
______ ft. to ______ ft. No. of sacks used ______
Method used pour
Cemented by BJ
Distance to septic system field lines or other concentrated contamination 150 ft.
Method of verification of above distance tape

13) TYPE PUMP:
☐ Turbine ☑ Jet ☐ Submersible ☐ Cylinder
☐ Other ______
Depth to pump bowls, cylinder, [✓ jet], etc., 60 ft.

10) SURFACE COMPLETION
☐ Specified Surface Slab Installed [Rule 338.44(2)(A)]
☑ Specified Steel Sleeve Installed [Rule 338.44(3)(A)]
☐ Pitless Adapter Used [Rule 338.44(3)(b)]
☐ Approved Alternative Procedure Used [Rule 338.71]

14) WELL TESTS:
Type test: ☐ Pump ☐ Bailer ☐ Jetted ☐ Estimated
Yield: ______ gpm with ______ ft. drawdown after ______ hrs.

11) WATER LEVEL:
Static level 35 ft. below land surface Date 3-12-96
Artesian flow ______ gpm. Date ______

15) WATER QUALITY:
Did you knowingly penetrate any strata which contained undesirable constituents?
☐ Yes ☑ No If yes, submit "REPORT OF UNDESIRABLE WATER"
Type of water? fresh Depth of strata 120 ft
Was a chemical analysis made? ☐ Yes ☑ No

12) PACKERS: Type Depth

RECEIVED
JUN 1997
TEXAS NATURAL RESOURCE CONSERVATION COMMISSION

I hereby certify that this well was drilled by me (or under my supervision) and that each and all of the statements herein are true to the best of my knowledge and belief. I understand that failure to complete items 1 thru 15 will result in the log(s) being returned for completion and resubmittal.

COMPANY NAME BJ's Water Well Serv. (Type or print) WELL DRILLER'S LICENSE NO. WP 2177

ADDRESS 2150 N. Liam Rd (Street or RFD) Vidor (City) TX (State) 77662 (Zip)

(Signed) B Jones (Licensed Well Driller) (Signed) Robert Ryan (Registered Driller Trainee)

Please attach electric log, chemical analysis, and other pertinent information, if available.

TNRCC-0199 (Rev. 11-01-94)

TNRCC COPY

MAP ID# 1 **Distance from Property: 0.00 mi. SE**

ID NUMBER:	**TX232600**
STATE ID :	**61-46-1**
OWNER NAME:	**HARDIN COUNTY LANDFILL**
DATE DRILLED:	**06/17/1996**
DEPTH DRILLED:	**225'**
STATIC LEVEL:	**50'**
WATER USAGE:	**INDUSTRIAL**
LONGITUDE:	**-94.356251000**
LATITUDE:	**30.338294000**

1 PAGE(S) OF DRILLERS' LOGS

Page # 1 out of 1

Water Well ID: 232600

Send original copy by certified mail to: TNR[illegible], P.O. Box 13087, Austin, TX 78711-3087 *Please use black ink.*

ATTENTION OWNER: *Confidentiality Privilege Notice on Reverse Side*

State of Texas
WELL REPORT

Texas Water Well Drillers Advisory Council
P.O. Box 13087
Austin, TX 78711-3087
512-239-0530

1) OWNER HARDIN County Landfill (Name) ADDRESS PO BOX 2996 (Street or RFD) Kountze (City) TX (State) 77625 (Zip)

2) ADDRESS OF WELL: County HARDIN Hwy 770 (Street, RFD or other) Kountze (City) TX (State) 77625 (Zip) GRID # 61-46-1

3) TYPE OF WORK (Check): ☒ New Well ☐ Deepening ☐ Reconditioning ☐ Plugging

4) PROPOSED USE (Check): ☐ Monitor ☐ Environmental Soil Boring ☐ Domestic ☒ Industrial ☐ Irrigation ☐ Injection ☐ Public Supply ☐ De-watering ☐ Testwell

If Public Supply well, were plans submitted to the TNRCC? ☐ Yes ☐ No

5)

N

6) WELL LOG:
Date Drilling:
Started 6-17 19 96
Completed 6-17 19 96

DIAMETER OF HOLE

Dia. (in.)	From (ft.)	To (ft.)
7	Surface	225

7) DRILLING METHOD (Check): ☐ Driven ☐ Air Rotary ☒ Mud Rotary ☐ Bored ☐ Air Hammer ☐ Cable Tool ☐ Jetted ☐ Other

From (ft.)	To (ft.)	Description and color of formation material
0	38	CLAY
38	70	SAND
70	105	CLAY
105	130	SAND
130	180	ShAle
180	205	Sandy ShAle
205	225	SAND

(Use reverse side if necessary)

8) Borehole Completion (Check): ☐ Open Hole ☒ Straight Wall ☐ Underreamed ☐ Gravel Packed ☐ Other

If Gravel Packed give interval ... from ___ ft. to ___ ft.

CASING, BLANK PIPE, AND WELL SCREEN DATA:

Dia. (in.)	New or Used	Steel, Plastic, etc. Perf., Slotted, etc. Screen Mfg., if commercial	Setting (ft.) From	Setting (ft.) To	Gage Casting Screen
4	N	PVC CASING	0	215	sch 40
4	N	PVC Screen	215	225	.010

9) CEMENTING DATA [Rule 338.44(1)]
Cemented from 0 ft. to 15 ft. No. of sacks used 8
ft. to ___ ft. No. of sacks used ___
Method used Tube
Cemented by Self
Distance to septic system field lines or other concentrated contamination 150 ft.+
Method of verification of above distance Measured

10) SURFACE COMPLETION
☒ Specified Surface Slab Installed [Rule 338.44(2)(A)]
☐ Specified Steel Sleeve Installed [Rule 338.44(3)(A)]
☐ Pitless Adapter Used [Rule 338.44(3)(b)]
☐ Approved Alternative Procedure Used [Rule 338.71]

11) WATER LEVEL:
Static level 50 ft. below land surface Date 6/17/96
Artesian flow ___ gpm. Date ___

12) PACKERS: Type Depth

13) TYPE PUMP:
☐ Turbine ☐ Jet ☒ Submersible ☐ Cylinder
☐ Other
Depth to pump bowls, cylinder, jet, etc., 100 ft.

14) WELL TESTS:
Type test: ☐ Pump ☐ Bailer ☒ Jetted ☐ Estimated
Yield: 40 gpm with 40 ft. drawdown after 1 hrs.

15) WATER QUALITY:
Did you knowingly penetrate any strata which contained undesirable constituents?
☐ Yes ☒ No If yes, submit "REPORT OF UNDESIRABLE WATER"
Type of water? ___ Depth of strata ___
Was a chemical analysis made? ☐ Yes ☒ No

I hereby certify that this well was drilled by me (or under my supervision) and that each and all of the statements herein are true to the best of my knowledge and belief. I understand that failure to complete items 1 thru 15 will result in the log(s) being returned for completion and resubmittal.

COMPANY NAME Holmes Water Well Drilling + Service Co (Type or print) WELL DRILLER'S LICENSE NO. 1301 WI

ADDRESS 290 Kings Row (Street or RFD) Lumberton (City) TX (State) 77657 (Zip)

(Signed) Eugene D. Holmes (Licensed Well Driller) (Signed) ___ (Registered Driller Trainee)

Please attach electric log, chemical analysis, and other pertinent information, if available.

TNRCC-0199 (Rev. 11-01-94)

TNRCC COPY

SUBMITTED DRILLERS REPORT DATABASE (SDRD)

MAP ID# 2 **Distance from Property: 0.03 mi. E**

TRACK #: **211340**

DATE ENTERED: **2010-03-26**

OWNER NAME: **IESI CORP.**

OWNER ADDRESS: **P.O. BOX 1509**
KOUNTZE, TX 77625

COUNTY: **HARDIN**

LATITUDE: **30.341389000** LONGITUDE: **-94.353889000**

WELL LOG:

DRILLING DATE (STARTED): **2008-07-11**

DRILLING DATE (COMPLETED): **2008-07-12**

DEPTH DRILLED: **160'**

WATER LEVEL:

STATIC LEVEL: **32'**

WATER LEVEL DATE: **2008-07-12**

TYPE OF WATER: **FRESH**

TYPE OF WORK:

NEW WELL

PROPOSED USE:

INDUSTRIAL

COMPANY INFORMATION:

COMPANY NAME: **BELLENGER WATER WELL SERVICE**

COMPANY ADDRESS: **P.O. BOX 1012**
SILSBEE, TX 77656

MAP ID# 3

Distance from Property: 0.10 mi. NW

TRACK #: **211311**

DATE ENTERED: **2010-03-25**

OWNER NAME: **HAL SMITH**

OWNER ADDRESS: **10309 PECK RD**
LUMBERTON, TX 77657

COUNTY: **HARDIN**

LATITUDE: **30.341389000** LONGITUDE: **-94.360834000**

WELL LOG:

DRILLING DATE (STARTED): **2008-05-15**

DRILLING DATE (COMPLETED): **2008-05-15**

DEPTH DRILLED: **160'**

WATER LEVEL:

STATIC LEVEL: **16'**

WATER LEVEL DATE: **2008-05-15**

TYPE OF WATER: **FRESH**

TYPE OF WORK:

NEW WELL

PROPOSED USE:

DOMESTIC

COMPANY INFORMATION:

COMPANY NAME: **BELLENGER WATER WELL SERVICE**

COMPANY ADDRESS: **P.O. BOX 1012**
SILSBEE, TX 77656

MAP ID# 4

Distance from Property: 0.10 mi. SE

TRACK #: **284555**
DATE ENTERED: **2012-04-23**
OWNER NAME: **FAIRWAYS EXPLORATION & PRODUCTION**
OWNER ADDRESS: **13430 NORTHWEST FRWY, STE. 800**
HOUSTON, TX 77040
COUNTY: **HARDIN**
LATITUDE: **30.334167000** LONGITUDE: **-94.355001000**

WELL LOG:

DRILLING DATE (STARTED): **2012-02-28**
DRILLING DATE (COMPLETED): **2012-02-28**
DEPTH DRILLED: **257'**

WATER LEVEL:

STATIC LEVEL: **41'**
WATER LEVEL DATE: **2012-02-28**
TYPE OF WATER: **FRESH**

TYPE OF WORK:

NEW WELL

PROPOSED USE:

RIG SUPPLY

COMPANY INFORMATION:

COMPANY NAME: **B & L WATER WELL SERVICE,INC.**
COMPANY ADDRESS: **P.O. BOX 213**
WINNIE, TX 77665

MAP ID# 5 **Distance from Property: 0.13 mi. E**

TRACK #: **155687**

DATE ENTERED: **2008-10-08**

OWNER NAME: **ROBERT JOHNSON**

OWNER ADDRESS: **2310 HIGHWAY 92 N**
SILSBEE, TX 77656

COUNTY: **HARDIN**

LATITUDE: **30.341111000** LONGITUDE: **-94.351945000**

WELL LOG:

DRILLING DATE (STARTED): **2006-12-15**

DRILLING DATE (COMPLETED): **2006-12-15**

DEPTH DRILLED: **360'**

WATER LEVEL:

STATIC LEVEL: **42'**

WATER LEVEL DATE: **2006-12-15**

TYPE OF WATER: **FRESH**

TYPE OF WORK:

NEW WELL

PROPOSED USE:

DOMESTIC

COMPANY INFORMATION:

COMPANY NAME: **BELLENGER WATER WELL SERVICE**

COMPANY ADDRESS: **P. O. BOX 1012**
SILSBEE, TX 77656

MAP ID# 6 **Distance from Property: 0.17 mi. SW**

TRACK #: **127906**
DATE ENTERED: **2007-11-27**
OWNER NAME: **HILTON, STEVEN**
OWNER ADDRESS: **5369 FM 770**
KOUNTZE, TX 77625
COUNTY: **HARDIN**
LATITUDE: **30.339445000** LONGITUDE: **-94.362223000**

WELL LOG:

DRILLING DATE (STARTED): **2007-06-16**
DRILLING DATE (COMPLETED): **2007-06-17**
DEPTH DRILLED: **374'**

WATER LEVEL:

STATIC LEVEL: **42'**
WATER LEVEL DATE: **2007-06-17**
TYPE OF WATER: **FRESH**

TYPE OF WORK:

NEW WELL

PROPOSED USE:

DOMESTIC

COMPANY INFORMATION:

COMPANY NAME: **BELLENGER WATER WELL**
COMPANY ADDRESS: **P O BOX 1012**
SILSBEE, TX 77656

SUBMITTED DRILLERS REPORT DATABASE (SDRD)

MAP ID# 7 **Distance from Property: 0.19 mi. SE**

TRACK #: **375993**

DATE ENTERED: **2014-09-26**

OWNER NAME: **BRUCE HOFFER**

OWNER ADDRESS: **P.O.BOX 1621**
KOUNTZE, TX 77625

COUNTY: **HARDIN**

LATITUDE: **30.335278000** LONGITUDE: **-94.349722000**

WELL LOG:		**WATER LEVEL:**	
DRILLING DATE (STARTED):	**2014-07-30**	STATIC LEVEL:	**29'**
DRILLING DATE (COMPLETED):	**2014-07-31**	WATER LEVEL DATE:	**2014-07-30**
DEPTH DRILLED:	**160'**	TYPE OF WATER:	**FRESH**

TYPE OF WORK:

NEW WELL

PROPOSED USE:

DOMESTIC

COMPANY INFORMATION:

COMPANY NAME: **BELLENGER WATER WELL SERVICE**

COMPANY ADDRESS: **P.O.BOX 1012**
SILSBEE, TX 77656

MAP ID# 8 **Distance from Property: 0.21 mi. E**

TRACK #: **211350**
DATE ENTERED: **2010-03-26**
OWNER NAME: **TODD SAVOY**
OWNER ADDRESS: **P.O. BOX 331**
CHINA, TX 77613
COUNTY: **HARDIN**
LATITUDE: **30.338056000** LONGITUDE: **-94.349722000**

WELL LOG:
DRILLING DATE (STARTED): **2008-08-04**
DRILLING DATE (COMPLETED): **2008-08-05**
DEPTH DRILLED: **485'**

WATER LEVEL:
STATIC LEVEL: **51'**
WATER LEVEL DATE: **2008-08-05**
TYPE OF WATER: **FRESH**

TYPE OF WORK:

NEW WELL

PROPOSED USE:

DOMESTIC

COMPANY INFORMATION:

COMPANY NAME: **BELLENGER WATER WELL SERVICE**
COMPANY ADDRESS: **P.O. BOX 1012**
SILSBEE, TX 77656

MAP ID# 9 **Distance from Property: 0.24 mi. SE**

TRACK #: **228904**
DATE ENTERED: **2010-08-31**
OWNER NAME: **ZACHARIAH SHELTON**
OWNER ADDRESS: **PO BOX 1798**
KOUNTZE, TX 77625
COUNTY: **HARDIN**
LATITUDE: **30.336389000** LONGITUDE: **-94.348889000**

WELL LOG:
DRILLING DATE (STARTED): **2009-05-14**
DRILLING DATE (COMPLETED): **2009-05-15**
DEPTH DRILLED: **350'**

WATER LEVEL:
STATIC LEVEL: **25'**
WATER LEVEL DATE: **2009-05-03**
TYPE OF WATER: **FRESH**

TYPE OF WORK:

NEW WELL

PROPOSED USE:

DOMESTIC

COMPANY INFORMATION:

COMPANY NAME: **BELLENGER WATER WELL SERVICE**
COMPANY ADDRESS: **PO BOX 1012**
SILSBEE, TX 77656

MAP ID# 10 **Distance from Property: 0.24 mi. SE**

TRACK #: **242617**
DATE ENTERED: **2011-01-28**
OWNER NAME: **MIKE CARBAUGH**
OWNER ADDRESS: **900 S. MAIN #928**
LUMBERTON, TX 77657
COUNTY: **HARDIN**
LATITUDE: **30.333334000** LONGITUDE: **-94.350556000**

WELL LOG:

DRILLING DATE (STARTED): **2009-04-28**
DRILLING DATE (COMPLETED): **2009-04-28**
DEPTH DRILLED: **200'**

WATER LEVEL:

STATIC LEVEL: **27'**
WATER LEVEL DATE: **2009-04-29**
TYPE OF WATER: **NOT REPORTED**

TYPE OF WORK:

NEW WELL

PROPOSED USE:

DOMESTIC

COMPANY INFORMATION:

COMPANY NAME: **JONES PUMP & WELL SERVICE**
COMPANY ADDRESS: **235 SHANNON RD**
VIDOR, TX 77662

MAP ID# 11 **Distance from Property: 0.28 mi. NE**

TRACK #: **211303**
DATE ENTERED: **2010-03-25**
OWNER NAME: **MARION CROW**
OWNER ADDRESS: **1409 TALL TIMBERS**
KOUNTZE, TX 77625
COUNTY: **HARDIN**
LATITUDE: **30.345556000** LONGITUDE: **-94.353889000**

WELL LOG:
DRILLING DATE (STARTED): **2008-02-19**
DRILLING DATE (COMPLETED): **2008-02-19**
DEPTH DRILLED: **160'**

WATER LEVEL:
STATIC LEVEL: **39'**
WATER LEVEL DATE: **2008-02-19**
TYPE OF WATER: **FRESH**

TYPE OF WORK:

NEW WELL

PROPOSED USE:

DOMESTIC

COMPANY INFORMATION:

COMPANY NAME: **BELLENGER WATER WELL SERVICE**
COMPANY ADDRESS: **P.O. BOX 1012**
SILSBEE, TX 77656

MAP ID# 12 **Distance from Property: 0.31 mi. E**

TRACK #: **228798**
DATE ENTERED: **2010-08-31**
OWNER NAME: **JASON DAUGEREAU**
OWNER ADDRESS: **PO BOX 1741**
KOUNTZE, TX 77625
COUNTY: **HARDIN**
LATITUDE: **30.338056000** LONGITUDE: **-94.348056000**

WELL LOG:		**WATER LEVEL:**	
DRILLING DATE (STARTED):	**2009-09-10**	STATIC LEVEL:	**31'**
DRILLING DATE (COMPLETED):	**2009-09-11**	WATER LEVEL DATE:	**2009-09-11**
DEPTH DRILLED:	**220'**	TYPE OF WATER:	**FRESH**

TYPE OF WORK:

NEW WELL

PROPOSED USE:

DOMESTIC

COMPANY INFORMATION:

COMPANY NAME: **BELLENGER WATER WELL SERVICE**
COMPANY ADDRESS: **PO BOX 1012**
SILSBEE, TX 77656

MAP ID# 13 **Distance from Property: 0.35 mi. E**

TRACK #: **228812**

DATE ENTERED: **2010-08-31**

OWNER NAME: **MATT WOODYARD**

OWNER ADDRESS: **11651 PEBBLE LN**
LUMBERTON, TX 77657

COUNTY: **HARDIN**

LATITUDE: **30.340278000** LONGITUDE: **-94.348056000**

WELL LOG:		**WATER LEVEL:**	
DRILLING DATE (STARTED):	**2009-08-27**	STATIC LEVEL:	**33'**
DRILLING DATE (COMPLETED):	**2009-08-28**	WATER LEVEL DATE:	**2009-08-28**
DEPTH DRILLED:	**160'**	TYPE OF WATER:	**FRESH**

TYPE OF WORK:

NEW WELL

PROPOSED USE:

DOMESTIC

COMPANY INFORMATION:

COMPANY NAME: **BELLENGER WATER WELL SERVICE**

COMPANY ADDRESS: **PO BOX 1012**
SILSBEE, TX 77656

MAP ID# 14 **Distance from Property: 0.37 mi. E**

ID NUMBER:	**TX232598**
STATE ID :	**61-46-1**
OWNER NAME:	**ERNEST BROWN**
DATE DRILLED:	**08/02/1977**
DEPTH DRILLED:	**120'**
STATIC LEVEL:	**NOT REPORTED**
WATER USAGE:	**DOMESTIC**
LONGITUDE:	**-94.346636000**
LATITUDE:	**30.336615000**

1 PAGE(S) OF DRILLERS' LOGS

Page # 1 out of 1

Water Well ID: 232598

Send original copy by
certified mail to the
Texas Water Development Board
P. O. Box 13087
Austin, Texas 78711

State of Texas

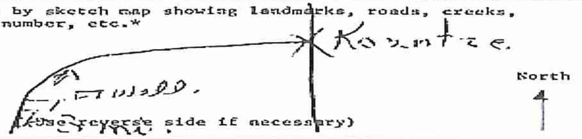
WATER WELL REPORT

For TWDB use only
Well No. 61-42-1D
Located on map 4-95
Received: 7-21
dr

1) OWNER:
Person having well drilled Ernest Brown (Name) Address 326 (Street or RFD) Kountze (City), Tex (State)
Landowner Same (Name) Address ______ (Street or RFD) ______ (City) ______ (State)

2) LOCATION OF WELL:
County Hardin, ______ miles in ______ (N.E., S.W., etc.) direction from ______ (Town)

Locate by sketch map showing landmarks, roads, creeks, hiway number, etc.*



(Use reverse side if necessary)

or

Give legal location with distances and directions from adjacent sections or survey lines.
Labor ______ League ______
Block ______ Survey ______
Abstract No. ______
(NW¼ NE¼ SW¼ SE¼) of Section ______

3) TYPE OF WORK (Check): [✓ New Well] Deepening Reconditioning Plugging

4) PROPOSED USE (Check): [✓ Domestic] Industrial Municipal Irrigation Test Well Other

5) TYPE OF WELL (Check): [✓ Rotary] Driven Dug Cable Jetted Bored

6) WELL LOG:
Diameter of hole 4 in. Depth drilled 120 ft. Depth of completed well 120 ft. Date drilled 8-2-77
All measurements made from 3 ft. above ground level.

From (ft.)	To (ft.)	Description and color of formation material
0	90	clay
90	120	sand

(Use reverse side if necessary)

9) Casing:
Type: Old [✓ New] Steel [✓ Plastic] Other
Cemented from ______ ft. to ______ ft.

Diameter (inches)	Setting From (ft.)	Setting To (ft.)	Gage
2	0	112	40

10) SCREEN:
Type PVC
Perforated [✓ Slotted]

Diameter (inches)	Setting From (ft.)	Setting To (ft.)	Slot Size
2	112	120	16

7) COMPLETION (Check): Straight wall [✓ Gravel packed] Other Under reamed Open Hole

8) WATER LEVEL:
Static level ______ ft. below land surface Date 8-2-77
Artesian pressure ______ lbs. per square inch Date ______
Depth to pump bowls, cylinder, jet, etc., ______ ft. below land surface.

11) WELL TESTS:
Was a pump test made? Yes [✓ No] If yes, by whom?
Yield: ______ gpm with ______ ft. drawdown after ______ hrs.
Bailer test ______ gpm with ______ ft. drawdown after ______ hrs.
Artesian flow ______ gpm
Temperature of water 67°

12) WATER QUALITY:
Was a chemical analysis made? Yes [✓ No]
Did any strata contain undesirable water? Yes [✓ No]
Type of water? good depth of strata 30'

I hereby certify that this well was drilled by me (or under my supervision) and that each and all of the statements herein are true to the best of my knowledge and belief.

NAME Eugene D. Holmes (Type or Print) Water Well Drillers Registration No. 1301
ADDRESS 290 Kings Row (Street or RFD) Lumberton (City) Texas (State)
(Signed) Eugene D. Holmes (Water Well Driller) Holmes Waterwell Sr. (Company Name)

Please attach electric log, chemical analysis, and other pertinent information, if available.

*Additional instructions on reverse side.

TWDBE-WD8

MAP ID# 15 **Distance from Property: 0.39 mi. SE**

TRACK #: **228887**

DATE ENTERED: **2010-08-31**

OWNER NAME: **ANGELA FREGIA**

OWNER ADDRESS: **12085 PELT RD.**
KOUNTZE, TX 77625

COUNTY: **HARDIN**

LATITUDE: **30.330833000** LONGITUDE: **-94.350834000**

WELL LOG:

DRILLING DATE (STARTED): **2009-06-04**

DRILLING DATE (COMPLETED): **2009-06-05**

DEPTH DRILLED: **257'**

WATER LEVEL:

STATIC LEVEL: **36'**

WATER LEVEL DATE: **2009-06-05**

TYPE OF WATER: **FRESH**

TYPE OF WORK:

NEW WELL

PROPOSED USE:

DOMESTIC

COMPANY INFORMATION:

COMPANY NAME: **BELLENGER WATER WELL SERVICE**

COMPANY ADDRESS: **PO BOX 1012**
SILSBEE, TX 77656

MAP ID# 16 **Distance from Property: 0.44 mi. E**

TRACK #: **392245**
DATE ENTERED: **2015-04-10**
OWNER NAME: **CONNIE L JACKSON**
OWNER ADDRESS: **3770 HWY 326 N**
KOUNTZE, TX 77625
COUNTY: **HARDIN**
LATITUDE: **30.341945000** LONGITUDE: **-94.346945000**

WELL LOG:		WATER LEVEL:	
DRILLING DATE (STARTED):	**2015-02-09**	STATIC LEVEL:	**32'**
DRILLING DATE (COMPLETED):	**2015-02-09**	WATER LEVEL DATE:	**2015-02-09**
DEPTH DRILLED:	**360'**	TYPE OF WATER:	**GOOD CLEAR**

TYPE OF WORK:

NEW WELL

PROPOSED USE:

DOMESTIC

COMPANY INFORMATION:

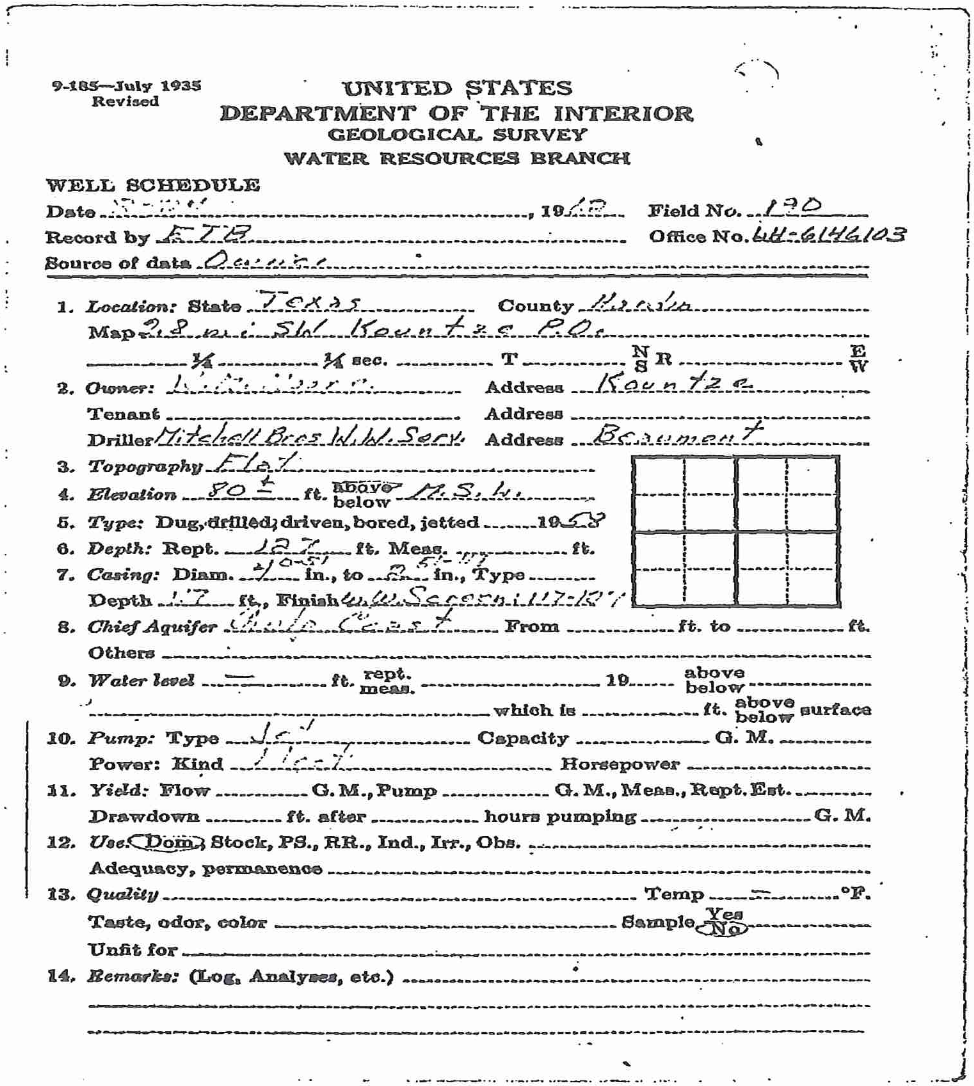
COMPANY NAME: **KENNETH HOLMES DRILLING**
COMPANY ADDRESS: **8313 REDDELL DR**
SILSBEE, TX 77656

MAP ID# 17 **Distance from Property: 0.48 mi. E**

STATE ID:	**61-46-103**
OWNER'S NAME:	**W.M. MOORE**
DATE DRILLED:	**00/00/1958**
DEPTH DRILLED:	**127'**
WATER USAGE:	**DOMESTIC**
LONGITUDE:	**-94.346940000**
LATITUDE:	**30.344170000**
SOURCE:	**TWDB**

Page # 1 out of 1

State ID: 61-46-103



9-185—July 1935
Revised

UNITED STATES
DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY
WATER RESOURCES BRANCH

WELL SCHEDULE

Date [illegible], 19[illegible] Field No. 130
Record by E.T.B. Office No. 64-6146103
Source of data Owner

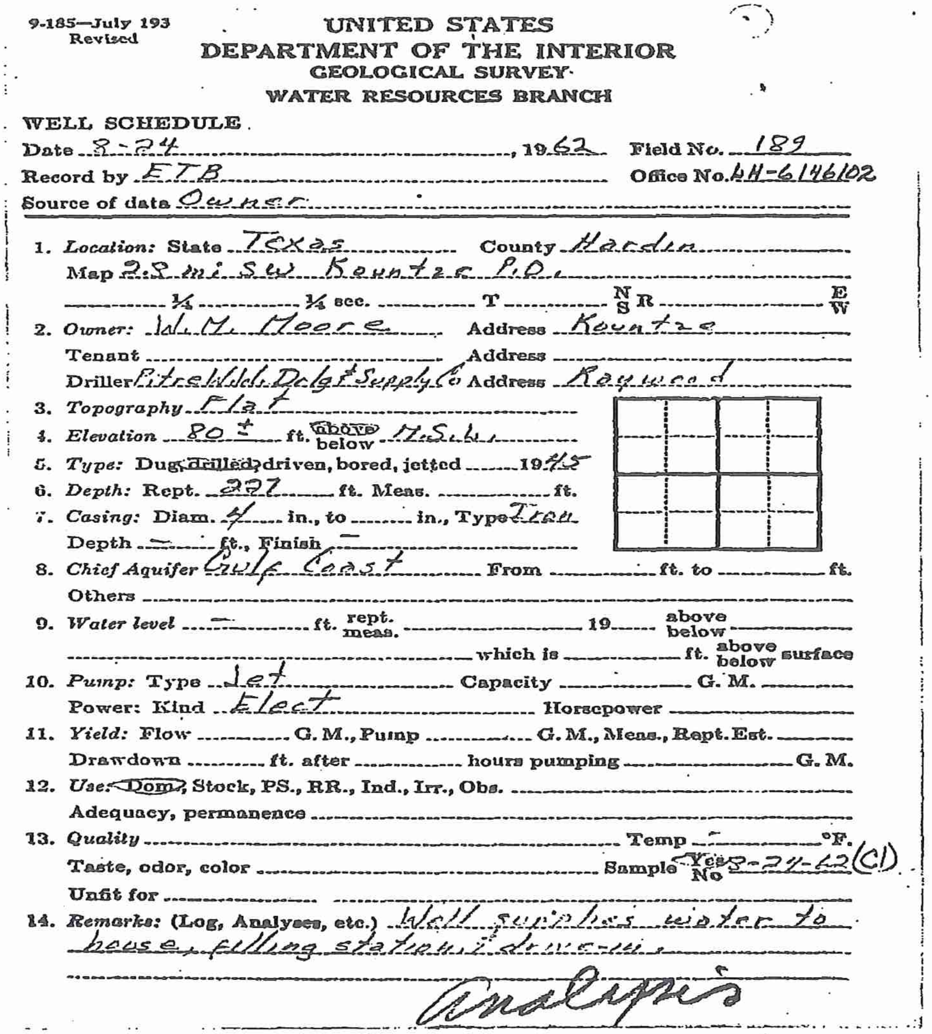
1. *Location:* State Texas County Hardin
   Map 2.8 mi SW Kountze P.O.
   ¼ ¼ sec. T N/S R E/W
2. *Owner:* [illegible] Address Kountze
   Tenant Address
   Driller Mitchell Bros W.W. Serv. Address Beaumont
3. *Topography* Flat
4. *Elevation* 80± ft. above/below M.S.L.
5. *Type:* Dug, [✓ drilled], driven, bored, jetted 1958
6. *Depth:* Rept. 127 ft. Meas. ft.
7. *Casing:* Diam. 4 (4-0-51) in., to 2 (2-51-?) in., Type
   Depth 117 ft., Finish W.W. Screen 117-127
8. *Chief Aquifer* Gulf Coast From ft. to ft.
   Others
9. *Water level* — ft. rept./meas. 19 above/below
   which is ft. above/below surface
10. *Pump:* Type Jet Capacity G. M.
    Power: Kind Elect Horsepower
11. *Yield:* Flow G. M., Pump G. M., Meas., Rept. Est.
    Drawdown ft. after hours pumping G. M.
12. *Use:* [✓ Dom.] Stock, PS., RR., Ind., Irr., Obs.
    Adequacy, permanence
13. *Quality* Temp — °F.
    Taste, odor, color Sample Yes [✓ No]
    Unfit for
14. *Remarks:* (Log, Analyses, etc.)

MAP ID# 17 **Distance from Property: 0.46 mi. E**

STATE ID:	**61-46-102**
OWNER'S NAME:	**W.M. MOORE**
DATE DRILLED:	**00/00/1945**
DEPTH DRILLED:	**227'**
WATER USAGE:	**DOMESTIC**
LONGITUDE:	**-94.347220000**
LATITUDE:	**30.343890000**
SOURCE:	**TWDB**

Page # 1 out of 3

State ID: 61-46-102



9-185—July 193
Revised

UNITED STATES
DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY
WATER RESOURCES BRANCH

WELL SCHEDULE

Date 8-24, 1962 Field No. 189

Record by ETB Office No. LH-6146102

Source of data Owner

1. *Location:* State Texas County Hardin
   Map 2.5 mi SW Kountze P.O.
   ¼ ¼ sec. T N/S R E/W
2. *Owner:* W. M. Moore Address Kountze
   Tenant Address
   Driller Pitre Wl. Drlg & Supply Co Address Raywood
3. *Topography* Flat
4. *Elevation* 80 ± ft. [✓ above] below M.S.L.
5. *Type:* Dug, [✓ drilled], driven, bored, jetted 1945
6. *Depth:* Rept. 227 ft. Meas. ft.
7. *Casing:* Diam. 4 in., to in., Type Iron
   Depth – ft., Finish –
8. *Chief Aquifer* Gulf Coast From ft. to ft.
   Others
9. *Water level* – ft. rept. meas. 19 above below
   which is ft. above below surface
10. *Pump:* Type Jet Capacity G. M.
    Power: Kind Elect Horsepower
11. *Yield:* Flow G. M., Pump G. M., Meas., Rept. Est.
    Drawdown ft. after hours pumping G. M.
12. *Use:* [✓ Dom.], Stock, PS., RR., Ind., Irr., Obs.
    Adequacy, permanence
13. *Quality* Temp – °F.
    Taste, odor, color Sample [✓ Yes] No 8-24-62 (Cl)
    Unfit for
14. *Remarks:* (Log, Analyses, etc.) Well supplies water to house, filling station & drive-in.

Analysis

Typewrite (Black ribbon) or Print Plainly
(soft pencil or black ink)
Do not use ball point pen

Texas Department of Health Laboratories
1100 West 49th Street
Austin, Texas 78756

TDWR ONLY

Organization No. ________ Lab No. 02

Work No. ________

CHEMICAL WATER ANALYSIS REPORT

County 100 HARDIN

State Well No. 61-46-102

________ Well No. ________

Date Collected 08-24-62

Send report to:

Data Collection and Evaluation Section
Texas Department of Water Resources
P.O. Box 13087
Austin, Texas 78711

Location 2.8 mi SW Kountze PO Sample No. ☐ By ETB-USGS

Source (type of well) ________ Owner W. M. Moore

Date Drilled ________ Depth 227 ft. WBF Gulf Coast Sands

Producing intervals ________ Water level ________ ft. Sample depth ☐☐☐☐ ft.

Sampled after pumping ________ hrs. Yield ________ GPM meas. est. Temperature ☐☐☐ °F ☐☐☐ °C

Point of collection ________ Appearance ☐ clear ☐ turbid ☐ colored ☐ other

Use Dom. Remarks ________

(FOR LABORATORY USE ONLY)

CHEMICAL ANALYSIS

KEY PUNCHED

Laboratory No. ________ Date Received ________ Date Reported ________

		MG/L	ME/L
Silica	00955		
Calcium	00915		
Magnesium	00925		
Sodium	00929		
Total			
☐ Potassium	00937		
[3] ☐ Manganese	01055		
☐ Boron	01022		
[3] ☐ Total Iron	01045	0	

%Na ________

SAR ________

RSC 0.0

☐ (other) ________ MG/L

Specific Conductance (micromhos/cm³) · 00095 · 488

Diluted Conductance (micromhos/cm³) x

"☐" items will be analyzed if checked.

		MG/L	ME/L
Carbonate	00445	0	
Bicarbonate	00440	188	
Sulfate	00945	5	
Chloride	00940	59	
Fluoride	00951		
Nitrate	71850		
pH	00403	7.5	
Total			
[1] Dissolved Solids (residue at 180° C)	70300		
Phenolphthalein Alkalinity as $CaCO_3$	00415		
Total Alkalinity as $CaCO_3$	00410		
Total Hardness as $CaCO_3$	00900	178	
[2] Nitrogen Cycle			
Ammonia - N	00610		
Nitrite - N	00615		
Nitrate - N	00620		
Organic Nitrogen	00605		

Analyst ________ Checked By ________

---

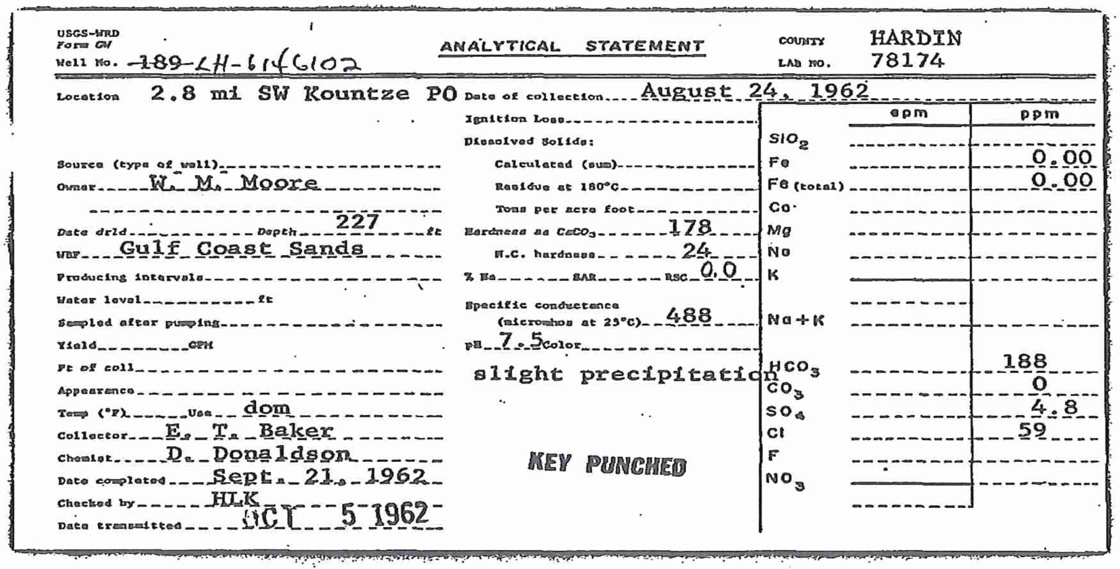
[1] The bicarbonate reported in this analysis can be converted by computation (multiplying by 0.4917) to an equivalent amount of carbonate, and the carbonate figure used in the computation of dissolved solids.

[2] Nitrogen cycle requires separate sample.

[3] Total Iron and Manganese require separate sample.

TDWR-0148 (Rev. 1-8-80)

State ID: 61-46-102



USGS-WRD
Form GW
Well No. ~~189~~-LH-6146102

ANALYTICAL STATEMENT

COUNTY HARDIN
LAB NO. 78174

Location 2.8 mi SW Kountze PO

Source (type of well)
Owner W. M. Moore
Date drld Depth 227 ft
WBF Gulf Coast Sands
Producing intervals
Water level ft
Sampled after pumping
Yield GPM
Pt of coll
Appearance
Temp (°F) Use dom
Collector E. T. Baker
Chemist D. Donaldson
Date completed Sept. 21, 1962
Checked by HLK
Date transmitted OCT 5 1962

Date of collection August 24, 1962
Ignition Loss
Dissolved Solids:
Calculated (sum)
Residue at 180°C
Tons per acre foot
Hardness as $CaCO_3$ 178
N.C. hardness 24
% Na SAR RSC 0.0
Specific conductance (micromhos at 25°C) 488
pH 7.5 Color

slight precipitation

KEY PUNCHED

	epm	ppm
SiO_2		
Fe		0.00
Fe (total)		0.00
Ca		
Mg		
Na		
K		
Na+K		
HCO_3		188
CO_3		0
SO_4		4.8
Cl		59
F		
NO_3		

SUBMITTED DRILLERS REPORT DATABASE (SDRD)

MAP ID# 18 **Distance from Property: 0.47 mi. NE**

TRACK #: **79360**

DATE ENTERED: **2006-03-28**

OWNER NAME: **BETHEL BAPTIST CHURCH**

OWNER ADDRESS: **1230 W. FEAGIN**
KOUNTZE, TX 77625

COUNTY: **HARDIN**

LATITUDE: **30.345278000** LONGITUDE: **-94.347778000**

WELL LOG:

DRILLING DATE (STARTED): **2004-02-25**

DRILLING DATE (COMPLETED): **2004-02-25**

DEPTH DRILLED: **180'**

WATER LEVEL:

STATIC LEVEL: **48'**

WATER LEVEL DATE: **2004-02-25**

TYPE OF WATER: **FRESH**

TYPE OF WORK:

NEW WELL

PROPOSED USE:

DOMESTIC

COMPANY INFORMATION:

COMPANY NAME: **BELLENGER WATER WELL**

COMPANY ADDRESS: **P. O. BOX 1012**
SILSBEE, TX 77656

MAP ID# 19 **Distance from Property: 0.49 mi. S**

TRACK #: **149219**
DATE ENTERED: **2008-08-11**
OWNER NAME: **MARK CROWDER**
OWNER ADDRESS: **5080 HIGHWAY 326**
KOUNTZE, TX 77625
COUNTY: **HARDIN**
LATITUDE: **30.328055000** LONGITUDE: **-94.356945000**

WELL LOG:		**WATER LEVEL:**	
DRILLING DATE (STARTED):	**2006-01-06**	STATIC LEVEL:	**45'**
DRILLING DATE (COMPLETED):	**2006-01-07**	WATER LEVEL DATE:	**2006-01-07**
DEPTH DRILLED:	**180'**	TYPE OF WATER:	**FRESH**

TYPE OF WORK:

NEW WELL

PROPOSED USE:

DOMESTIC

COMPANY INFORMATION:

COMPANY NAME: **BELLENGER WATER WELL**
COMPANY ADDRESS: **P.O. BOX 1012**
SILSBEE, TX 77656

MAP ID# 20 **Distance from Property: 0.50 mi. SE**

TRACK #: **330449**
DATE ENTERED: **2013-08-05**
OWNER NAME: **CHRIS BOLSER**
OWNER ADDRESS: **LANGHAM LOOP**
KOUNTZE, TX 77625
COUNTY: **HARDIN**
LATITUDE: **30.329722000** LONGITUDE: **-94.349167000**

WELL LOG:

DRILLING DATE (STARTED): **2005-02-25**
DRILLING DATE (COMPLETED): **2005-02-25**
DEPTH DRILLED: **392'**

WATER LEVEL:

STATIC LEVEL: **44'**
WATER LEVEL DATE: **2005-02-25**
TYPE OF WATER: **FRESH**

TYPE OF WORK:

NEW WELL

PROPOSED USE:

DOMESTIC

COMPANY INFORMATION:

COMPANY NAME: **BELLENGER WATER WELL**
COMPANY ADDRESS: **NOT REPORTED**
NOT REPORTED

MAP ID# 21 **Distance from Property: 0.52 mi. N**

TRACK #: **131319**

DATE ENTERED: **2008-01-10**

OWNER NAME: **LANGSTON, LEO**

OWNER ADDRESS: **2052 LANGSTON LP.**
KOUNTZE, TX 77656

COUNTY: **HARDIN**

LATITUDE: **30.348889000** LONGITUDE: **-94.356389000**

WELL LOG:		**WATER LEVEL:**	
DRILLING DATE (STARTED):	**2006-03-21**	STATIC LEVEL:	**53'**
DRILLING DATE (COMPLETED):	**2006-03-22**	WATER LEVEL DATE:	**2006-03-22**
DEPTH DRILLED:	**180'**	TYPE OF WATER:	**FRESH**

TYPE OF WORK:

NEW WELL

PROPOSED USE:

DOMESTIC

COMPANY INFORMATION:

COMPANY NAME: **BELLENGER WATER WELL**

COMPANY ADDRESS: **PO BOX 1012**
SILSBEE, TX 77656

SUBMITTED DRILLERS REPORT DATABASE (SDRD)

MAP ID# 22 **Distance from Property: 0.53 mi. SE**

TRACK #: **308946**

DATE ENTERED: **2013-01-15**

OWNER NAME: **TERRI LOMBARDO**

OWNER ADDRESS: **4886 HWY 326**
KOUNTZE, TX 77625

COUNTY: **HARDIN**

LATITUDE: **30.328611000** LONGITUDE: **-94.351389000**

WELL LOG:		**WATER LEVEL:**	
DRILLING DATE (STARTED):	**2012-09-05**	STATIC LEVEL:	**48'**
DRILLING DATE (COMPLETED):	**2012-09-05**	WATER LEVEL DATE:	**2012-09-05**
DEPTH DRILLED:	**500'**	TYPE OF WATER:	**FRESH**

TYPE OF WORK:

NEW WELL

PROPOSED USE:

DOMESTIC

COMPANY INFORMATION:

COMPANY NAME: **JONES WATER WELL SERVICE**

COMPANY ADDRESS: **205 SHANNON RD**
VIDOR, TX 77662

MAP ID# 23 **Distance from Property: 0.53 mi. E**

ID NUMBER:	**TX232597**
STATE ID :	**61-46-1**
OWNER NAME:	**DANIEL BROWN**
DATE DRILLED:	**08/02/1977**
DEPTH DRILLED:	**120'**
STATIC LEVEL:	**NOT REPORTED**
WATER USAGE:	**DOMESTIC**
LONGITUDE:	**-94.345664000**
LATITUDE:	**30.342739000**

1 PAGE(S) OF DRILLERS' LOGS

Page # 1 out of 1

Water Well ID: 232597

Send original copy by
certified mail to the
Texas Water Development Board
P. O. Box 13087
Austin, Texas 78711

State of Texas

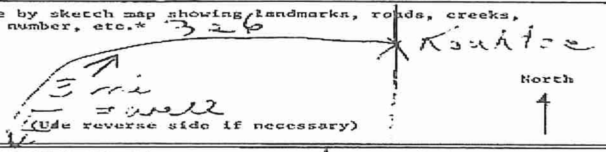
WATER WELL REPORT

For TWDB use only
Well No. 61-46-10
Located on map
Received:

1) OWNER:
Person having well drilled Daniel Brown (Name) Address 326 (Street or RFD) Kountze (City) Tex (State)
Landowner Same (Name) Address (Street or RFD) (City) (State)

2) LOCATION OF WELL:
County Hardin, ______ miles in ______ (N.E., S.W., etc.) direction from ______ (Town)

Locate by sketch map showing landmarks, roads, creeks, hiway number, etc.*



(Use reverse side if necessary)

or

Give legal location with distances and directions from adjacent sections or survey lines.
Labor ______ League ______
Block ______ Survey ______
Abstract No. ______
(NW¼ NE¼ SW¼ SE¼) of Section ______

3) TYPE OF WORK (Check):
[✓ New Well] Deepening
Reconditioning Plugging

4) PROPOSED USE (Check):
[✓ Domestic] Industrial Municipal
Irrigation Test Well Other

5) TYPE OF WELL (Check):
[✓ Rotary] Driven Dug
Cable Jetted Bored

6) WELL LOG:
Diameter of hole 1/ in. Depth drilled 120 ft. Depth of completed well 120 ft. Date drilled 8/2/77
All measurements made from 3 ft. above ground level.

From (ft.)	To (ft.)	Description and color of formation material
0	90	Clay
90	120	SAND

(Use reverse side if necessary)

9) Casing:
Type: Old [✓ New] Steel [✓ Plastic] Other
Cemented from ______ ft. to ______ ft.

Diameter (inches)	Setting From (ft.)	Setting To (ft.)	Gage
2	0	112	40

10) SCREEN:
Type PVC
Perforated [✓ Slotted]

Diameter (inches)	Setting From (ft.)	Setting To (ft.)	Slot Size
2	112	120	.10

7) COMPLETION (Check):
Straight wall [✓ Gravel packed] Other
Under reamed Open Hole

8) WATER LEVEL:
Static level ______ ft. below land surface Date 9/4/77
Artesian pressure ______ lbs. per square inch Date ______
Depth to pump bowls, cylinder, jet, etc., ______ ft. below land surface.

11) WELL TESTS:
Was a pump test made? Yes [✓ No] If yes, by whom?
Yield: ______ gpm with ______ ft. drawdown after ______ hrs.
Bailer test ______ gpm with ______ ft. drawdown after ______ hrs.
Artesian flow ______ gpm
Temperature of water 61°

12) WATER QUALITY:
Was a chemical analysis made? Yes [✓ No]
Did any strata contain undesirable water? Yes [✓ No]
Type of water? Good depth of strata 30'

I hereby certify that this well was drilled by me (or under my supervision) and that each and all of the statements herein are true to the best of my knowledge and belief.

NAME Eugene D. Holmes (Type or Print) Water Well Drillers Registration No. 1301
ADDRESS 220 K[illegible] Ro[illegible] (Street or RFD) Lumberton (City) Texas (State)
(Signed) Eugene D. Holmes (Water Well Driller) Holmes Water Well Co. (Company Name)

Please attach electric log, chemical analysis, and other pertinent information, if available.

*Additional instructions on reverse side.

TWDBE-WD-8

SUBMITTED DRILLERS REPORT DATABASE (SDRD)

MAP ID# 24 **Distance from Property: 0.53 mi. NE**

TRACK #: **197065**
DATE ENTERED: **2009-10-22**
OWNER NAME: **RANDEE WALDEN**
OWNER ADDRESS: **2220 LANGSTON LOOP**
KOUNTZE, TX 77625
COUNTY: **HARDIN**
LATITUDE: **30.346667000** LONGITUDE: **-94.347778000**

WELL LOG:

DRILLING DATE (STARTED): **2009-02-25**
DRILLING DATE (COMPLETED): **2009-02-25**
DEPTH DRILLED: **440'**

WATER LEVEL:

STATIC LEVEL: **52'**
WATER LEVEL DATE: **2009-02-25**
TYPE OF WATER: **NOT REPORTED**

TYPE OF WORK:

NEW WELL

PROPOSED USE:

DOMESTIC

COMPANY INFORMATION:

COMPANY NAME: **WEST WATER WELL**
COMPANY ADDRESS: **P.O. BOX 82**
BATSON, TX 77519

e from Property: 0.57 mi. NW

)

)RILLING CO

:0

IAL

5000

5000

GS

Send original copy by certified mail to the
Texas Department of Water Resources
P. O. Box 13087
Austin, Texas 78711

State of Texas
WATER WELL REPORT

ATTENTION OWNER: *Confidentiality Privilege Notice on Reverse Side*

For TDWR use only
Well No. 61-46-1F
Located on map
Received:

1) OWNER Big Six Drilling Co. (Name) Address 1500 San Felipe Houston Tex 77063 (Street or RFD) (City) (State) (Zip)

2) LOCATION OF WELL: County Hardin Co. miles in West (N.E., S.W., etc.) direction from Kountze Texas (Town)

Driller must complete the legal description to the right with distance and direction from two intersecting section or survey lines, or he must locate and identify the well on an official Quarter- or Half-Scale Texas County General Highway Map and attach the map to this form.

☐ Legal description:
Section No. ______ Block No. ______ Township ______
Abstract No. ______ Survey Name ______
Distance and direction from two intersecting section or survey lines ______

☑ See attached map.

3) TYPE OF WORK (Check):
☑ New Well ☐ Deepening
☐ Reconditioning ☐ Plugging

4) PROPOSED USE (Check):
☐ Domestic ☑ Industrial ☐ Public Supply
☐ Irrigation ☐ Test Well ☐ Other

5) DRILLING METHOD (Check):
☒ ~~Mud~~ Rotary ☐ Air Hammer ☐ Driven ☒ ~~Bored~~
☐ Air Rotary ☐ Cable Tool ☑ ~~Jetted~~ ☐ Other

6) WELL LOG:

Date drilled 3-23-80

DIAMETER OF HOLE Dia. (in.)	From (ft.)	To (ft.)
4"	Surface	130

7) BOREHOLE COMPLETION:
☐ Open Hole ☑ ~~Straight~~ Wall ☐ Underreamed
☐ Gravel Packed ☐ Other
If Gravel Packed give interval . . . from ______ ft. to ______ ft.

From (ft.)	To (ft.)	Description and color of formation material
0	18	Clay
18	130	Sand.

(Use reverse side if necessary)

8) CASING, BLANK PIPE, AND WELL SCREEN DATA:

Dia. (in.)	New or Used	Steel, Plastic, etc. Perf., Slotted, etc. Screen Mgf., if commercial	Setting (ft.) From	To	Gage Casing Screen
~~2"~~			~~0~~	~~100~~	
3"	N.	Steel Shot Hole	~~100~~		
			0	100	
		perf	100	130	

CEMENTING DATA
Cemented from ______ ft. to ______ ft.
Method used ______
Cemented by ______ (Company or Individual)

9) WATER LEVEL:
Static level 14 ft. below land surface Date ______
Artesian flow ______ gpm. Date ______

10) PACKERS: Type Depth

11) TYPE PUMP:
☐ Turbine ☐ Jet ☐ Submersible ☐ Cylinder
☐ Other ______
Depth to pump bowls, cylinder, jet, etc., ______ ft.

12) WELL TESTS:
☐ Type Test: ☐ Pump ☐ Bailer ☐ Jetted ☐ Estimated
Yield: ______ gpm with ______ ft. drawdown after ______ hrs.

13) WATER QUALITY:
Did you knowingly penetrate any strata which contained undesirable water? ☐ Yes ☑ No
If yes, submit "REPORT OF UNDESIRABLE WATER"
Type of water? Fresh Depth of strata 12 ft.
Was a chemical analysis made? ☐ Yes ☑ No

I hereby certify that this well was drilled by me (or under my supervision) and that each and all of the statements herein are true to the best of my knowledge and belief.

NAME Samuel C. Gore (Type or Print) Water Well Drillers Registration No. 91

ADDRESS S. R. 1 Box 1945 (Street or RFD) Silsbee (City) Texas (State) [illegible] (Zip)

RECEIVED
JUL 20 1981

(Signed) Samuel C. Gore (Water Well Driller) Sam Gore Water Well Serv. (Company Name)

Please attach electric log, chemical analysis, and other pertinent information, if available.

TDWR-0392 (Rev. 1-12-79)

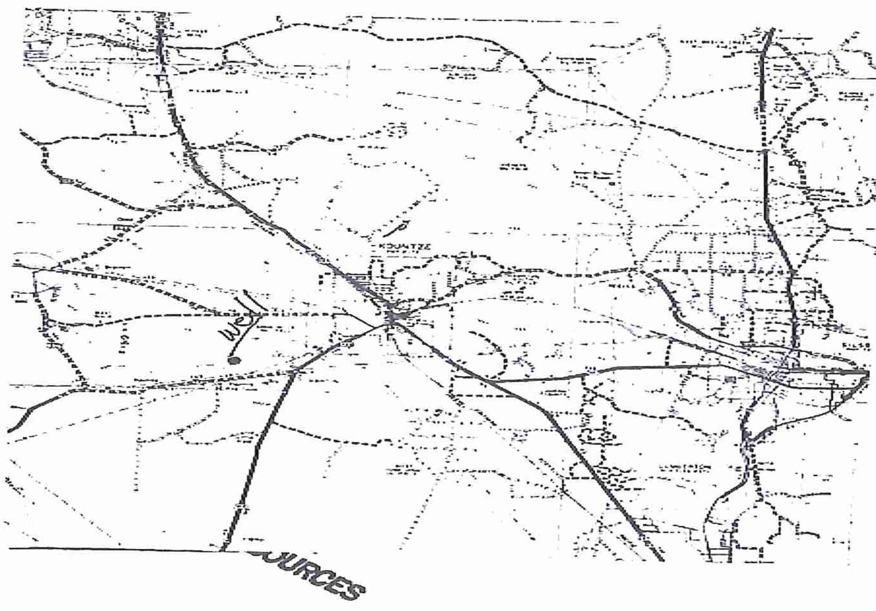
DEPARTMENT OF WATER RESOURCES COPY

IMPORTANT NOTICE FOR PERSONS HAVING WELLS DRILLED CONCERNING PRIVILEGE OF CONFIDENTIALITY

The Water Well Drillers Board and the Department of Water Resources are concerned that some persons having water wells drilled may not be aware of the confidentiality privilege provision of Section 5 of the Water Well Drillers Act. Section 5, the Reporting of Well Logs, reads as follows:

> "Every registered water well driller drilling, deepening, or otherwise altering a water well within this State shall make and keep, or cause to be made and kept, a legible and accurate well log, and within sixty (60) days from the completion or cessation of drilling, deepening or otherwise altering such a water well, shall deliver or transmit by certified mail a copy of such well log to the Commission, and the owner thereof or the person having had such well drilled. The well log required herein shall at the request in writing to the Commission, by certified mail, by the owner or the person having such well drilled be held as confidential matter and not made of public record."

The last sentence specifies the means whereby you can, if you wish, assure that logs of your wells will be kept confidential. Please note that the term "Commission" in the above-quoted section and elsewhere in the Water Well Drillers Act now properly means the Texas Department of Water Resources (P. O. Box 13087; Austin, Texas 78711).



MAP ID# 26 **Distance from Property: 0.57 mi. SW**

TRACK #: **213620**
DATE ENTERED: **2010-04-21**
OWNER NAME: **JIM TUCKER**
OWNER ADDRESS: **4089 F.M. 770**
KOUNTZE, TX 77625
COUNTY: **HARDIN**
LATITUDE: **30.333334000** LONGITUDE: **-94.366944000**

WELL LOG:		**WATER LEVEL:**	
DRILLING DATE (STARTED):	**2010-03-01**	STATIC LEVEL:	**23'**
DRILLING DATE (COMPLETED):	**2010-03-06**	WATER LEVEL DATE:	**2010-03-01**
DEPTH DRILLED:	**240'**	TYPE OF WATER:	**FRESH**

TYPE OF WORK:

NEW WELL

PROPOSED USE:

DOMESTIC

COMPANY INFORMATION:

COMPANY NAME: **W.M. JONES WATERWELL**
COMPANY ADDRESS: **1555 EVANGELINE DR.**
VIDOR, TX 77662

MAP ID# 27 Distance from Property: 0.58 mi. NE

TRACK #: **338046**
DATE ENTERED: **2013-08-22**
OWNER NAME: **WILLIS**
OWNER ADDRESS: **2184 LANGSTON LOOP**
KOONTZE, TX 77625
COUNTY: **HARDIN**
LATITUDE: **30.346945000** LONGITUDE: **-94.346945000**

WELL LOG:

DRILLING DATE (STARTED): **2005-10-17**
DRILLING DATE (COMPLETED): **2005-10-17**
DEPTH DRILLED: **190'**

WATER LEVEL:

STATIC LEVEL: **NOT REPORTED**
WATER LEVEL DATE: **2005-10-17**
TYPE OF WATER: **NOT REPORTED**

TYPE OF WORK:

NEW WELL

PROPOSED USE:

DOMESTIC

COMPANY INFORMATION:

COMPANY NAME: **NOT REPORTED**
COMPANY ADDRESS: **NOT REPORTED**
NOT REPORTED

MAP ID# 28 **Distance from Property: 0.61 mi. E**

TRACK #: **246137**
DATE ENTERED: **2011-03-09**
OWNER NAME: **DUIE SUTTERFIELD**
OWNER ADDRESS: **1025 WILSON**
PORT NECHES, TX 77651
COUNTY: **HARDIN**
LATITUDE: **30.344445000** LONGITUDE: **-94.344722000**

WELL LOG:
DRILLING DATE (STARTED): **2007-11-28**
DRILLING DATE (COMPLETED): **2007-11-29**
DEPTH DRILLED: **140'**

WATER LEVEL:
STATIC LEVEL: **36'**
WATER LEVEL DATE: **2007-11-29**
TYPE OF WATER: **FRESH**

TYPE OF WORK:

NEW WELL

PROPOSED USE:

DOMESTIC

COMPANY INFORMATION:

COMPANY NAME: **JONES WATER WELL SERVICE**
COMPANY ADDRESS: **205 SHANNON RD**
VIDOR, TX 77662

MAP ID# 29 **Distance from Property: 0.64 mi. SE**

TRACK #: **335190**

DATE ENTERED: **2013-08-15**

OWNER NAME: **JODY OVERTON**

OWNER ADDRESS: **2099 DOWLEN RD.**
BEAUMONT, TX 77706

COUNTY: **HARDIN**

LATITUDE: **30.326667000** LONGITUDE: **-94.352501000**

WELL LOG:		**WATER LEVEL:**	
DRILLING DATE (STARTED):	**2010-12-22**	STATIC LEVEL:	**40'**
DRILLING DATE (COMPLETED):	**2010-12-22**	WATER LEVEL DATE:	**2010-12-22**
DEPTH DRILLED:	**160'**	TYPE OF WATER:	**FRESH**

TYPE OF WORK:

NEW WELL

PROPOSED USE:

DOMESTIC

COMPANY INFORMATION:

COMPANY NAME: **NOT REPORTED**

COMPANY ADDRESS: **NOT REPORTED**
NOT REPORTED

MAP ID# 30 **Distance from Property: 0.68 mi. W**

TRACK #: **128253**

DATE ENTERED: **2007-11-30**

OWNER NAME: **POOL, CARLA & DAN**

OWNER ADDRESS: **8941 PRINCETON**
LUMBERTON, TX 77657

COUNTY: **HARDIN**

LATITUDE: **30.336945000** LONGITUDE: **-94.370278000**

WELL LOG:

DRILLING DATE (STARTED): **2007-06-26**

DRILLING DATE (COMPLETED): **2007-06-26**

DEPTH DRILLED: **250'**

WATER LEVEL:

STATIC LEVEL: **25'**

WATER LEVEL DATE: **2007-06-26**

TYPE OF WATER: **FRESH**

TYPE OF WORK:

NEW WELL

PROPOSED USE:

DOMESTIC

COMPANY INFORMATION:

COMPANY NAME: **BJ'S WATER WELL DRILLING**

COMPANY ADDRESS: **14 WALKER CREEK RD.**
JASPER, TX 75951

SUBMITTED DRILLERS REPORT DATABASE (SDRD)

MAP ID# 30 **Distance from Property: 0.68 mi. W**

TRACK #: **128253**

DATE ENTERED: **2007-11-30**

OWNER NAME: **POOL, CARLA & DAN**

OWNER ADDRESS: **8941 PRINCETON**
LUMBERTON, TX 77657

COUNTY: **HARDIN**

LATITUDE: **30.336945000** LONGITUDE: **-94.370278000**

WELL LOG:		**WATER LEVEL:**	
DRILLING DATE (STARTED):	**2007-06-26**	STATIC LEVEL:	**25'**
DRILLING DATE (COMPLETED):	**2007-06-26**	WATER LEVEL DATE:	**2007-06-26**
DEPTH DRILLED:	**250'**	TYPE OF WATER:	**FRESH**

TYPE OF WORK:

NEW WELL

PROPOSED USE:

DOMESTIC

COMPANY INFORMATION:

COMPANY NAME: **BJ'S WATER WELL DRILLING**

COMPANY ADDRESS: **14 WALKER CREEK RD.**
JASPER, TX 75951

MAP ID# 31 **Distance from Property: 0.71 mi. E**

TRACK #: **228891**

DATE ENTERED: **2010-08-31**

OWNER NAME: **SYNERGY OIL + GAS**

OWNER ADDRESS: **9821 KATY FREEWAY SUITE 805**
HOUSTON, TX 77024

COUNTY: **HARDIN**

LATITUDE: **30.335278000** LONGITUDE: **-94.341111000**

WELL LOG:	**WATER LEVEL:**
DRILLING DATE (STARTED): **2009-05-31**	STATIC LEVEL: **31'**
DRILLING DATE (COMPLETED): **2009-05-31**	WATER LEVEL DATE: **2009-05-31**
DEPTH DRILLED: **185'**	TYPE OF WATER: **FRESH**

TYPE OF WORK:

NEW WELL

PROPOSED USE:

RIG SUPPLY

COMPANY INFORMATION:

COMPANY NAME: **BELLENGER WATER WELL SERVICE**

COMPANY ADDRESS: **PO BOX 1012**
SILSBEE, TX 77656

MAP ID# 32

Distance from Property: 0.72 mi. NE

TRACK #: **211390**
DATE ENTERED: **2010-03-26**
OWNER NAME: **JACK CAMPBELL JR.**
OWNER ADDRESS: **P.O. BOX 1068**
KOUNTZE, TX 77625
COUNTY: **HARDIN**
LATITUDE: **30.347500000** LONGITUDE: **-94.344445000**

WELL LOG:		WATER LEVEL:	
DRILLING DATE (STARTED):	**2009-03-19**	STATIC LEVEL:	**31'**
DRILLING DATE (COMPLETED):	**2009-03-20**	WATER LEVEL DATE:	**2009-03-19**
DEPTH DRILLED:	**150'**	TYPE OF WATER:	**FRESH**

TYPE OF WORK:

NEW WELL

PROPOSED USE:

DOMESTIC

COMPANY INFORMATION:

COMPANY NAME: **BELLENGER WATER WELL SERVICE**
COMPANY ADDRESS: **P.O. BOX 1012**
SILSBEE, TX 77656

MAP ID# 33 **Distance from Property: 0.75 mi. SE**

TRACK #: **252466**

DATE ENTERED: **2011-05-09**

OWNER NAME: **MARK CROWDER**

OWNER ADDRESS: **5150 HWY 326**
KOUNTZE, TX 77625

COUNTY: **HARDIN**

LATITUDE: **30.325278000** LONGITUDE: **-94.351389000**

WELL LOG:		**WATER LEVEL:**	
DRILLING DATE (STARTED):	**2011-05-02**	STATIC LEVEL:	**40'**
DRILLING DATE (COMPLETED):	**2011-05-02**	WATER LEVEL DATE:	**2011-05-02**
DEPTH DRILLED:	**250'**	TYPE OF WATER:	**GOOD**

TYPE OF WORK:

NEW WELL

PROPOSED USE:

DOMESTIC

COMPANY INFORMATION:

COMPANY NAME: **DALE'S WATER WELL'S**

COMPANY ADDRESS: **3710 SWINNEY RD**
SILSBEE, TX 77656

MAP ID# 34 Distance from Property: 0.77 mi. NE

TRACK #: **133825**
DATE ENTERED: **2008-02-08**
OWNER NAME: **SHELTON**
OWNER ADDRESS: **3250 HWY. 326**
KOUNTZE, TX 77625
COUNTY: **HARDIN**
LATITUDE: **30.347222000** LONGITUDE: **-94.343334000**

WELL LOG:

DRILLING DATE (STARTED): **2006-04-05**
DRILLING DATE (COMPLETED): **2006-04-05**
DEPTH DRILLED: **130'**

WATER LEVEL:

STATIC LEVEL: **22'**
WATER LEVEL DATE: **2006-04-05**
TYPE OF WATER: **NOT REPORTED**

TYPE OF WORK:

NEW WELL

PROPOSED USE:

IRRIGATION

COMPANY INFORMATION:

COMPANY NAME: **WESST WATER WELL SERV.**
COMPANY ADDRESS: **PO BOX 82**
BATSON, TX 77519

MAP ID# 35 **Distance from Property: 0.78 mi. SE**

TRACK #: **335292**

DATE ENTERED: **2013-08-15**

OWNER NAME: **MARK CROWDER**

OWNER ADDRESS: **5080 HWY. 326 N.**
KOUNTZE, TX 77625

COUNTY: **HARDIN**

LATITUDE: **30.324722000** LONGITUDE: **-94.351667000**

WELL LOG:

DRILLING DATE (STARTED): **2011-12-01**

DRILLING DATE (COMPLETED): **2011-12-02**

DEPTH DRILLED: **160'**

WATER LEVEL:

STATIC LEVEL: **41'**

WATER LEVEL DATE: **2011-12-02**

TYPE OF WATER: **FRESH**

TYPE OF WORK:

NEW WELL

PROPOSED USE:

DOMESTIC

COMPANY INFORMATION:

COMPANY NAME: **NOT REPORTED**

COMPANY ADDRESS: **NOT REPORTED**
NOT REPORTED

TEXAS WATER DEVELOPMENT BOARD GROUNDWATER DATABASE (TWDB)

MAP ID# 36 **Distance from Property: 0.83 mi. N**

STATE ID:	**61-46-105**
OWNER'S NAME:	**J.J. OLUMS WELL 1**
DATE DRILLED:	**00/00/1950**
DEPTH DRILLED:	**8400'**
WATER USAGE:	
LONGITUDE:	**-94.362500000**
LATITUDE:	**30.352220000**
SOURCE:	**TWDB**

Page # 1 out of 1

State ID: 61-46-105

CROSS REFERENCE SHEET

Name or Subject CR-GWTD
HARDIN Located Well Data
LH 61-46-105

Date

Regarding Electric Log

SEE

Name or Subject GW-SC
ELECTRIC LOG FILE Q-127

B-152(62-1)

MAP ID# 36 Distance from Property: 0.84 mi. N

STATE ID:	**61-46-104**
OWNER'S NAME:	**KIRBY LUMBER CO WELL 1**
DATE DRILLED:	**00/00/1950**
DEPTH DRILLED:	**8518'**
WATER USAGE:	
LONGITUDE:	**-94.362780000**
LATITUDE:	**30.352220000**
SOURCE:	**TWDB**

State ID: 61-46-104

CROSS REFERENCE SHEET

Name or Subject	CR-GWTD HARDIN	 Located Well Data LH 61-46-104	**Date**
Regarding	Electric Log		

SEE

Name or Subject	GW-SC ELECTRIC LOG FILE	 Q-179

B-152(62-1)

MAP ID# 37 **Distance from Property: 0.88 mi. S**

TRACK #: **149212**
DATE ENTERED: **2008-08-11**
OWNER NAME: **DAVID KNAPP**
OWNER ADDRESS: **5062 HIGHWAY 326**
KOUNTZE, TX 77625
COUNTY: **HARDIN**
LATITUDE: **30.322500000** LONGITUDE: **-94.358334000**

WELL LOG:

DRILLING DATE (STARTED): **2006-01-17**
DRILLING DATE (COMPLETED): **2006-01-17**
DEPTH DRILLED: **350'**

WATER LEVEL:

STATIC LEVEL: **42'**
WATER LEVEL DATE: **2006-01-17**
TYPE OF WATER: **FRESH**

TYPE OF WORK:

NEW WELL

PROPOSED USE:

DOMESTIC

COMPANY INFORMATION:

COMPANY NAME: **BELLENGER WATER WELL**
COMPANY ADDRESS: **P.O. BOX 1012**
SILSBEE, TX 77656

MAP ID# 38 Distance from Property: 0.92 mi. NE

TRACK #: **42480**

DATE ENTERED: **2004-08-06**

OWNER NAME: **CLEWW LEWIS**

OWNER ADDRESS: **1746 OLD COURTHOUSE**
KOUNTZE, TX 77625

COUNTY: **HARDIN**

LATITUDE: **30.351945000** LONGITUDE: **-94.344722000**

WELL LOG:

DRILLING DATE (STARTED): **2003-10-06**

DRILLING DATE (COMPLETED): **2003-10-06**

DEPTH DRILLED: **200'**

WATER LEVEL:

STATIC LEVEL: **40'**

WATER LEVEL DATE: **2003-10-06**

TYPE OF WATER: **NOT REPORTED**

TYPE OF WORK:

REPLACEMENT

PROPOSED USE:

DOMESTIC

COMPANY INFORMATION:

COMPANY NAME: **WEST WATER WELL SERV.**

COMPANY ADDRESS: **P. O. BOX 82**
BATSON, TX 77519

MAP ID# 39 **Distance from Property: 0.92 mi. S**

TRACK #: **131320**

DATE ENTERED: **2008-01-10**

OWNER NAME: **WILLIAMS, JEFF**

OWNER ADDRESS: **275 FM 412 #23**
LUMBERTON, TX 77656

COUNTY: **HARDIN**

LATITUDE: **30.321944000** LONGITUDE: **-94.356389000**

WELL LOG:

DRILLING DATE (STARTED): **2007-04-20**

DRILLING DATE (COMPLETED): **2007-04-22**

DEPTH DRILLED: **383'**

WATER LEVEL:

STATIC LEVEL: **30'**

WATER LEVEL DATE: **2007-04-22**

TYPE OF WATER: **FRESH**

TYPE OF WORK:

NEW WELL

PROPOSED USE:

DOMESTIC

COMPANY INFORMATION:

COMPANY NAME: **BELLENGER WATER WELL**

COMPANY ADDRESS: **PO BOX 1012**
SILSBEE, TX 77656

MAP ID# 40 **Distance from Property: 0.95 mi. NE**

STATE ID:	**61-46-106**
OWNER'S NAME:	**AUDIE TAYLOR WELL 2**
DATE DRILLED:	**00/00/1950**
DEPTH DRILLED:	**8616'**
WATER USAGE:	
LONGITUDE:	**-94.348890000**
LATITUDE:	**30.354440000**
SOURCE:	**TWDB**

State ID: 61-46-106

CROSS REFERENCE SHEET

Name or Subject	CR-GWTD HARDIN	Located Well Data LH 61-46-106	Date
Regarding	Electric Log		

SEE

Name or Subject	GW-SC ELECTRIC LOG FILE	Q-126

B-152(62-1)

TEXAS WATER DEVELOPMENT BOARD GROUNDWATER DATABASE (TWDB)

MAP ID# 41 **Distance from Property: 0.97 mi. NW**

STATE ID:	**61-46-107**
OWNER'S NAME:	**FEE NO.1**
DATE DRILLED:	**00/00/1955**
DEPTH DRILLED:	**8506'**
WATER USAGE:	
LONGITUDE:	**-94.373890000**
LATITUDE:	**30.346940000**
SOURCE:	**TWDB**

State ID: 61-46-107

CROSS REFERENCE SHEET

Name or Subject CR-GWTD
HARDIN Located Well Data
LH 61-46-107

Date

Regarding Electric Log

SEE

Name or Subject GW-SC
ELECTRIC LOG FILE Q-55

B-152(62-1)

NWIS	United States Geological Survey National Water Information System
VERSION DATE: 5/2015	

This USGS National Water Information System database only includes groundwater wells. The USGS defines this well type as: A hole or shaft constructed in the earth intended to be used to locate, sample, or develop groundwater, oil, gas, or some other subsurface material. The diameter of a well is typically much smaller than the depth. Wells are also used to artificially recharge groundwater or to pressurize oil and gas production zones. Additional information about specific kinds of wells should be recorded under the secondary site types or the Use of Site field. Underground waste-disposal wells should be classified as waste-injection wells.

SDRD Select Submitted Drillers Report Database Wells

ERSION DATE: 4/2016

This Texas Water Development Board database was created from the online Texas Well Report Submission and Retrieval System (a cooperative TDLR, TWDB system) that registered water-well drillers use to submit their required reports. The system was started in February 2001 and is optional for the drillers to use. This data excludes the following well types: Monitor Wells, Environmental Soil Borings, Injections Wells, De-watering and Test Wells.

CEQ Texas Commission on Environmental Quality Water Wells

ERSION DATE: NR

The Texas Commission on Environmental Quality (TCEQ) maintains a filing system of plotted and unnumbered water wells. Plotted water wells are filed according to the County indicated by the driller and the state well number assigned by State of Texas personnel. Given the available location information provided by the driller, personnel identify where the approximate well location should be. After well placement a state well number is assigned indicating that the well lies within a specific 2.5' section of a 7.5' quadrangle. This method allows for quicker, more refined, reference when researching a specific area. Unnumbered water wells have not been assigned a state well number. This can occur for a variety of reasons; however it does not mean the well cannot be accurately spotted. Unnumbered water well records are filed according to County and are often broken up by year or by a span of years.

WDB Texas Water Development Board Groundwater Database

ERSION DATE: 1/2016

The Texas Water Development Board Groundwater Database contains information for more than 123,500 sites in Texas including data on water wells, springs, oil/gas tests, water levels, and water quality. The purpose of the Board's data collection effort over the years has been to gain representative information about aquifers in the state in order to do water planning. It is very important, however, to realize that the wells in the database represent only a small percentage of the wells that actually exist in Texas. A registered water well driller is required by law to send in a report to the State for every well that is drilled. This requirement began in 1965, and we estimate that approximately 500,000 wells have been drilled in Texas since then. Of the 1,000,000 plus water wells drilled in Texas over the past 100 years, more than 130,000 have been inventoried and placed into the TWDB groundwater database. State well numbers have been assigned to these based on their location within numbered 7 1/2 minute quadrangles formed by lines of latitude and longitude. This database contains well information including location, depth, well type, owner, driller, construction and completion data.

UD Water Utility Database

ERSION DATE: 2/2011

The Water Utility Database is defined as a collection of data from Texas Water Districts, Public

Drinking Water Systems and Water and Sewer Utilities who submit information to the TCEQ. This database is an integrated database designed and developed to replace over 160 stand alone legacy systems representing over 5 million records of the former Texas Water Commission and the Texas Department of Health.