FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS
TCEQ PERMIT NO. MSW-1983E

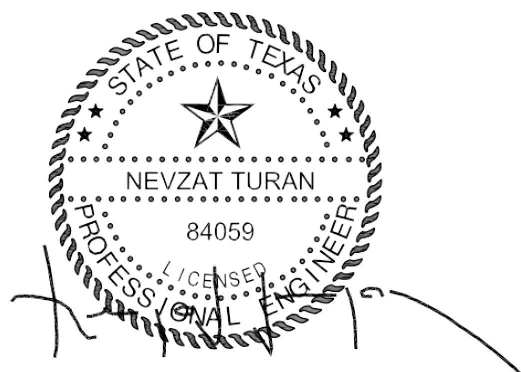
MAJOR PERMIT AMENDMENT APPLICATION

VOLUME 2 OF 4

Prepared for

Texas Regional Landfill Company, LP

February 2023



02/09/2023

Prepared by

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WCG Project No. 0771-356-11-35

This document intended for permitting purposes only.

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NEVZAT TURAN
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LICENSED
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TARRANT COUNTY, TEXAS
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MAJOR PERMIT AMENDMENT APPLICATION

PART III – SITE DEVELOPMENT PLAN

APPENDIX IIID
LINER QUALITY CONTROL PLAN

Prepared for

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CONTENTS

CONTENTS (Continued)

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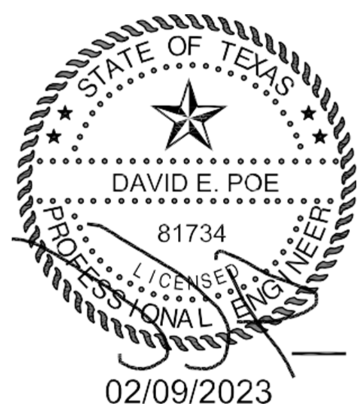
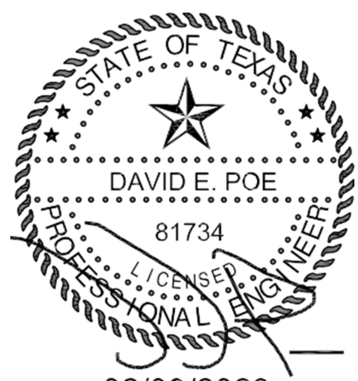


TABLES

Tables **Page No.**



02/09/2023

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 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 (geocomposite) 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 – In-Situ Shale Liner
- Section 4 – Construction Quality Assurance for Geosynthetics
- Section 5 – Construction Quality Assurance for Piping
- Section 6 – Liners Constructed Below the Highest Groundwater Level
- Section 7 – Documentation

1.2 Definitions

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

ASTM

This means the American Society for Testing and Materials.

Ballast Evaluation Report (BER)

Certification report for the constructed ballast, prepared and sealed by the POR and submitted to TCEQ.

Construction Quality Assurance (CQA)

A planned system of activities that provides the permittee 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 plans and specifications for a project.

Construction Quality Assurance Professional of Record (POR)

The POR is an authorized representative of the permittee and has overall responsibility for construction quality assurance that confirms that the facility was constructed in 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 interpretation. Experience and education must include geotechnical engineering, engineering geology, soil mechanics, geotechnical laboratory testing, construction quality assurance, and quality control testing, and hydrogeology. POR or his designated representative will be on-site during all liner system construction. 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 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 liner system 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 2 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 employed under the supervision of the qualified CQA monitor, who is required to be at the site during liner system construction.

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 LQCP.

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 geotextile or geocomposite, 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 non-conformances include, but are not limited to, physical defects, test failures, and inadequate documentation.

Permittee

Waste Connections (i.e., Texas Regional Landfill Company, LP) is the permittee, owner, and operator of the facility. Permittee, owner, and operator refer to the same entity throughout this plan.

Permittee'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.

Quality Assurance

This is a planned and systematic pattern of procedures and documentation to ensure that items of work or services meet the requirements of the contract documents. Quality assurance includes quality control. Quality assurance will be performed by the POR and CQA monitor.

Quality Control

These actions provide a means to measure and regulate the characteristics of an item or service to comply with the requirements of the contract documents. Quality control will be performed by the contractor.

Recompacted Clay Liner

Refers to the areas of liner that are over-excavated of alluvium soils and reconstructed with compacted clay soils and protective cover. These areas are identified on the figures and confirmed in the field by the POR as not meeting the in-situ shale criteria as described in this LQCP.

Registered Surveyor

Registered surveyor in this plan means an individual who, during the entire duration of surveying work, holds a valid registration from the Board of Professional Engineers and Land Surveyors.

Soil Liner Evaluation Report (SLER)

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

2 CONSTRUCTION QUALITY ASSURANCE EARTHWORK AND DRAINAGE AGGREGATES

2.1 Introduction

This section of the LQCP addresses the construction of the soil and underdrain 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 treatment of problems.

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

- Subgrade preparation
- Liner clay soil stockpile
- Recompacted clay liner placement and testing
- General and structural fill
- Drainage aggregates
- Excavation dewatering

2.2 Floor and Sidewall Liner

The proposed liner system for the facility is shown on engineering details in Appendix IIIA – Landfill Unit Design and is described as follows:

- 4-foot-thick (minimum) intact in-situ unweathered shale (which, if used, must meet all the physical properties for a constructed liner as detailed in Title 30 TAC §330.339(c)(5)); or
- 3-foot-thick recompacted clay liner having a hydraulic conductivity (k) of less than $1x10^{-7}$ cm/s and meeting all the criteria for a constructed liner as detailed in Title 30 TAC §330.339, overlain by a 1-foot-thick protective soil cover.

The existing landfill sectors at the site have been constructed and approved using both of the above liner systems. In general, the acceptable (intact and low permeability) in-situ shale material is present on the base (floor) areas and lower portions of the sidewalls, with recompacted clay liner used in areas of sidewalls with exposed alluvium soils (the water-bearing formation). A sector layout plan identifying the sector designations and locations, and dates of their Soil and Liner Evaluation Reports (SLERs) (as of the initial submittal date of this LQCP), is provided in Appendix IIID-C of this LQCP. Areas to receive the recompacted clay liner are also shown on figures included in Appendix IIID-C.

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.

2.3.1 Subgrade

Subgrade refers to a surface which is exposed after stripping topsoil or excavating to establish the grade directly beneath the recompacted clay liner in areas to receive constructed liner. The alternative in-situ shale liner is discussed in Section 3 of this LQCP. The prepared subgrade for unconstructed sectors must conform to the Excavation Plan included in Appendix IIIA – Landfill Unit Design.

Prior to beginning liner construction in areas to receive recompacted clay liner, 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 areas. Unstable areas will be undercut to firm material and refilled with suitable compacted general fill. Soil used for backfill beneath recompacted clay liner will meet the material requirements as general/structural fill and will be installed in accordance with the procedures described in Section 2.3.4. The subgrade fill will be free of organic matter, foreign objects, and other deleterious matter, compacted sufficiently to provide a firm base for recompacted clay liner placement. The subgrade will also be scarified prior to placement of the first lift of clay liner. The subgrade preparation specifications for each liner construction event will be developed in accordance with this section. Construction project specifications and construction plans will be developed for each cell construction event in accordance with this LQCP consistent with the Excavation Plan included in Appendix IIIA – Landfill Unit Design and the sector design as contained in the approved Site Development Plan.

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

The POR will approve the prepared subgrade prior to the placement of the underdrain, recompacted clay liner, or structural fill. Approval will be based on a review of visual inspection and test information, if applicable, and CQA monitoring of the subgrade preparation.

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 recompacted clay liner will achieve the required minimum thickness.

Excavation, grading, and conformation testing of the in-situ shale liner is discussed in Section 3 of this LQCP.

2.3.2 Excavations in Shaley Limestone or Fractured Shale

Prior to recompacted clay liner or underdrain geocomposite placement within an area that contains shaley limestone or cracked, fractured or weathered shale, that area will be excavated to a minimum depth of 3 feet, with a minimum 10-foot overlap into intact in-situ shale on all sides. After the shaley limestone or cracked shale has been removed, the subgrade preparation requirements of Section 2.3.1 will apply.

2.3.3 Recompacted Clay Liner

The recompacted clay liner will consist of a minimum 3-foot-thick compacted clay soil (measured perpendicular to the subgrade surface) that will extend along the floor and sidewalls of the landfill in areas not demonstrated to have in-situ shale (refer to Section 3 of this LQCP). The recompacted clay liner will be constructed in continuous 6-inch-thick compacted lifts installed parallel to the prepared floor and sidewall subgrades. Details depicting the liner system are included in Appendix IIIA – Landfill Unit Design. Clay Liner material will comply with the requirements of Table 2-1 below, and Section 2.3.3.1 of this LQCP. Construction methods for recompacted clay liner are presented in Section 2.3.3.2 of this LQCP.

2.3.3.1 Clay Liner Borrow Material

Adequate clay liner material will be available from proposed landfill excavations onsite or offsite borrow sources. The clay 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 Title 30 TAC §330.339(c)(5) prior to liner construction. As necessary, an off-site borrow source can be used for clay liner and protective cover construction.

Soils used in clay liners will have the following minimum values verified by testing in a soil laboratory prior to liner construction.

Table 2-1
Required Properties for Recompacted Clay Liner

Test[1]	Specification	Standard	Frequency
Moisture/Density Relationship	Determine moisture/density curve using a minimum of four data points	ASTM D 698	One per soil type
Coefficient of Permeability (Remolded Sample)[2]	$1.0x10^{-7}$ cm/s or less	COE EM1110-2-1906	
Plasticity Index	15 minimum	ASTM D 4318	
Liquid Limit, percent	30 minimum	ASTM D 4318	
Percent Passing No. 200 Mesh Sieve	30 minimum	ASTM D 1140	
Percent Passing 1-inch Sieve	100	ASTM D 448	
Unified Soil Classification	Reported in moisture/density test for soils meeting liquid limit, elastic limit, and percent passing -200	ASTM D 2487	

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 (determined using Moisture/density test) at or above the optimum moisture content.

Representative pre-construction sampling and testing will be performed on soils (on or offsite) to be used as clay liner material. Prior to construction of each clay liner construction event, conformance tests that include USCS classification, liquid limit, plastic limit, percent passing the No. 200 sieve, Standard Proctor (ASTM D 698) compaction test and coefficient of permeability test will be performed for each material type proposed. The coefficient of permeability test specimens for the liner will be prepared by laboratory compaction to a dry density of approximately 95 percent of the Standard Proctor maximum dry density at a moisture content greater than optimum. One Standard Proctor moisture-density relationship and remolded coefficient of permeability test will be required for each different material. The soil is considered as a separate soil borrow source if the liquid limit or plasticity index is determined to vary by more than 10 points. Additional conformance tests will be conducted if there are visual changes (color, texture, etc.) in borrow material or as determined necessary by the POR. The liquid limit and plastic limit testing will be performed on the separate borrow source as an initial determination. If the liquid limit or plasticity index varies by more than 10 points then all other testing listed in Table 2-1 will be performed on the material as a separate borrow source.

The CQA monitor, Earthwork Contractor, and/or Operator will identify the clay material during cell excavation, and the clay material will be stockpiled separately, if stockpiling is required. The liquid limit and plastic limit testing will be performed on the separate borrow source as an initial determination. If the liquid limit or plasticity index varies by more than 10 points from previous soil testing results then all other testing listed in Table 2-1 will be performed on the material as a separate borrow source.

The physical characteristics of the clay liner soils will be evaluated through visual observation before and during construction. To adjust moisture to the material properly, any clod sizes will first be crushed 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 clay liner will not be more than 10 percent by weight. Water used for the clay 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 or observed to contain excessive organic material or sediment.

2.3.3.2 Recompacted Clay Liner Construction

This LQCP has been developed in accordance with the TCEQ Regulations. The requirements for testing and evaluation of the clay 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 clay liner material will be placed in maximum 8-inch-thick loose lifts to produce a compacted lift thickness of approximately 6 inches. The clay liner will have elevations, slopes, and thickness as depicted on the drawings included in Appendix IIIA – Landfill Unit Design.

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). The compaction of the clay liner will be verified by a third-party independent laboratory to result in a coefficient of permeability of $1x10^{-7}$ cm/s or less.

The clay liner must be compacted with a pad/tamping-foot 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 liner lifts surface for bonding between 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 penetration during initial 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, weight of the compaction equipment is important. The minimum weight of the compactor should be 40,000 pounds, and a minimum of four passes are recommended for the compaction process. A pass is defined as one pass (1 direction) of the compactor, not just an axle, over a given area. The recommended minimum of five passes is for a vehicle with front and rear drums. The Caterpillar 815B and 825C are examples of equipment typically used to achieve satisfactory results.

The clay liner will not be compacted solely 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. 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 the next lift.

CQA testing of the clay liner will be performed as the liner is being constructed. Testing of the clay liner is addressed in Section 2.4 of this LQCP. Sections of recompacted clay liner which do not pass both the density and moisture requirements will be reworked with additional passes of the compactor until the section in question passes. All field density and moisture content test results will be incorporated into the SLER.

Hydraulic conductivity samples will be obtained by pushing a sampler through each lift of the constructed clay liner prior to construction of the next lift. 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 (e.g., samples for hydraulic conductivity) will be backfilled with bentonite or a bentonite/clay soil mixture consisting of at least 20 percent bentonite and compacted by hand tamping.

If the integrity of the "A" sample appears to have been compromised during the 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 maximum 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 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., six hydraulic conductivity tests per 100,000 square feet of reconstructed liner area for 3-foot-thick clay liner. 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. 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.

Clay liner construction and testing will be conducted in a systematic and timely fashion on each lift. Delays will be avoided in clay liner construction. After excavation is completed, construction and testing of the clay 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 finished surface of the final lift of clay liner must be sufficiently smoothed by construction equipment sufficient to provide a uniform surface that can be used for conformation surveying. Areas to receive geocomposite will be smooth-drum rolled prior to geocomposite installation. Smooth-drum rolling of recompacted clay liner not receiving geocomposite will not be required, as it interferes with the interface strength of the clay liner and overlying protective cover soils. After completion, the surface of the final lift of clay liner will 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 each sector. Top of clay liner surveying will be performed within a tolerance of -0.0 feet to +0.2 feet. Survey frequency is included in Table 2-2.

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

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

Upon approval of a SLER by TCEQ and prior to waste placement, SLER markers will be installed to clearly indicate the limits of constructed and approved liner areas in accordance with Section 13.1 – 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.

2.3.4 General Fill/Structural Fill

General fill/structural fill material will be comprised of uncontaminated earthen material placed under controlled conditions. Soil fill material placed below the recompacted clay liner (e.g., over-excavated areas within the liner construction area) or as fill for the perimeter berm or areas outside the limits of waste will be placed in uniform lifts which do not exceed 8 inches in loose thickness similar to compacted clay liner. General structural fill (e.g., perimeter berm construction or fill placed outside the limits of waste) will be placed in uniform lifts which do not exceed 12 inches in loose thickness and will be compacted to at least 90 percent of Standard Proctor maximum dry density (ASTM D 698). The fill placed below the compacted clay liner will be compacted to at least 95 percent of Standard Proctor maximum dry density (ASTM D 698) at a moisture content range at or above the optimum moisture content when it is used as fill below liner grades.

General and structural fill will be free of organics, angular rocks, and foreign objects larger than 1 inch. Soils described in Section 2.3.3.1 of this LQCP are acceptable as general and structural fill.

2.3.5 Drainage Aggregate in Sumps

The coarse aggregate (i.e., filter material) selected for placement around sidewall underdrain sumps and within the underdrain pipe trenches will consist of normal (i.e., typical unit weight of 90 to 120 pcf) or lightweight (i.e., unit weight not to exceed 70 pcf) materials that comply with the following criteria. The aggregate will meet the gradation for ASTM D 448, size number 467 or Grade 57 (nominal aggregate size 1.5 inches to No. 4). 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 1.0×10^{-2} cm/s and meet the filter gradation requirements given below (in no case will the maximum rock size be more than 2 inches) for the specific collection pipe perforation design:

For circular holes in the collection pipe:

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

For slots in the collection pipe:

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

Note that "85 Percent Size of Filter Material" corresponds to the d_{85} of the coarse aggregate surrounding the collection pipe (i.e., the particle size for which 85 percent of the filter material particles are smaller than). The coarse aggregate will be tested for gradation (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 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 organic matter, 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 SLER.

2.3.6 Protective Cover

Protective cover will be placed over the recompacted clay liner in accordance with this section and approved Excavation Plan (Appendix IIIA – Landfill Design Unit) for each liner construction. The protective cover will consist of soil materials that have not previously come in contact with solid waste. 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 and POR.

The thickness of the protective cover layer placed over the compacted clay 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 of 2 reference points. The survey results for the protective cover will be included in the SLER.

During construction the CQA monitor will:

- Verify that grade control is performed prior to work.
- Verify that underlying clay liner installation is not damaged during placement operations or by survey grade controls.
- Verify that the cover soil for sidewalls is pushed from the toe up the slope.
- 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.

2.3.7 Anchor Trench Backfill

The anchor trench backfill (if anchor trenches used) for geosynthetic anchoring will be uncontaminated earthen material and will be placed in uniform lifts which do not exceed 12 inches in loose thickness and will be compacted to at least 90 percent of Standard Proctor maximum dry density (ASTM D 698). In-place moisture/density tests may be taken at the discretion of the CQA monitor to evaluate the quality of the backfill. The test results will not be required as part of the SLER. Note that anchor trenching for the underdrain geocomposite will not be required, as demonstrated by the infinite slope stability analyses included in Appendix IIID-C of this LQCP.

2.3.8 Surface Water Removal

The excavation may encounter water from storm events or groundwater. Soil for liner, general fill, or structural fill will not be placed in standing water or over soft or pumping subgrade. The excavation area will therefore have a temporary sump area to collect water entering the excavation and will be graded to allow drainage at planned areas. 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 recompacted clay liner will be removed promptly after the end of a rainfall event. The POR will inspect and approve the constructed area that received rainfall prior to placement of overlying liner system component. Surface water removed from the excavation areas will be discharged per the site's TPDES permit requirements.

2.3.9 Excavations Below Groundwater

The remaining landfill excavations (Sectors 4, 5 and 6) extend below the highest measured groundwater levels (potentiometric surface) in the water-bearing alluvium in the remaining sectors. An updated (2022) Highest Measured Groundwater Elevation Map is provided in Appendix IIID-A as Figure IIID-A-1. The potential short-term hydrostatic pressure acting on the remaining Sectors 4, 5 and 6 sidewall clay liners will be mitigated by the implementation of temporary underdrain dewatering systems or ballasting as discussed below. The dewatering systems (if required) will be constructed of 200-mil double-sided geocomposite drainage layer installed below the recompacted clay liner that will drain groundwater from beneath the clay liner system. The geocomposite placed on the sidewalls of the cell will drain into a drainage trench and 4-inch-diameter perforated HDPE pipe (SDR 17) installed at the lower shale/alluvium contact. The HDPE drainage trench and pipe will be installed with a minimum 0.5 percent slope, and will drain into an 18-inch-diameter sidewall sump also installed below the recompacted clay liner as shown on Figures IIID-C-5 and IIID-C-6, or drain into unconstructed areas of the sector.

The temporary sidewall sumps will be constructed to maintain positive drainage within the underdrain trench and geocomposite underdrain. The temporary sumps will consist of a horizontal 18-inch-diameter HDPE enveloped in drainage stone and non-woven geotextile, and an 18-inch-diameter sidewall riser pipe for removal of the underdrain groundwater by pumping. The sidewall sumps will be equipped with a submersible pump that will remove groundwater to the surface for discharge into the landfill's surface water management system.

Clay liner areas that extend below the highest measured groundwater potentiometric surface will be designed and constructed to provide long-term protection against uplift from hydrostatic forces by the use of ballast placed over the recompacted clay liner in accordance with Title 30 TAC §330.203. Ballast, if required, will be placed and verified as described in Appendix IIID-B. Example ballast calculations are provided in Appendix IIID-B. Additional discussion of the underdrain system for each of the remaining unconstructed landfill sectors is provided in Section 6 of this LQCP.

2.3.10 Control of Seepage During Construction

Seepage of free water from the exposed soils within the bottom of the disposal cell is not expected during liner construction due to the temporary dewatering system that will be in place before liner construction is initiated and the shale exposed within the cell floor. During construction, the subgrade must be maintained in a firm and unyielding condition to provide a satisfactory foundation for construction of the clay liner. If unexpected seepage is encountered, the POR will inspect the seeps and delineate the area. Per the POR's direction, the wet soils will be reworked or over-excavated and replaced with compacted clayey soil to seal off the seepage. Soft areas will be undercut to firm material and backfilled with suitable compacted 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 subgrade for recompacted clay liner placement as described in Section 2.3.1 of this LQCP.

2.3.11 Liner Tie-In Construction

Newly constructed recompacted clay liners will be tied-in with any adjoining existing liners or into competent in-situ shale. Additionally, terminations will be constructed for future tie-ins along edges where the clay liner will be extended in the future. The tie-ins with existing clay liners will be constructed utilizing a sloped transition a minimum of 15-foot-wide for the 3-foot-thick clay liner. 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. 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 Title 30 TAC §330.143(b)(1).

2.4 Construction Testing

2.4.1 Standard Operating Procedures

Qualified CQA monitors will perform field and laboratory tests in accordance with applicable standards specified in this LQCP. All quality control testing and evaluation of recompacted clay liners will be performed during construction of the liner Standard operating and test procedures will be utilized per the POR's direction. Sampling from the recompacted clay liner lifts will be performed in accordance with ASTM D 1587. The sampling holes (e.g., samples for coefficient of permeability test) will be backfilled with bentonite or bentonite/liner soil material mixture. The standard operating procedure will be prepared or modified by the POR during construction, as necessary, to address site specific construction issues. Prior written approval will be obtained from the TCEQ if any changes to material requirements or procedures set forth in this LQCP will be made.

The following test standards apply as called out in this LQCP and in the technical specifications provided in this LQCP.

Standard Test Method	Test Description
ASTM D 698	Laboratory compaction characteristics of soil using standard effort
ASTM D 422	Particle size analysis of soils

Standard Test Method	Test Description
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 6938	In-place density and water content of soil and soil-aggregate by nuclear methods (shallow depth)
ASTM D 2216	Laboratory determination of water (moisture) content of soil and rock by mass
ASTM D 2434	Method of test for permeability of porous granular material
ASTM D 5084	Method of test for permeability of fine-grained soils
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

2.4.2 Test Frequencies

This LQCP establishes the minimum test frequencies for the recompacted clay liner construction quality assurance. The test frequencies for clay liner are listed in Table 2-2. Additional testing must be conducted 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.

The cell construction may include placement of general fill in order to establish excavation grades and structural fill for construction of perimeter berms around the exterior limits of the sector. Testing will be limited to one Standard Proctor moisture-density relationship (ASTM D 698) per borrow source (as defined in Section 2.3.3.1) per project.

2.5 Reporting

The POR will submit to the TCEQ a SLER for approval of each clay liner area. Section 7 of this LQCP presents the documentation requirements for the SLER.

Table 2-2
Required Tests and Observations on Recompacted 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 [2]	95% Maximum Standard Proctor Dry Density. Standard Proctor optimum moisture content or greater 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) [1]	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 [3]	1 each 5,000 square feet with a minimum of 2 reference points by a licensed Texas land surveyor	Survey subgrade and top of clay liner and protective cover layer	3 feet minimum compacted clay liner thickness and 1 foot minimum protective cover thickness

1 Field permeability testing in accordance with Title 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.
2 This method is not applicable if the field nuclear gauge reads both density and moisture.
3 The liner will be constructed in parallel lifts and not horizontal lifts for sidewalls.

3 IN-SITU SHALE LINER

3.1 Introduction

This section addresses the specifications and CQA requirements for confirmation of the in-situ shale liner. In-situ shale liner refers to the intact unweathered shale stratum that is present at the base of the landfill (floor grades) and some sidewall areas, and which, if suitable in accordance with this section, will serve as the liner in these areas of the landfill.

In-situ liner has been previously approved by TCEQ for this facility, and the existing sectors have been constructed accordingly. The geotechnical studies at the site, as well as previously submitted and approved SLERs, indicate that the shale is a dense, clayey, low permeability mass, highly suitable for use as a landfill liner.

3.2 In-Situ Shale Liner Specifications

3.2.1 In-Situ Shale Liner Material Requirements

Specifications for the physical properties of the in-situ shale liner are presented in Table 3-1.

3.2.2 In-Situ Shale Liner Construction

The landfill floor will be excavated down into the unweathered shale stratum. Also, in-situ shale exposed on the sidewalls during excavation, where present, will be utilized for the liner on the sidewalls. If shaley limestone is encountered or the existing shale is cracked (or otherwise exhibits primary or secondary features such as jointing, fractures, bedding planes, solution cavities, root holes, desiccation shrinkage cracks, etc., that have a coefficient of permeability greater than $1x10^{-7}$ cm/s), the area will be reworked or excavated and lined with a recompacted clay liner as detailed in Section 2 of this LQCP. All areas of the landfill must have a minimum thickness of 4 feet of intact in-situ shale liner, or at least 3 feet of recompacted clay liner (recompacted clay will be overlain by 1 foot of protective cover).

3.3 In-Situ Shale Liner CQA

3.3.1 Field Evaluation/Monitoring During Construction

Once the excavation of the landfill has achieved final excavation grades, the exposed shale surface will be observed by the POR or their CQA monitor in order to confirm the continuous presence of the shale on the floor and sidewall.

3.3.2 In-Situ Liner Testing

Representative samples of the in-situ shale liner material will be tested in the laboratory. Field permeability testing may also be required in addition to laboratory testing. The test methods and frequencies for CQA testing of the in-situ shale liner are given in Table 3-2. Sampling and test locations will be selected by CQA personnel. The CQA monitor will perform the field sampling and testing in accordance with the following procedures:

1. **Augering** (or similar coring or manual extraction of samples) for obtaining test specimens and for thickness verification) will be performed at the minimum frequency given in Table 3-2. Augering will be performed to a depth of 4 feet below the excavated subgrade with a motorized hand auger, a mobile truck-mounted rig, or equivalent equipment. Holes will be augered perpendicular to the final in-situ liner surface (note that the reason for designating the in-situ liner as being 4 feet thick is that the upper foot of the shale will be classified as protective cover).

2. **Thickness Verification.** To verify thickness of the in-situ liner, the CQA technician will supervise augering activities and will verify the presence of the required thickness of intact shale material.

3. **Soil Sampling and Testing.** Bulk soils retrieved from the augering activities may be combined into composite samples and used for laboratory testing of particle size analysis and Atterberg limits. Undisturbed samples will be obtained for laboratory hydraulic conductivity testing of in-situ material (note that a thin-walled sampler (e.g., Shelby tube) may be used to obtain undisturbed specimens for hydraulic conductivity testing; however, given the stiff brittle nature of the shale, a thick-walled Shelby tube, rotary core, or similar device for obtaining undisturbed soil core samples may be used). Laboratory testing will be performed at the frequency indicated in Table 3-2.

4. **Field Permeability Testing.** Although not expected or conducted in the past at this facility, if field permeability testing is deemed necessary it will be performed at the rate of one sealed double-ring infiltrometer (SDRI) series or approved equivalent number of two-stage borehole (i.e., "Boutwell") tests for each 50,000 ft^2 of in-situ liner surface. Field permeability testing will only be required on specific areas of in-situ shale liner that laboratory testing or the

POR have determined that the quality of the in-situ material in a given area is marginal.

5. **Survey Documentation.** A registered surveyor will identify and survey the auger locations, and these locations and the surveyed information will be included in the SLER.
6. **Filling of Auger Holes.** After completion of testing/verification activities, auger holes will be filled in with a bentonite grout, bentonite pellets, or by tamping in lifts a mixture of the augered shale and 20 percent (minimum) by weight of powdered bentonite.

3.3.3 Deficiencies, Problems, and Repairs

Sections of in-situ liner that do not meet the requirements of Section 3.2.2 or that do not pass required field or laboratory tests (e.g., not meeting the required hydraulic conductivity) will be excavated and replaced with recompacted clay liner in accordance with Section 2 of this LQCP. If a failure occurs, first the failing area will be defined. This will be accomplished performing additional tests between the failed test and the nearest adjacent passing test locations. If those additional tests pass, then the area between the failed test and the additional passing tests must be reworked and retested until passing. If the additional tests fail, then additional tests must be performed halfway between the initial additional tests and the adjacent passing tests to further define the failing area. This procedure must be repeated until the failing area is defined, reworked, and retested with passing results. All field test results will be reported in the SLER whether they indicate passing or failing values.

3.3.4 In-Situ Shale Liner Documentation

Documentation of the in-situ shale liner will be included in the SLER. The required SLER contents are described subsequently in Section 7 of this LQCP.

Table 3-1
Material Specifications for In-Situ Shale Liner

Property	Qualifier	Units	Specified Values	Test Method [1]
Thickness	Minimum	Feet	4	Augering and Measurements by CQA Monitor
Maximum Particle Size	Maximum (nominal)	Inches	1	ASTM D 422
Percent Passing #200 Sieve	Minimum	Percent	30	ASTM D 422
Liquid Limit	Minimum	Percent	30	ASTM D 4318
Plasticity Index	Minimum	Percent	15	ASTM D 4318
Hydraulic Conductivity	Maximum	cm/s	$1x10^{-7}$	ASTM D 5084[(2)]

1 CQA testing frequencies are provided in Table 3-2.
2 Refer to Table 3-2 for additional hydraulic conductivity testing requirements.

Table 3-2
Field Testing Requirements for In-Situ Shale Liner

Test	Method	Minimum Frequency of Testing	Passing Criteria
Layer Thickness Verification	Augering and Measurements by CQA Monitor	One per 5,000 ft^2 of surface area	> 4-feet-thick
Particle Size Analysis	ASTM D 422	One per 50,000 ft^2 per foot of liner thickness (minimum one test for each unit thickness of liner, regardless of liner area or length)	See Table 3-1
Atterberg Limits	ASTM D 4318	1 per 50,000 ft^2 per foot of liner thickness (minimum 1 test for each unit thickness of liner, regardless of liner area or length)	See Table 3-1
Laboratory Hydraulic Conductivity [2]	ASTM D 5084 [1]	1 per 50,000 ft^2 per foot of liner thickness (minimum 1 test for each unit thickness of liner, regardless of liner area or length)	$<1x10^{-7}$ cm/s

1 Hydraulic conductivity tests will be performed on undisturbed specimens obtained as described in Section 3.3.2. The hydraulic conductivity tests will be performed using tap water or a 0.05N solution of $CaSO_4$, and at an effective stress of 20 psi. Distilled or deionized water will not be used. The permeant should be deaired. All hydraulic conductivity test data will be submitted with the SLER.
2 See Section 3.3.2 for a discussion on the possible use of field permeability testing.

4 CONSTRUCTION QUALITY ASSURANCE FOR GEOSYNTHETICS

4.1 Introduction

Section 4 describes CQA procedures for the installation of geosynthetic components. For this LQCP, geosynthetics is limited to the geocomposite used in the liner underdrain system, and for non-woven geotextile used within the pipe trenches.

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

- Geotextiles
- Drainage Layer
  - Double-sided drainage geocomposite (200-mil on limited sidewalls)

The overall goal of the geosynthetics quality assurance program is to assure that proper construction techniques and procedures are used, the geosynthetic contractor implements his quality control 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 construction event. The quality assurance 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.

4.2 Geosynthetics Quality Assurance

4.2.1 General

To monitor compliance, a quality assurance program will include the following:

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

Conformance testing refers to material testing performed by an independent third party laboratory that takes place prior to material installation.

Quality assurance 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.

4.3 Geotextiles

Geotextiles will be used to prevent clogging of drainage materials. Main usage of geotextiles will be enveloping drainage stone used for drainage trenches and sumps in the temporary dewatering underdrain system.

4.3.1 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 this LQCP and the specifications has been received and reviewed for compliance with this LQCP.
- 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.

4.3.2 Testing

The geotextile manufacturer will conduct manufacturer quality control (MQC) testing and certify that the materials delivered to the site comply with project specifications outlined in this LQCP. 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.

4.3.3 Geotextile Installation

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 4.4.4.
- 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 outlined in this LQCP.

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

4.3.4 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.

4.4 Drainage Geocomposite – Geonet and Geotextile

Drainage geocomposite will be used for the underdrain installed below the recompacted clay liner. Drainage geocomposite used for the construction will meet the requirements set forth in Appendix IIID-C along with this LQCP. Manufacturer's testing for drainage geocomposite is listed in Table 4-1. The drainage geocomposite will also meet the drainage requirements listed in Table 4-1.

4.4.1 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.

4.4.2 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 4-1. The minimum testing frequency will be one test sample per 100,000 square feet of geocomposite. See footnotes 2 and 3 of Table 4-1 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 4-1.

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 4-1, footnotes 2 and 3).

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

Table 4-1
Geotextile and Drainage Geocomposite Required Testing and Properties[1]

Responsible Party	Material	Test	Standard	Required Underdrain Property[4]
Manufacturer	Geotextile (before lamination)	Unit Weight	ASTM D 5261	8 oz/sy
		Apparent Opening Size	ASTM D 4751	0.180 mm
		Grab Strength	ASTM D 4632	220 lb
		Grab Elongation	ASTM D 4632	50%
		Tear Strength	ASTM D 4533	95 lb
		Puncture Strength	ASTM D 6241	575 lb
		Permeability	ASTM D 4491	1.3 cm^{-1}
		UV Stability	ASTM D 4355	70%
Manufacturer	HDPE Geonet (before lamination)	Density	ASTM D 1505	0.94 g/cm^3
		Thickness	ASTM D 5199	0.2 inch
		Carbon Black Content	ASTM D 1603	2%
		Tensile Strength	ASTM D 7179	55 lb/in
		Compressive Strength	ASTM D 6364	
		Transmissivity	ASTM D 4716	14.49 gpm/ft
Third Party Laboratory	Drainage Geocomposite	Transmissivity	ASTM D 4716	See Note 2
		Strength	ASTM D 5321	
Manufacturer		Ply Adhesion	ASTM D 7005	1.0 lb/in

[1] The minimum testing frequency will be one test sample per 100,000 square feet. The drainage geocomposite will be double-sided only, with the geonet heat fused to the non-woven geotextile.

[2] As noted in Appendix IIID, Appendix IIID-C, the transmissivity of the sidewall liner double-sided geocomposite will be measured at a minimum gradient of 0.33 under normal pressures of 750, 1,500 and 2,500 psf (or higher), boundary conditions consisting of soil/geocomposite/soil with a minimum seating time of 100 hours. The minimum transmissivity will be 2.0 x 10^{-3} 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., 4,000 psf) with all the other assumptions the same as the first three tests.

[3] Minimum required property values for the geotextile and drainage geocomposite transmissivity are based on calculations provided in Appendix IIID-C. The geonet properties are based on values specified by multiple manufacturers which are consistent with GRI-GM-4. In addition, each material will be tested prior to construction to verify that it meets the minimum required properties. At the time of each construction event, an updated GRI-GM-4 will be used if available.

4.4.3 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.
- The supporting surface does not contain angular stones that could damage the geocomposite or the geomembrane.

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

- 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 4.4.4.
- Verify that equipment used does not damage the drainage geocomposite 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.
- 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.
- 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 geotextile component is overlapped and either heat bonded or sewn together.
- Observe that sandbags or other methods are used to secure the geocomposite prior to recompacted clay liner placement. Clay liner soil placement over the geocomposite will be by pushing soil from the toe of slope upward. Placement of soil from the top of slope downward in sections of slope with geocomposite will not be allowed.

4.4.4 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 jointed.

4.5 Equipment on Geosynthetic Materials

Construction equipment on the underdrain system will be minimized to reduce the potential for puncture. The CQA monitor will verify that small equipment such as generators are placed on scrap material (rub sheets) above geosynthetic materials. 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 recompacted clay soil layers are being placed.

Unless otherwise specified by the POR, all lifts of protective soil material 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 geosynthetic-lined area.

5 QUALITY ASSURANCE FOR PIPING

5.1 Introduction

This section describes CQA procedures for the installation of HDPE pipe for the underdrain (including both 4-inch-diameter collection piping and 18-inch-diameter sidewall sumps and sump risers). This LQCP stresses careful documentation during the quality assurance process, from the selection of materials through installation.

The goal of the pipe quality assurance 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 that will be developed in accordance with this LQCP for each future construction event. All pipe designs will be prepared to provide adequate pipe size (diameter), strength (SDR value), and opening sizing (perforations or slots) to provide reliable performance of the pipe.

The quality assurance program is intended to identify and define problems that may occur during construction and to observe that these problems are corrected before construction is complete. A construction report, prepared after project completion, will document that the constructed facility meets design standards and specifications.

5.2 Pipe and Fittings

5.2.1 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 IIID-C. To monitor compliance, the CQA Program includes: (1) a review of the manufacturer's quality control testing, (2) material conformance testing, and (3) construction monitoring. Conformance testing refers to testing by an independent third party laboratory that will take place prior to material installation on materials delivered to the site as provided by the manufacturer.

5.2.2 Delivery

The CQA monitor will observe:

- That upon delivery, the pipe and pipe fittings are in compliance with the requirements of the construction specifications that will be developed in accordance with this LQCP for each liner construction sequence.
- That a storage location is selected in which the pipe and pipe fittings are protected from excessive heat, cold, construction traffic, hazardous chemicals, and solvents. 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 have been protected.
- That upon transporting pipe and fittings from the storage location to the construction site, the contractor will use pliable straps, slings, or rope to lift the pipe. Steel cables or chains will not be allowed to transport or lift the pipe.
- 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 this material in order to maintain compliance with this portion of the LQCP.

5.2.3 Conformance Testing

Prior to the installation of pipe, the pipe manufacturer will provide to the Operator and 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, perforations, and production lot.
- Properties sheet including, at a minimum, all specified properties, measured using test methods indicated in the specifications that will be developed in accordance with this LQCP 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 CQA monitor will observe that:

- The property values certified by the pipe manufacturer meet all of the specifications that will be developed in accordance with 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.
- The certified properties meet the specifications that will be developed in accordance with this LQCP for each liner construction sequence.

5.2.4 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 may clog the pipe.
- Pipe perforations 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 drainage trench.
- Pipe perforations are to the correct size and spacing according to the specifications that will be developed in accordance with this LQCP for each liner construction. Perforations can be either factory installed slots or factory 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 procedures are 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.
- Observe that the people and equipment utilized to install the pipe do not damage the pipe or any other component of the liner system.

6 LINERS CONSTRUCTED BELOW THE HIGHEST MEASURED GROUNDWATER LEVEL

6.1 Introduction

Recompacted clay liners constructed below the groundwater surface may potentially experience uplift due to hydrostatic pressure acting on the bottom of the clay liner. This section of the LQCP describes procedures for short and long-term protection of the clay liner system from damage resulting from hydrostatic pressure uplift that may result from the clay liner being constructed below the highest measured groundwater table.

The geology of the site generally consists of alluvium overlying weathered and unweathered shale strata. The base (floor) of the proposed cell excavations will be primarily founded in the shale, which will act as an in-situ liner. The shallow groundwater is contained within the alluvium which overlies the shale and will be exposed in the excavation sidewalls for unconstructed Sectors 4, 5 and 6. A temporary underdrain dewatering system will be constructed below the recompacted clay liner for the perimeter sidewalls contacting the water-bearing alluvium in Sectors 4 and 5.

Long-term liner stability of the recompacted clay liner will be provided in the form of ballasting that will be created by the weight of clay liner and protective cover, waste, and final cover as applicable. Example ballast calculations are presented in Appendix IIID-B – Example Ballast Thickness Calculations. A copy of the TCEQ's Ballast Evaluation Report form (TCEQ-10072) is included in Appendix IIID-D. The highest measured groundwater surface (2022) is presented on Figure IIID-A-1 (Appendix IIID-A) and will be used for future ballast demonstration calculations. Figures presenting the locations of the sidewall underdrains and sidewall sumps, and details of the underdrain system are included in Appendix IIID-C of this LQCP.

6.2 Highest Measured Groundwater Levels

The Highest Measured Groundwater Elevation Map is included as Figure IIID-A-1 in this appendix. Detailed groundwater information is presented in Appendix IIIG – Geology Report.

During the design and subsequent ballast demonstrations for Sectors 4, 5 and 6, the highest measured groundwater level will be adjusted upward for possible higher well level data available at the time of design or demonstration, and the highest measured groundwater potentiometric contours for each sector will be incorporated as appropriate. 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. Adjusting the elevations of the relief system design based on future changes in groundwater elevations does not require a permit modification.

6.3 Temporary Dewatering System

A temporary dewatering system was installed in portions of the developed disposal areas, and future temporary dewatering systems will be installed in the undeveloped Sectors 4 and 5.

6.3.1 Dewatering System for Developed Areas

The existing dewatering system includes a trench installed at the perimeter sidewalls or at the toe of excavation in areas subject to hydrostatic uplift. The dewatering system consists of a trench with a minimum depth of 2 feet and minimum width of 6 inches. The temporary trench drain consists of a perforated pipe enveloped with gravel and a geotextile wrap. A geocomposite drainage layer extends up the slope above the highest measured groundwater elevations (or beyond the alluvium layer elevation, as determined in the field by the POR) to maintain the drawdown condition produced by the open excavation. The trenches are constructed at the contact of and overlain alluvium soil layer and the unweathered shale and a minimum of 2 feet of penetration into the shale. A Ballasting Evaluation Report (BER) was submitted for all developed areas (combined) of the landfill in June 2021, which demonstrates that the developed areas of the landfill are adequately ballasted to allow discontinuation of groundwater removal and monitoring. A copy of the approval letter is included in Appendix IIID-E.

6.3.2 Dewatering System for Undeveloped Areas

The temporary dewatering system design presented in this application has been developed to prevent the build-up of hydrostatic groundwater uplift in the undeveloped sectors, specifically Sectors 4 and 5. As discussed in Section 6.3.2.3 below, groundwater in Sector 6 will be ballasted by the 3-foot-thick recompacted clay liner and 1-foot-thick protective cover soils and will not require an underdrain.

The temporary dewatering system will collect groundwater from the water-bearing alluvium exposed during cell excavations. The design of the system is further discussed in Appendix IIID-C. The excavation sidewall underdrain dewatering system design is based on the highest measured groundwater contours shown on Figure IIID-A-1 (2021) in Appendix IIID-A.

Appendix IIID-C includes the design calculations for the temporary dewatering system that will be installed beneath the sidewall liner. As shown in Appendix IIID-C, a drainage geocomposite will convey groundwater to a collection trench installed at the toe or on the sidewall of the sector. Any water collected in the sidewall sump (if used) will be removed by a submersible pump and pumped to the perimeter stormwater system where it will be discharged from the site consistent with the TPDES stormwater permit.

The temporary dewatering system will remain operational until enough ballast is placed in the form of protective cover and solid waste over the impacted area. Once sufficient ballast is in place and with the written approval (BER) of TCEQ, the dewatering system will be decommissioned. Example ballast evaluation calculations are presented in Appendix IIID-B. The pumps will be activated upon installation of the dewatering system and will remain operational until the BER is approved by the TCEQ. The pumps will be operated automatically by pressure transducers.

The underdrain system proposed for each of the unconstructed sectors (4 and 5) are discussed in the following sections.

6.3.2.1 Sector 4 Groundwater Uplift Control

Review of geological and seasonal high groundwater information for Sector 4 indicates that two waterbearing zones (referred to as Upper Zone A and Lower Zone B on Figure IIID-C-2) intercept the sidewall at varying elevations. The eastern portion of Sector 4 intercepts the Upper Zone A between approximate elevations 632 and 656 ft-msl. The western portion of Sector 4 intercepts the Lower Zone B at approximate elevations 572 to 579 ft-msl at the westernmost end of Zone B existing in the sidewall, and approximate elevations 630 to 634 at the easternmost end Zone B. Separate underdrain systems are proposed for these two alluvium zones, with independent sidewall sumps proposed for each zone (thus reducing the risk of hydraulic loading of the lower sump from the upper (higher) elevation sump). Note that the limits of geocomposite underdrain system shown on Figure IIID-C-2 are estimates only, based on limited geological investigations, and will be confirmed in the field during sector construction, and adjusted accordingly (including potential relocation of the sidewall sumps to facilitate field conditions).

The waterbearing alluvium will be dewatered by the installation of a geocomposite drainage layer and toe drain as represented in details included on Figures IIID-C-5 and IIID-C-6. For Zone A, the geocomposite will be installed into a collection trench located near the shale/alluvium interface, and will extend up the 3H:1V slope approximately 25 feet (which equates to approximately 8 feet vertical) which will provide for dewatering and drainage of the porous alluvium. Details of the sidewall trench and sump are provided on Figures IIID-C-5 and IIID-C-6. For the Lower Zone B, the geocomposite will be installed in a collection trench located near the shale/alluvium interface and will extend up the slope approximately 10 feet (which equates to approximately 3 foot vertical). The shorter geocomposite length is based on the depth of alluvium in Zone B being approximately 4 to 5 feet in thickness. Note that for both Zone A and Zone B, over excavation of the alluvium strata will extend vertically until shale meeting the requirements of Section 3 of this LQCP is encountered, or more typically to the top of the sidewall slope.

Calculations were performed for the worst-case Zone A conditions, assuming a 20-foot-thick waterbearing alluvium thickness that demonstrates the adequacy of the proposed 200-mil double-sided geocomposite underdrain and the 4-inch-diameter trench drain piping are attached. Calculations were performed assuming a trench length of 1,800 feet, which is a conservative assumption of the trench length for the upper Zone A in Sector 4.

6.3.2.2 Sector 5 Groundwater Uplift Control

Review of the geological and highest groundwater information for Sector 5 indicates that the entire exterior sidewall of the cell may be waterbearing alluvium soils, and subject to groundwater uplift. Unlike Sector 4, it is proposed the entire exterior sidewall of Sector 5 will be constructed with a geocomposite underdrain overlain by the recompacted clay liner. A detail of the Sector 5 underdrain is presented on Figure IIID-C-5. Both the floor and sidewall of the sector will be inspected after excavation, and in the event alluvium is encountered in the floor of the sector, the underdrain design will be modified to incorporate drainage geocomposite underdrain and recompacted clay liner construction in these floor areas also. As the area of drainage for Sector 5 is smaller than Sector 4 (in both vertical slope height and length of slope), the geocomposite and drainage pipe calculations performed for Sector 4 (attached) are assumed sufficient.

6.3.2.3 Sector 6 Groundwater Uplift Control

Review of geological logs installed in the vicinity of the future Sector 6 indicate the waterbearing alluvium has a uniform thickness of 4 to 5 feet or less along the proposed Sector 6 exterior berm. For Figure IIID-C-4, it was assumed that the waterbearing alluvium is present between approximate elevations 570 to 575. For this sector, calculations are included in Section IIID-C that demonstrate that the 5-foot-thick water bearing stratum is adequately ballasted by the 3-foot-thick clay liner and 1-foot-thick soil protective cover. No underdrain is proposed.

As shown on Figure IIID-A-1, the highest measured groundwater elevation is reported as approximately 580 ft-msl along the exterior berm/embankment alignment at Sector 6. However, post-construction groundwater elevations will also be controlled by both the excavation of the Sector 6 disposal area, and by the excavation of the borrow area/floodplain storage area located immediately west of the Sector 6 perimeter berm/embankment, as shown on Figure IIID-C-1. As shown, the borrow area/floodplain storage area floor ranges from approximate elevation 576 ft-msl at the southern end of the excavation, to approximately 568 ft-msl at the northern end of the excavation. The excavation will facilitate partial (or complete) short and long-term dewatering of the alluvium stratum in the Sector 6 perimeter berm/embankment.

After excavation of Sector 6, and inspection will be conducted of the sector sidewalls by the POR, and in the event the waterbearing formation is observed to be of greater thickness than assumed, the ballasting calculations will be updated, and if needed, an underdrain consistent with the underdrains proposed for Sectors 4 and 5 will be installed.

6.4 Temporary Dewatering System Materials

6.4.1 Dewatering System Drainage Aggregate

Refer to Section 2.3.5 of this LQCP.

6.4.2 Dewatering System Piping

The dewatering trench pipe will consist of a 4-inch-diameter HDPE SDR-17 pipe, or a POR-approved equivalent.

Typical total pipe perforation will be 1 square inch per 1 lineal foot of pipe length. Perforation sizes (hole diameter or slot width) will be in accordance with the gradation versus perforation requirements outlined in Section 2.3.5.

Refer to Section 5 of this LQCP for pipe manufacturing and installation requirements.

6.4.3 Geotextile

The non-woven geotextile will be wrapped around the drainage stone and the collection pipe in the temporary dewatering trench. It will have a weight of at least 6 oz/sy. There will not be any direct contact between the geotextile and any compaction equipment.

6.4.4 Drainage Geocomposite

A drainage geocomposite will be used for the dewatering layer. The drainage geocomposite will meet the requirements set forth in Appendix IIID-C and will also meet the requirements of the construction drawings and specifications for each specific underdrain construction event. Design flow capacity for the drainage geocomposite is estimated in Appendix IIID-C. The POR will ensure that the flow capacity of drainage geocomposite is equivalent to the required capacity estimated in Appendix IIID-C under similar loading conditions, by reviewing manufacturer's certification.

Refer to Section 4 of this LQCP for geocomposite testing and installation requirements.

6.4.5 Documentation

Dewatering system installation will be incorporated into the SLER for each liner construction event in accordance with Section 7. The installed dewatering system will be operated until a BER is prepared and approved by the TCEQ.

6.4.6 Dewatering System Operation

When pumps are used for the dewatering system, regardless of its location, they will be inspected on a weekly basis to monitor groundwater discharge at the pump outlet pipe. The pumps will be equipped with pressure transducers and the transducer readings will be recorded on a weekly basis to ensure that groundwater pressure in the sump is below the liner elevations. As an alternative to measuring groundwater levels with automatic pressure transducers, the groundwater levels in the dewatering sump may be checked manually by using a calibrated rod that will be lowered into the extraction riser or an equivalent method. The POR will identify the allowable head in the groundwater dewatering sump for each installation. All information/data generated associated with each groundwater dewatering operation will be kept in the site operating record. Each groundwater dewatering system installed will be operational until a BER is approved by the TCEQ.

6.5 Liner System Ballast

Ballasting is required to protect the liner system from hydrostatic uplift in areas of the landfill excavation which have been identified to exist below the highest measured groundwater potentiometric surface as defined in Section 6.2. The recompacted clay liner and protective cover soil, as well as additional waste placed above the liner system will provide the necessary ballast (weight) for protection of the liner system from hydrostatic uplift acting at the bottom of the recompacted clay liner.

The factor of safety against hydrostatic uplift must be calculated for those portions of the liner where the liner is below the highest groundwater potentiometric surface. The calculated factor of safety against uplift at the liner (using the weight of the protective cover and waste) must be 1.5. The thickness of ballast required to ballast the uplift force must be calculated and submitted with the SLER. Procedures for calculating the anticipated hydrostatic uplift forces, factor of safety against uplift, and required thickness of ballast are included in Appendix IIID-B. Additionally, example ballast calculations are included in Appendix IIID-B. The most recent highest measured groundwater elevation data as defined in Section 6.2 will be used for ballast calculations. 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. Note that a value of 1,200 pcy (45 pcf) for solid waste will be used for future ballast calculations.

6.5.1 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 operator 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. When waste is used as ballast the factor of safety against hydrostatic pressure uplift at the geomembrane liner will be 1.5. This documentation will be placed in the site operating record.

Additionally, the Site Manager will complete and sign a waste-as-ballast placement record that will be attached to the BER (see Section 7 for BER required documentation). 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 approved liner evaluation reports.

6.6 Liner Performance Verification

When ballast is required for a liner, the POR or his representative will verify that the ballast meets the established criteria and uplift or seeps through the liner system did not occur during construction. The verification, including but not limited to inspections, compaction, weight, density of material, thickness, and top elevations, will be documented in the BER, which will be submitted to the TCEQ for approval (see Section 7). In the event that uplift or seeps occur, the POR will develop a corrective action to remediate the uplift. The POR will immediately contact the TCEQ and implement initial procedures as soon as the uplift is detected.

6.6.1 Observations for Indications of Seepage

The POR or his representative 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 all seepage has been controlled.

6.7 Documentation

Documentation for issues related to construction below the groundwater elevation table will be included in the SLER and BER. These documents are discussed in detail in Section 7. Documentation specifically related to liners constructed below the highest measured groundwater potentiometric surface will include:

- A current highest measured potentiometric surface map and recent water-level information (Section 6.2).
- A discussion addressing the areas (if any) where the bottom of compacted clay liner extends below the highest measured potentiometric level.
- A discussion identifying the groundwater condition.
- Uplift and ballast calculations for liners with an installed dewatering system.
- A discussion addressing any seepage that may have been encountered.
- Description of the dewatering system installed.

The BER will contain the documentation substantiating that the appropriate depth of ballast has been placed over the liner system and that the liner did not experience hydrostatic uplift.

7 DOCUMENTATION

The quality assurance 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. Standard report forms will be provided by the POR prior to construction.

7.1 Preparation of SLER

The POR will submit to the TCEQ a SLER for review and acceptance of each clay liner construction event.

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

At a minimum, the SLER 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 areas of recompacted clay lining, underdrains, underdrain piping and sumps and sidewall risers. The POR will review and verify that as-built drawings are correct. As-built drawings will be included in the SLER.

7.2 Reporting Requirements

The SLER will be signed and sealed by the POR and signed by the Site Manager 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 no response is received, either oral or written, within 14 days of receipt by the Waste Permits Division of the TCEQ, the report will be considered accepted. Any notice of deficiency received from the TCEQ will be promptly addressed and incorporated into the SLER report. No solid waste will be placed over the constructed liner areas until the final acceptance is obtained from the TCEQ. Additionally, if a new liner area is constructed a pre-opening inspection will be requested of the TCEQ prior to accepting any solid waste into the newly constructed liner area. 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 waste placement, provided the SLER 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 within six months, then the POR will visually observe that the liner is not damaged (i.e., excessive erosion or dessication) due to prolonged exposure of the surface of the in-situ shale or clay liner protective cover and will submit a letter report of the findings to the TCEQ. 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.

7.3 Ballast Evaluation Report

Existing and future dewatering system BERs will be submitted in accordance with this section. A BER will be completed and filed with the TCEQ documenting that enough ballast has been placed in a lined area to offset the potential hydrostatic uplift forces which may exist below the liner system. At a minimum, the information listed below will be included as applicable with the BER.

- 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.

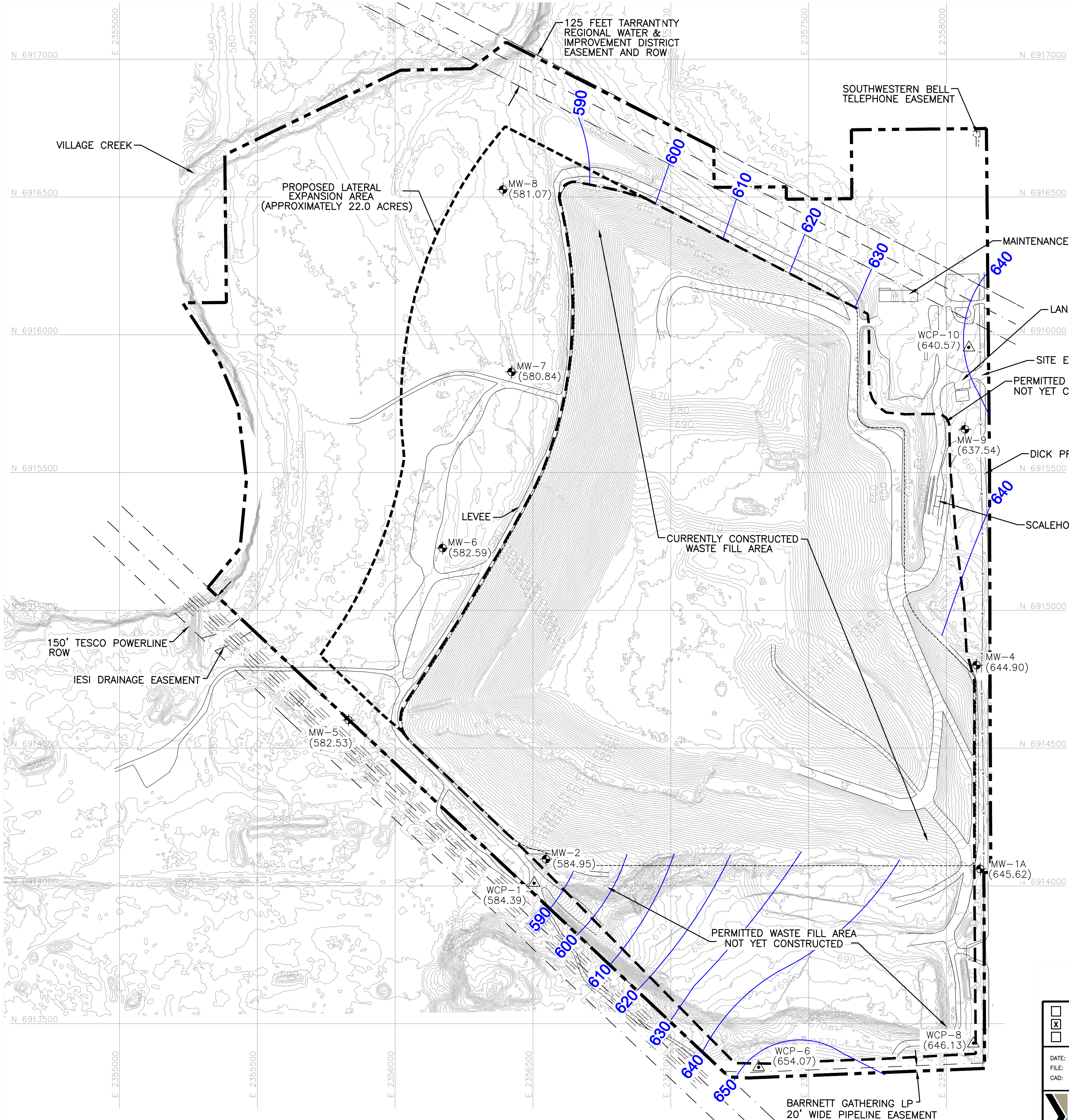
- If waste is used for ballast, documentation of the observations that the initial 5 feet of waste used for ballast on the liner system is free of brush and large bulky items, which may not be compacted to the required density.
- A waste-as-ballast placement record (Appendix IIID-D) completed and signed 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.
- 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.
- A BER will be prepared and signed and sealed by a professional engineer licensed to practice in Texas.

APPENDIX IIID-A

HIGHEST MEASURED GROUNDWATER MAP

STATE OF TEXAS
DAVID E. POE
81734
LICENSED
PROFESSIONAL ENGINEER

02/09/2023



N

0 200 400

SCALE IN FEET

LEGEND

- PERMIT BOUNDARY
- PERMITTED LIMITS OF WASTE
- PROPOSED LIMITS OF WASTE
- 630 — EXISTING CONTOUR
- N 6915000 — STATE PLANE COORDINATE
- MW-2 (583.95) — EXISTING GROUNDWATER MONITORING WELL
- WCP-1 (584.39) — EXISTING GROUNDWATER PIEZOMETER
- 640 — HIGHEST MEASURED GROUNDWATER ELEVATION SURFACE CONTOUR (SEE NOTE 4)

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS PROVIDED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN 02-17-2022.
2. MONITORING WELL AND PIEZOMETER LOCATION COORDINATES OBTAINED FROM BORING LOGS, INSTALLATION REPORTS, AND AS-BUILT SURVEY REPORTS.
3. GROUNDWATER ELEVATION DATA FOR MONITORING WELLS OBTAINED FROM THE FACILITY'S SUBTITLE D GROUNDWATER MONITORING DATABASE. GROUNDWATER DATA FOR PIEZOMETERS OBTAINED FROM INVESTIGATION REPORTS.
4. GROUNDWATER CONTOURS WERE PRODUCED USING EACH POINTS HISTORICALLY HIGHEST MEASURED GROUNDWATER ELEVATION AND DO NOT REPRESENT A SINGLE GROUNDWATER MONITORING EVENT OR ACTUAL GROUNDWATER FLOW.

Highest Measured Groundwater Data

Measurement Point	Groundwater Elevation (FT-MSL)	Measurment Date
MW-1A	645.62	7/19/2019
MW-2	583.95	5/17/2019
MW-4	644.90	5/17/2019
MW-5	582.53	6/23/2004
MW-6	582.89	6/21/2016
MW-7	580.84	6/1/2021
MW-8	581.07	6/1/2021
MW-9	637.54	5/17/2019
WCP-1	584.39	5/17/2019
WCP-6	654.07	6/25/2019
WCP-8	646.13	5/17/2019
WCP-10	640.57	5/17/2019

STATE OF TEXAS
DAVID E. POE
81734
LICENSED
PROFESSIONAL ENGINEER
02/09/2023

☐ DRAFT
☒ FOR PERMITTING PURPOSES ONLY
☐ ISSUED FOR CONSTRUCTION

DATE: 12/2022
FILE: 0771-356-11
CAD: IIID-A-1 HIGHEST GROUNDWATER.DWG

DRAWN BY: JDW
DESIGN BY: AKE
REVIEWED BY: CRM

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

PREPARED FOR
TEXAS REGIONAL LANDFILL COMPANY, LP

REVISIONS

NO.	DATE	DESCRIPTION

MAJOR PERMIT AMENDMENT
HIGHEST MEASURED GROUNDWATER ELEVATION MAP

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

WWW.WCGRP.COM

FIGURE IIID-A-1

O:\0771\356\EXPANSION 2022\PART III\IIID\IIID-A-1 HIGHEST GROUNDWATER.dwg, jwilson, 1:2

APPENDIX IIID-B

EXAMPLE BALLAST THICKNESS CALCULATIONS

Includes pages IIID-B-1 through IIID-B-7

STATE OF TEXAS
DAVID E. POE
81734
LICENSED
PROFESSIONAL ENGINEER

02/09/2023

EXAMPLE BALLAST CALCULATIONS

Introduction

For excavations extending below the groundwater potentiometric surface, the MSW Regulations require that long-term hydrostatic uplift pressures on the base of the floor or sidewall liner systems be offset using ballast in accordance with the regulations contained in 30 TAC §330.337. The hydrostatic uplift pressures on the liner system and the ballast requirements to offset the uplift pressures were evaluated and an example is included in this Appendix. The ballast calculations include an evaluation of the magnitude of the hydrostatic uplift pressures on the floor and sidewall liner systems based on the difference in elevation between the highest measured potentiometric surface and the base of the liner system. In addition, the resistance pressure of the proposed liner system was evaluated and compared to the hydrostatic uplift pressure to determine if additional ballast in the form of solid waste or soil will be necessary.

At the Fort Worth C&D Landfill, excavation below the potentiometric surface or through water bearing zones will be necessary in future Sectors 4, 5 and 6. An alluvium stratum, will be exposed within the sidewall excavation along the outer slopes of the Sectors which will require ballasting.

Short-term dewatering requirements for construction of the liner system and placement of required ballast are presented in Appendix IIID-C. The location of the Sidewall Dewatering Systems are shown on Figures IIID-C-1 through IIID-C-4.

Excavation Through Alluvium Stratum

The alluvium stratum and associated groundwater is exposed in the exterior sidewall along the outer slopes of Sectors 4, 5 and 6. The excavation base (floor) in this area will be in the shales underlying the alluvium and therefore will not be subjected to hydrostatic forces.

The hydrostatic pressures on the sidewall liner will be offset by the protective cover layers and additional solid waste or soil ballast above the liner system as required.

The waste ballast will be placed out onto the floor of the landfill a minimum horizontal distance of 100 feet from the toe of the slope before temporary dewatering methods are discontinued. The waste-as-ballast example calculations are provided in this appendix.

Ballast Evaluation Report

A Ballast Evaluation Report (BER) will be prepared for each landfill sector constructed below the seasonal high-water table. The BER will be prepared in accordance with the Section 7.3 of the LQCP.

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. 4 (Sector 4) 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-6. Sheet IIID-B-7 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 at base of recompacted clay liner, ft
P_{H20} = Maximum uplift pressure created by groundwater head, psf
$R_{pc, v}$ = Counteracting ballast pressure from clay liner and protective cover - vertical, psf
$R_{pc, n}$ = Counteracting ballast pressure from clay liner and protective cover - normal, psf
E_{H20} = Highest potentiometric surface elevation, ft-msl
E_{exc} = Elevation of excavation grade, 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 1,200 lb/cy per 30 TAC Section 330.337(h)(2))
$T_{pc, v}$ = Thickness of clay liner and protective cover as ballast - vertical, ft
$T_{pc, n}$ = Thickness of clay liner and 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 clay liner and protective cover installed - vertical
$FS_{pc, n}$ = Calculated factor of safety with clay liner and 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
$E_{top\ waste, v}$ = 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 including intermediate cover, ft (note this thickness is assumed the same for the vertical and normal directions)

Example calculation using Evaluation Point No. 4:

Parameters:

E_{H20} =	646.9	ft-msl	γ_{pc} =	120	pcf
E_{exc} =	631.7	ft-msl	γ_{waste} =	1,200	lb/cy
γ_{H20} =	62.4	pcf	$E_{fc,\,v}$ =	709.8	ft-msl
β = side slope angle =	18.43	degrees	$E_{fc,\,n}$ =	709.8	ft-msl
$\cos\beta$ =	0.9487		T_{fc} =	2	ft
$T_{pc,\,v}$ =	4.2	ft ($T_{pc,v}/\cos\beta$)			
$T_{pc,\,n}$ =	4.0	ft			

Calculate the maximum groundwater head at the base of the clay liner.

$H = E_{H20} - E_{liner}$
$H = 15.2$ ft

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

$P_{H2O} = (\gamma_{H2O} \times H)$
$P_{H2O} = 948$ psf

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

$R_{pc,\,v} = (\gamma_{pc} \times T_{pc,\,v})$
$R_{pc,\,v} = 504$ psf

$R_{pc,\,n} = (\gamma_{pc} \times T_{pc,\,n})$
$R_{pc,\,n} = 480$ psf

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

$FS_{pc,\,v} = R_{pc,\,v}/P_{H2O} =$ 0.5

$FS_{pc,\,n} = R_{pc,\,n}/P_{H2O} =$ 0.5

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.

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.

$T_{waste,\,v} = [(1.5 \times P_{H2O}) - R_{pc,\,v}]/\gamma_{waste}$
$T_{waste,\,v} =$ 20.7 ft

$E_{waste,\,v} = E_{exc} + T_{pc,\,v} + T_{waste,\,v}$
$E_{waste,\,v} =$ 656.6 ft-msl

$T_{waste,\,n} = [(1.5 \times P_{H2O}) - R_{pc,\,n}]/\gamma_{waste}$
$T_{waste,\,n} =$ 21.2 ft

$E_{waste,\,n} = E_{exc} + T_{pc,\,n} + T_{waste,\,n}$
$E_{waste,\,n} =$ 656.9 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.

$E_{top\ waste,\,v} = E_{fc,\,v} - T_{fc}$
$E_{top\ waste,\,v} =$ 707.8 ft-msl

$E_{top\ waste,\,v} > E_{waste,\,v}$
707.8 > 656.6

$E_{top\ waste,\,n} = E_{fc,\,n} - T_{fc}$
$E_{top\ waste,\,n} =$ 707.8 ft-msl

$E_{top\ waste,\,n} > E_{waste,\,n}$
707.8 > 656.9

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.

Prep By: MB
Date: 2/1/2023

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EXAMPLE BALLAST THICKNESS CALCULATIONS
EVALUATION OF SIDEWALL OF LINER

Chkd By: DEP
Date: 2/1/2023

Unit Weight of Water =	62.4	pcf
it Weight of Clay Liner/Protective Cover =	120	pcf
Unit Weight of Waste =	1200	pcy
Unit Weight of Final Cover =	120	pcf

Thickness of Clay Liner and Protective Cover - Vertical =	4.2	ft
Thickness of Clay Liner and Protective Cover - Normal =	4.0	ft
Thickness of Final Cover/Int Cover =	2.0	ft

Evaluation Point	Highest Potentiometric Surface Elevation E_{H2O} (ft-msl)	Excavation Grade E_{liner} (ft-msl)	Maximum Groundwater Head at Base of Clay Liner H (ft)	Maximum Uplift Pressure Created by Groundwater Head P_{H2O} (psf)	Elevation of Top of Clay Liner/ Protective Cover - Vertical $E_{pc,v}$ (ft-msl)	Elevation of Top of Protective Cover - Normal $E_{pc,n}$ (ft-msl)	Counteracting Ballast Pressure from Clay Liner/ Protective Cover - Vertical $R_{pc,v}$ (psf)	Counteracting Ballast Pressure from Clay Liner/ Protective Cover - Normal $R_{pc,n}$ (psf)	Calculated Factor of Safety with Clay Liner/ Protective Cover Installed - Vertical	Calculated Factor of Safety with Clay Liner/ 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 [2]	575.0	570.0	5.0	312	574.2	574.0	504	480	1.6	1.5	YES	YES	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
2 [2]	571.0	570.0	1.0	62	574.2	574.0	504	480	8.1	7.7	YES	YES	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
3	615.0	571.0	44.0	2,746	575.2	575.0	504	480	0.2	0.2	NO	NO	81.3	81.9	656.5	656.9	688.4	688.4	YES	YES	5,775	5,760	2.1	2.1	YES	YES
4	646.9	631.7	15.2	948	635.9	635.7	504	480	0.5	0.5	NO	NO	20.7	21.2	656.6	656.9	707.8	707.8	YES	YES	3,940	3,924	4.2	4.1	YES	YES
5	643.8	630.5	13.3	830	634.7	634.5	504	480	0.6	0.6	NO	NO	16.7	17.2	651.4	651.7	714.5	714.5	YES	YES	4,291	4,276	5.2	5.2	YES	YES
6	640.0	630.0	10.0	624	634.2	634.0	504	480	0.8	0.8	NO	NO	9.7	10.3	643.9	644.3	662.7	662.7	YES	YES	2,011	1,996	3.2	3.2	YES	YES
7	635.2	630.0	5.2	324	634.2	634.0	504	480	1.6	1.5	YES	YES	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

[1] Refer to Sheet IIID-B-7 for the highest measured groundwater contours.
[2] Highest measured groundwater elevation used in calculation adjusted downward to account for dewatering into borrow excavation located west of perimeter berm.

N

0 200 400

SCALE IN FEET

LEGEND

	PERMIT BOUNDARY
	PERMITTED LIMIT OF WASTE
	PROPOSED LIMIT OF WASTE
630	EXISTING CONTOUR
N 6915000	STATE PLANE COORDINATE
	SECTOR BOUNDARY
550	PROPOSED EXCAVATION CONTOUR
570	PERMITTED EXCAVATION CONTOUR
MW-2A	PERMITTED GROUNDWATER MONITORING WELL
MW-9A	PROPOSED GROUNDWATER MONITORING WELL
MW-4	PERMITTED GROUNDWATER MONITORING WELL (TO BE ABANDONED)
GMP-4B	PERMITTED GAS MONITORING PROBE
	LIMITS OF GEOCOMPOSITE UNDERDRAIN (SEE NOTE 4)
	SECTOR 6 ALLUVIUM REQUIRING BALLASTING
640	HIGHEST MEASURED GROUNDWATER CONTOUR
1 N/A	BALLAST EVALUATION POINT (SEE NOTE 5)

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS PROVIDED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN 02-17-2022.
2. THE EXPANSION LIMIT OF WASTE IS LOCATED A MINIMUM OF 125-FT FROM THE PERMIT BOUNDARY.
3. REFER TO "ENVIRONMENTAL MONITORING" SECTION FOR INFORMATION REGARDING LANDFILL PERIMETER GROUNDWATER AND LFG MONITORING NETWORKS.
4. WATER BEARING ALLUVIUM LIMITS TO BE CONFIRMED BY POR DURING CELL EXCAVATION.
5. BALLAST EVALUATION POINT IS SHOWN WITH CALCULATED TOP OF WASTE ELEVATION REQUIRED FOR BALLAST IN FT-MSL. N/A INDICATES THAT RECOMPACTED CLAY LINER AND PROTECTIVE COVER IS SUFFICIENT BALLAST.

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02/09/2023

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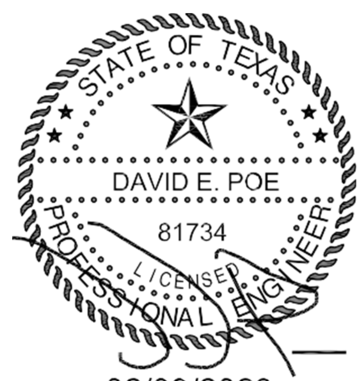
APPENDIX IIID-C

TEMPORARY DEWATERING SYSTEM DESIGN

Includes pages IIID-C-1 through IIID-C-53



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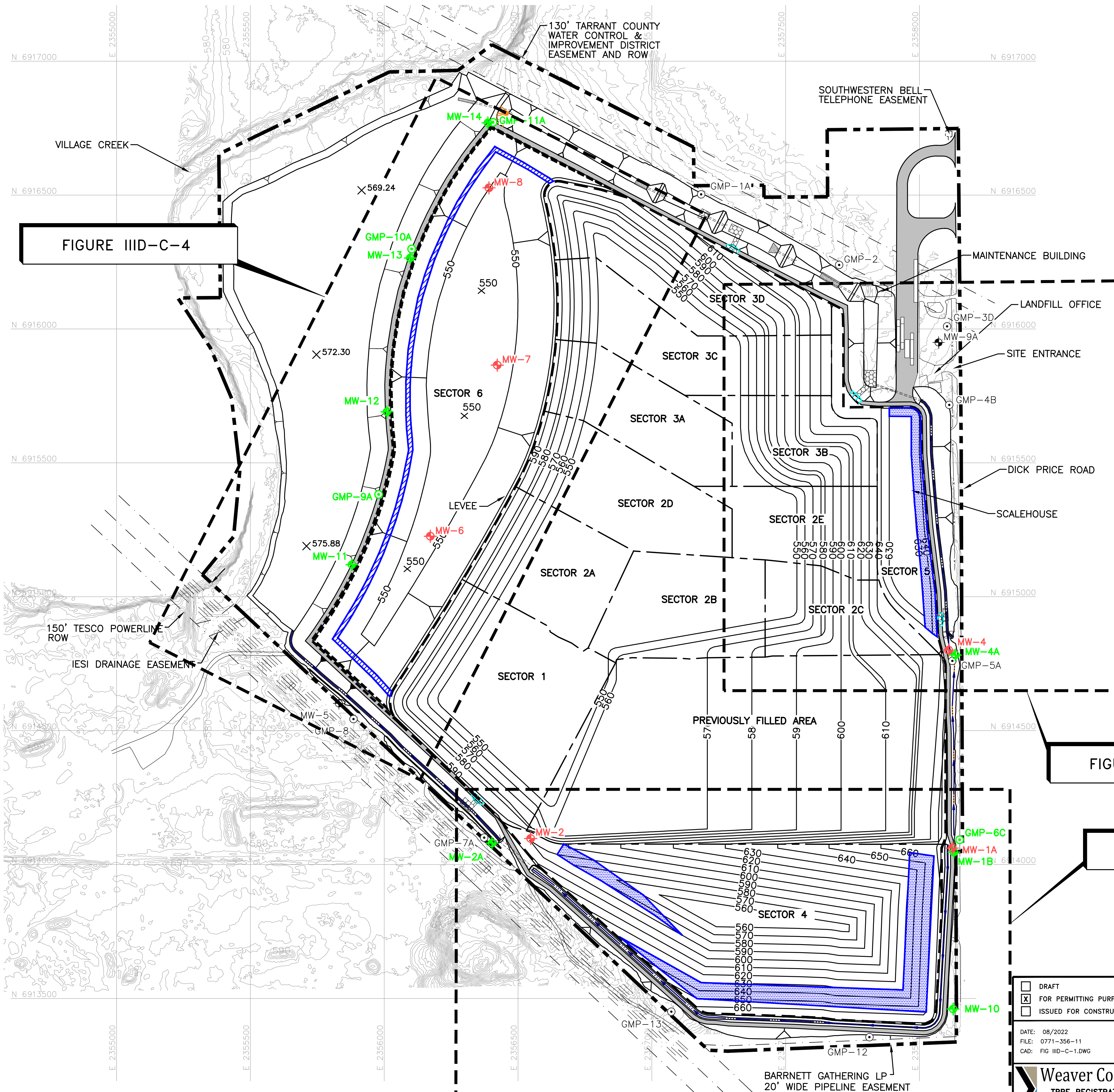
INTRODUCTION

The following temporary dewatering system design provides demonstration of the adequacy of the dewatering underdrain proposed to be installed in the sidewalls of Sectors 4, 5 and 6. The following is noted in support of the calculations presented in this appendix:

- The calculations presented in this appendix are applicable to the remaining sidewall drains to be installed in the remaining Sectors 4 and 5.
- Geocomposite drainage layer will be installed up a minimum 8 vertical feet (approximately 25 feet of geocomposite on the sidewall slope) from the sidewall drainage pipe trench in Sector 4; and the entire exterior sidewall slope in Sector 5.
- All geocomposite will be installed on the 3H:1V sidewall, and anchored at the bottom into the drainage pipe trench. Anchoring of the geocomposite on the sidewall at the top is not required, as demonstrated by the infinite slope stability analysis included in this appendix. The actual locations of sidewall geocomposite will be field-verified by the POR based on observations of the excavated slopes, and the physical identification of the porous alluvium layer within the sidewall.
- 4-inch-diameter HDPE pipe trench drain will be designed at the approximate locations shown on Figures IIID-C-2 and 3, and will be field-verified by POR during field inspection of alluvium boundaries at the time of excavation.
- The geocomposite and pipe capacity demonstrations presented in this appendix were prepared for a 20 foot (vertical) water table depth draining into the geocomposite. The pipe demonstration calculations were performed assuming an 1,800-foot-long, 4-inch-diameter collection pipe installed with a slope of 0.5 percent. These are conservative assumptions as the groundwater table at the sidewall interface will decrease after excavation and during dewatering system operation due to dewatering of the alluvium formation into the excavation.



LEGEND

- PERMIT BOUNDARY
- PERMITTED LIMIT OF WASTE
- PROPOSED LIMIT OF WASTE
- 630 EXISTING CONTOUR
- N 6915000 STATE PLANE COORDINATE
- SECTOR BOUNDARY
- 550 PROPOSED EXCAVATION CONTOUR
- 570 PERMITTED EXCAVATION CONTOUR
- MW-2A PERMITTED GROUNDWATER MONITORING WELL
- MW-9A PROPOSED GROUNDWATER MONITORING WELL
- MW-4 PERMITTED GROUNDWATER MONITORING WELL (TO BE ABANDONED)
- GMP-4B PERMITTED GAS MONITORING PROBE
- LIMITS OF GEOCOMPOSITE UNDERDRAIN (SEE NOTE 4)
- SECTOR 6 ALLUVIUM REQUIRING BALLASTING

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS PROVIDED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN 02-17-2022.
2. THE EXPANSION LIMIT OF WASTE IS LOCATED A MINIMUM OF 125-FT FROM THE PERMIT BOUNDARY.
3. REFER TO "ENVIRONMENTAL MONITORING" SECTION FOR INFORMATION REGARDING LANDFILL PERIMETER GROUNDWATER AND LFG MONITORING NETWORKS.
4. WATER BEARING ALLUVIUM LIMITS TO BE CONFIRMED BY POR DURING CELL EXCAVATION.

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FIGURE IIID-C-1

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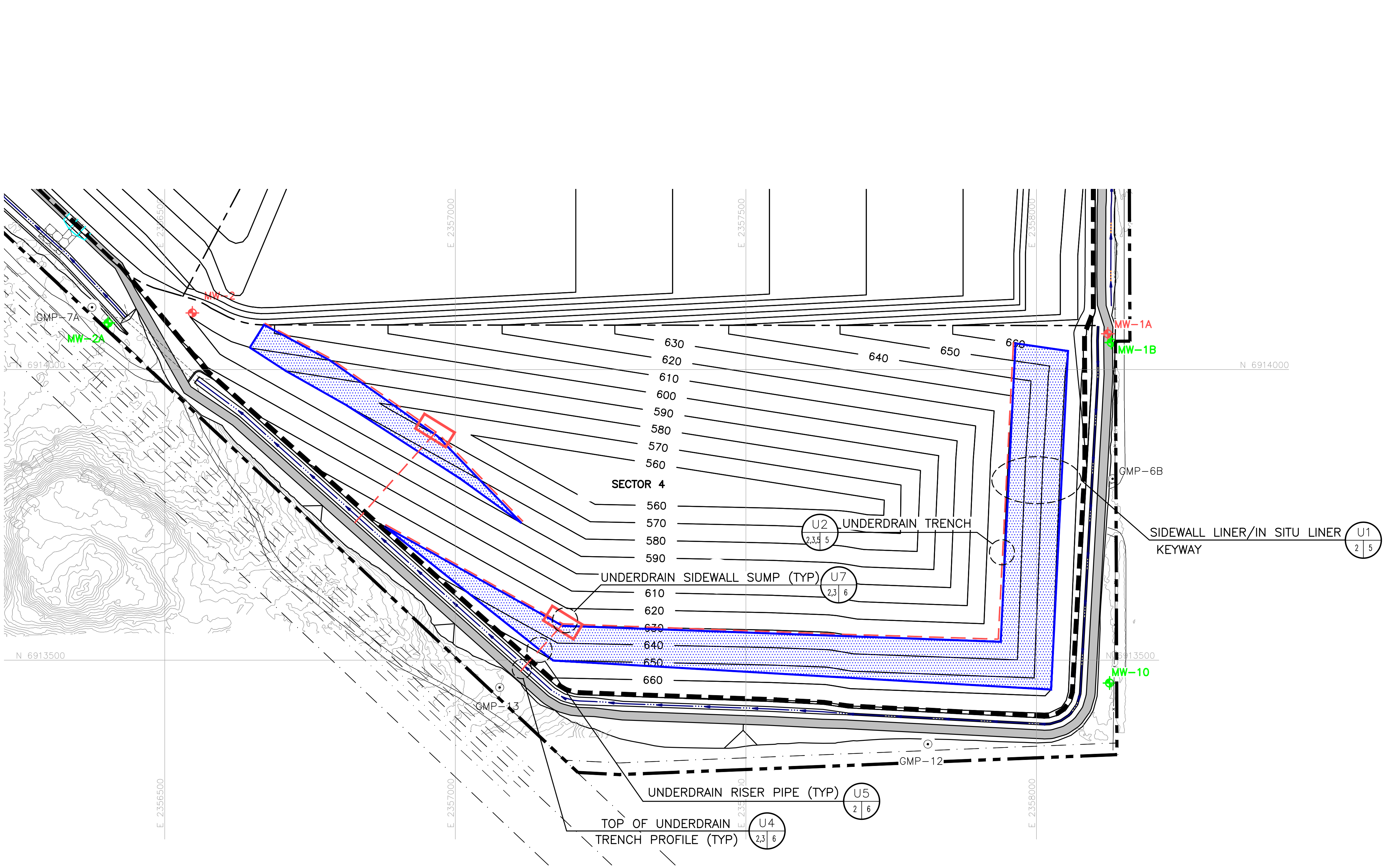
O:\0771\356\EXPANSION 2022\PART III\IIID\FIGURE IIID-C-1.dwg, jwilson, 1:2



LEGEND

- PERMIT BOUNDARY
- PERMITTED LIMIT OF WASTE
- PROPOSED LIMIT OF WASTE
- 630 EXISTING CONTOUR
- N 6915000 STATE PLANE COORDINATE
- SECTOR BOUNDARY
- 560 PROPOSED BASE GRADE (SEE NOTE 4)
- 570 PERMITTED BASE GRADE (SEE NOTE 4)
- MW-2A EXISTING/PERMITTED GROUNDWATER MONITORING WELL
- MW-9A APPROVED GROUNDWATER MONITORING WELL
- MW-4 PERMITTED GROUNDWATER MONITORING WELL (TO BE ABANDONED)
- GMP-4B PERMITTED GAS MONITORING PROBE
- LIMITS OF GEOCOMPOSITE UNDERDRAIN (SEE NOTE 5)
- UNDERDRAIN TRENCH
- UNDERDRAIN SIDEWALL SUMP

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS PROVIDED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN 02-17-2022.
2. THE EXPANSION LIMIT OF WASTE IS LOCATED A MINIMUM OF 125-FT FROM THE PERMIT BOUNDARY.
3. REFER TO "ENVIRONMENTAL MONITORING" SECTION FOR INFORMATION REGARDING LANDFILL PERIMETER GROUNDWATER AND LFG MONITORING NETWORKS.
4. BASE GRADES SHOWN ARE THE TOP OF PROTECTIVE COVER COMPONENT OF THE CONSTRUCTED LINER SYSTEM OR THE TOP OF THE IN-SITU LINER SYSTEM. THE ELEVATION OF DEEPEST EXCAVATION (EDE) FOR THE FACILITY IS 550 FT, MSL BASED ON USE OF IN-SITU LINER (OR ELEVATION 546 FT, MSL IF CONSTRUCTED LINER IS USED).
5. WATER BEARING ALLUVIUM LIMITS TO BE CONFIRMED BY POR DURING CELL EXCAVATION.

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FIGURE IIID-C-2

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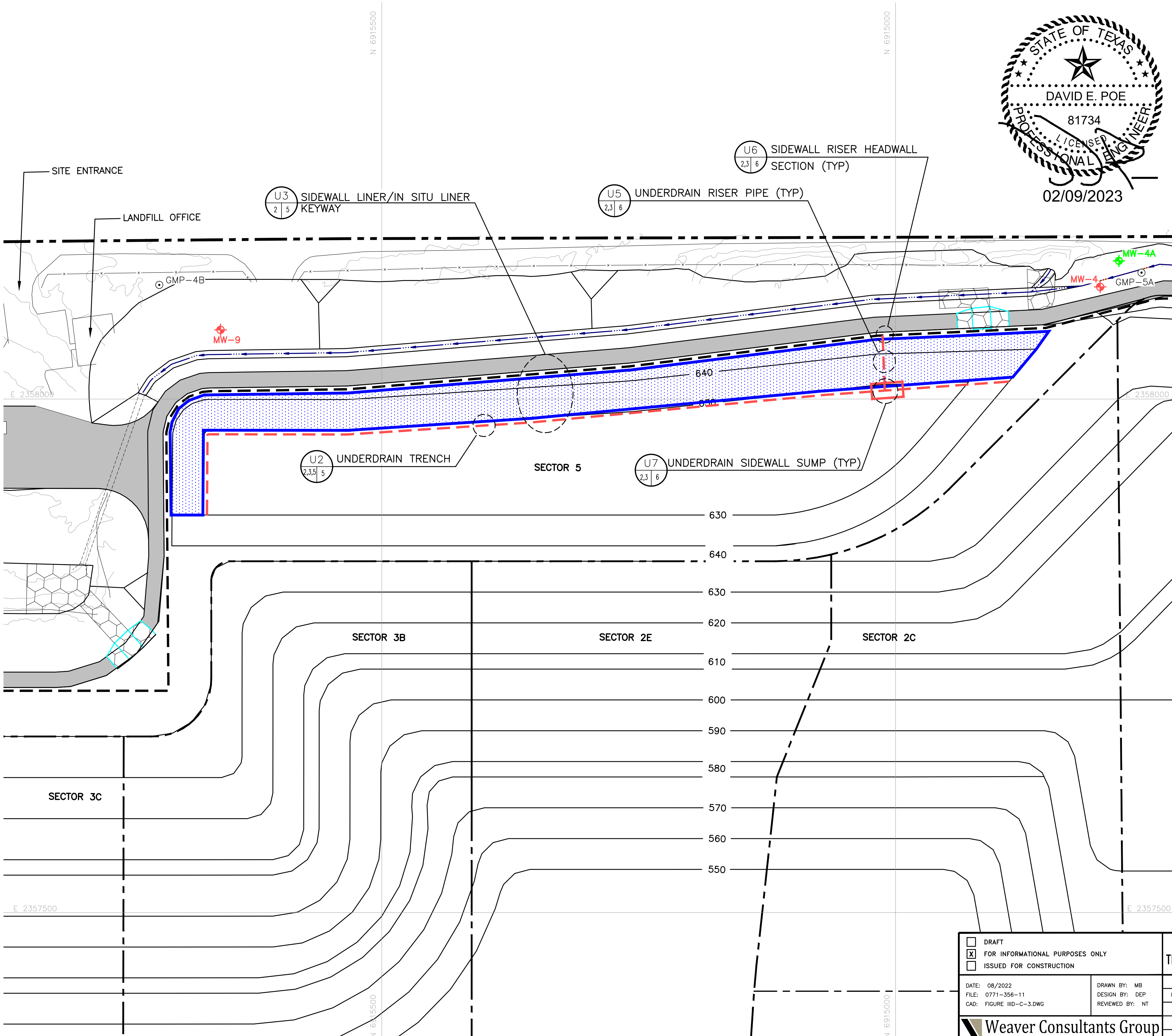
O:\0771\356\EXPANSION 2022\PART III\IIID\FIGURE IIID-C-2.dwg, jwilson, 1:2



LEGEND

- PERMIT BOUNDARY
- PERMITTED LIMIT OF WASTE
- PROPOSED LIMIT OF WASTE
- 630 EXISTING CONTOUR
- N 6915000 STATE PLANE COORDINATE
- SECTOR BOUNDARY
- 570 PERMITTED BASE GRADE (SEE NOTE 4)
- MW-2A EXISTING/PERMITTED GROUNDWATER MONITORING WELL
- MW-9A APPROVED GROUNDWATER MONITORING WELL
- MW-4 PERMITTED GROUNDWATER MONITORING WELL (TO BE ABANDONED)
- GMP-4B PERMITTED GAS MONITORING PROBE
- LIMITS OF GEOCOMPOSITE UNDERDRAIN (SEE NOTE 5)
- UNDERDRAIN TRENCH
- UNDERDRAIN SIDEWALL SUMP

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS PROVIDED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN 02-17-2022.
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4. BASE GRADES SHOWN ARE THE TOP OF PROTECTIVE COVER COMPONENT OF THE CONSTRUCTED LINER SYSTEM OR THE TOP OF THE IN-SITU LINER SYSTEM. THE ELEVATION OF DEEPEST EXCAVATION (EDE) FOR THE FACILITY IS 550 FT, MSL BASED ON USE OF IN-SITU LINER (OR ELEVATION 546 FT, MSL IF CONSTRUCTED LINER IS USED).
5. WATER BEARING ALLUVIUM LIMITS TO BE CONFIRMED BY POR DURING CELL EXCAVATION.

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FIGURE IIID-C-3

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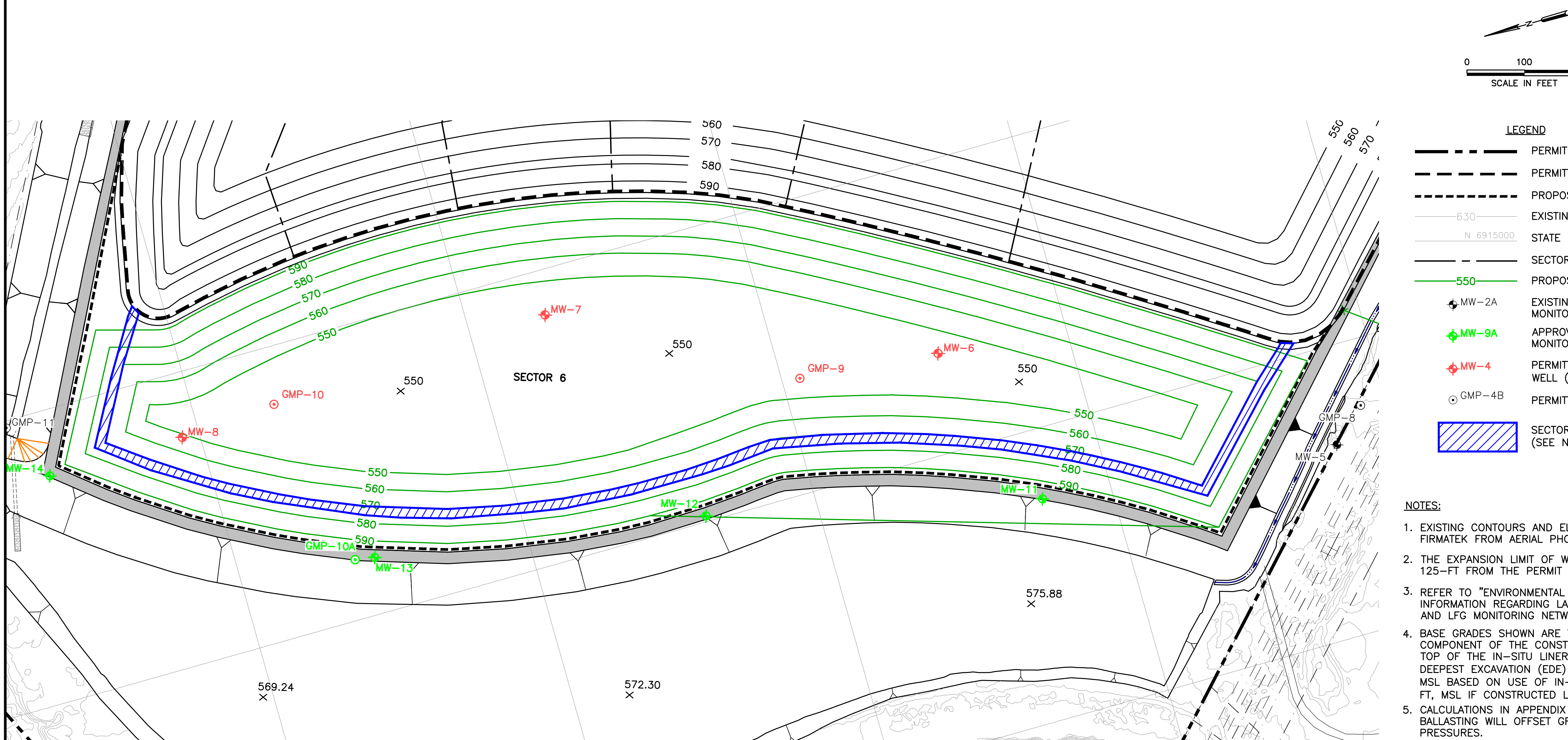
O:\0771\356\EXPANSION 2022\PART III\IIID\FIGURE IIID-C-3.dwg, jwilson, 1:2



LEGEND

- PERMIT BOUNDARY
- PERMITTED LIMIT OF WASTE
- PROPOSED LIMIT OF WASTE
- 630 EXISTING CONTOUR
- N 6915000 STATE PLANE COORDINATE
- SECTOR BOUNDARY
- 550 PROPOSED BASE GRADE (SEE NOTE 4)
- MW-2A EXISTING/PERMITTED GROUNDWATER MONITORING WELL
- MW-9A APPROVED GROUNDWATER MONITORING WELL
- MW-4 PERMITTED GROUNDWATER MONITORING WELL (TO BE ABANDONED)
- GMP-4B PERMITTED GAS MONITORING PROBE
- SECTOR 6 ALLUVIUM REQUIRING BALLASTING (SEE NOTE 5)

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS PROVIDED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN 02-17-2022.
2. THE EXPANSION LIMIT OF WASTE IS LOCATED A MINIMUM OF 125-FT FROM THE PERMIT BOUNDARY.
3. REFER TO "ENVIRONMENTAL MONITORING" SECTION FOR INFORMATION REGARDING LANDFILL PERIMETER GROUNDWATER AND LFG MONITORING NETWORKS.
4. BASE GRADES SHOWN ARE THE TOP OF PROTECTIVE COVER COMPONENT OF THE CONSTRUCTED LINER SYSTEM OR THE TOP OF THE IN-SITU LINER SYSTEM. THE ELEVATION OF DEEPEST EXCAVATION (EDE) FOR THE FACILITY IS 550 FT, MSL BASED ON USE OF IN-SITU LINER (OR ELEVATION 546 FT, MSL IF CONSTRUCTED LINER IS USED).
5. CALCULATIONS IN APPENDIX IIID-C DEMONSTRATE THAT THE LINER BALLASTING WILL OFFSET GROUNDWATER PIEZOMETRIC UPLIFT PRESSURES.

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DAVID E. POE
81734
LICENSED
PROFESSIONAL ENGINEER
02/09/2023

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DATE: 08/2022
FILE: 0771-356-11
CAD: FIGURE IIID-C-4.DWG

DRAWN BY: MB
DESIGN BY: DEP
REVIEWED BY: NT

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

PREPARED FOR
TEXAS REGIONAL LANDFILL COMPANY, LP

REVISIONS

NO.	DATE	DESCRIPTION

MAJOR PERMIT AMENDMENT
BALLAST PLAN
SECTOR 6
FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

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FIGURE IIID-C-4

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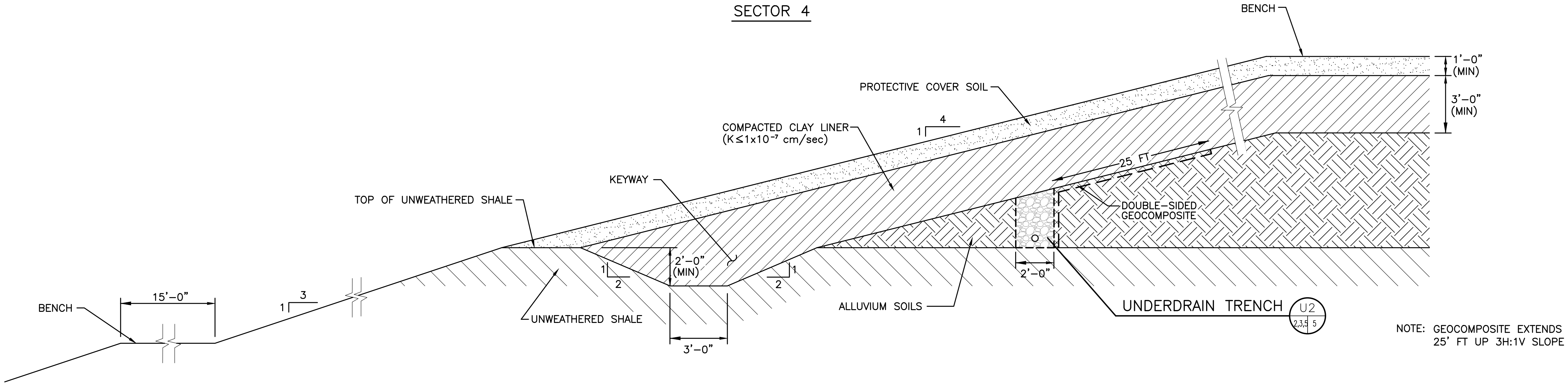
O:\0771\356\EXPANSION 2022\PART III\IIID\FIGURE IIID-C-4.dwg, jwilson, 1:2

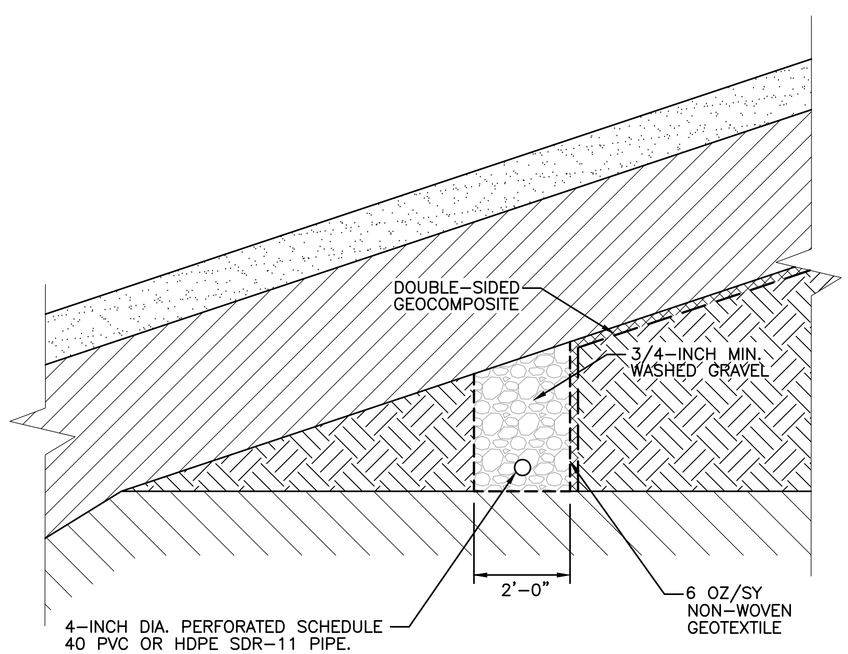
SECTOR 4



NOTE: GEOCOMPOSITE EXTENDS 25' FT UP 3H:1V SLOPE

SIDEWALL LINER/IN-SITU LINER KEYWAY U1 2 5

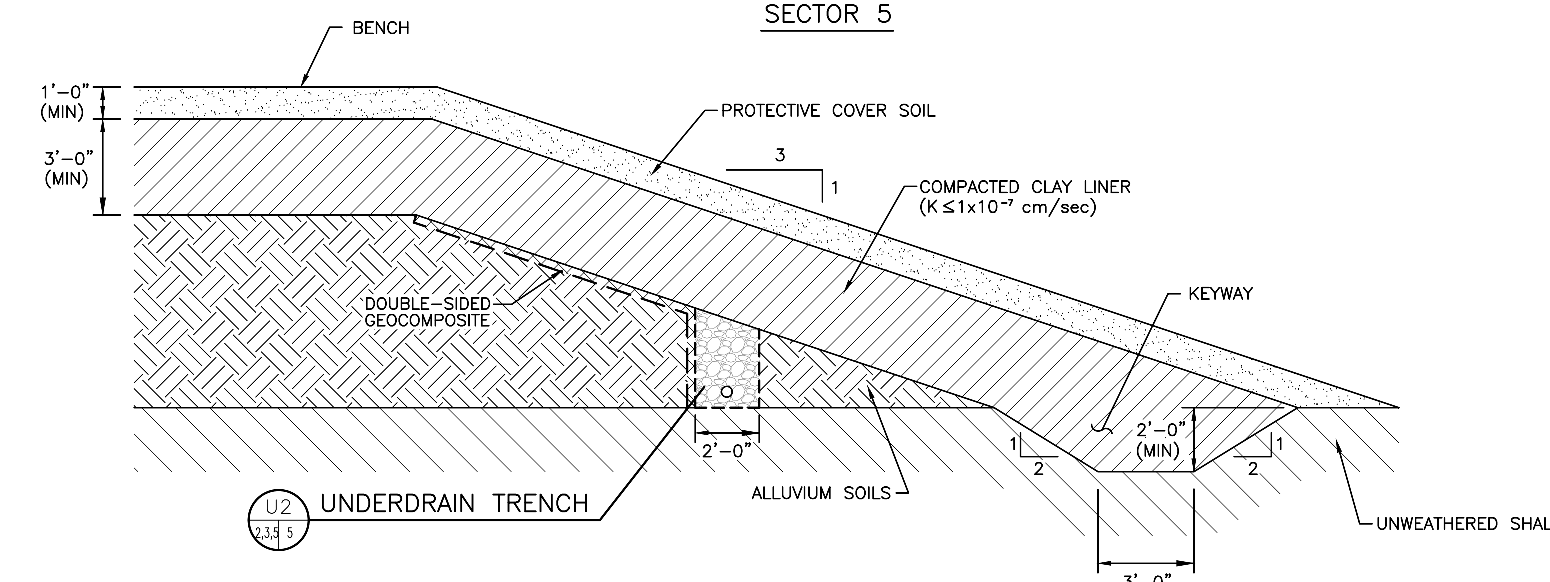
NTS



UNDERDRAIN TRENCH U2 2,3,5 5

0 2 4
SCALE IN FEET

SECTOR 5



SIDEWALL LINER/IN-SITU LINER KEYWAY U3 3 5

NTS

NOTE: GEOCOMPOSITE EXTENDS FROM TOE OF SLOPE TO TOP OF SLOPE

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DATE: 08/2022	DRAWN BY: SRF	
FILE: 0771-356-11	DESIGN BY: DEP	
CAD: FIGURE IIID-C-5.DWG	REVIEWED BY: NT	

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NO.	DATE	DESCRIPTION

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MAJOR PERMIT AMENDMENT
LINER SYSTEM/UNDERDRAIN DETAILS

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

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FIGURE IIID-C-5

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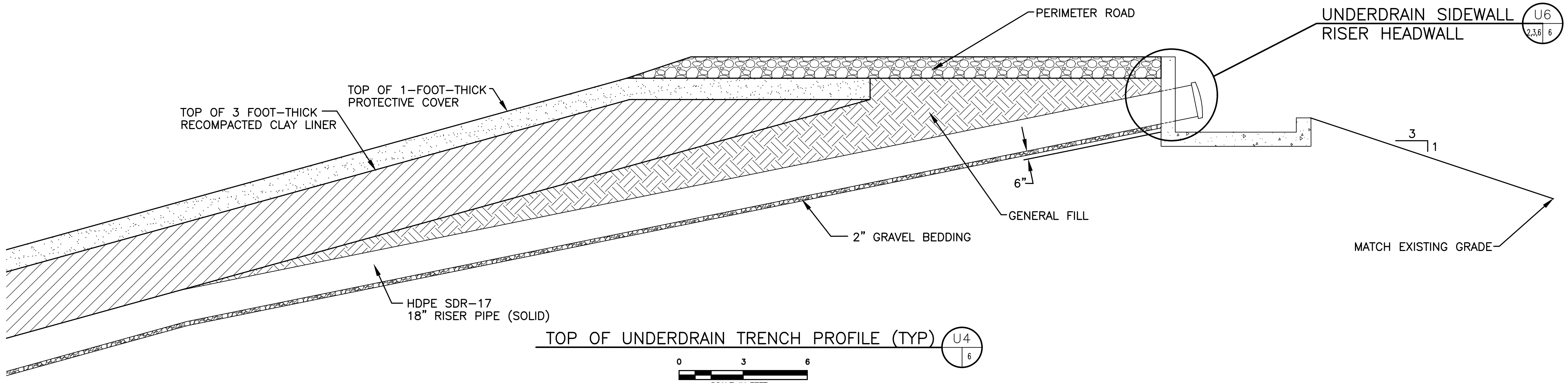
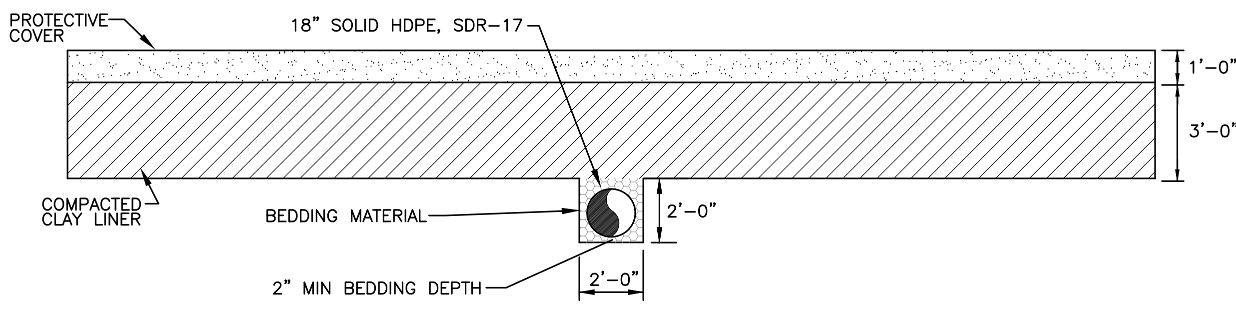
O:\0771\356\EXPANSION 2022\PART III\IIID\FIGURE IIID-C-5.dwg, jwilson, 1:2



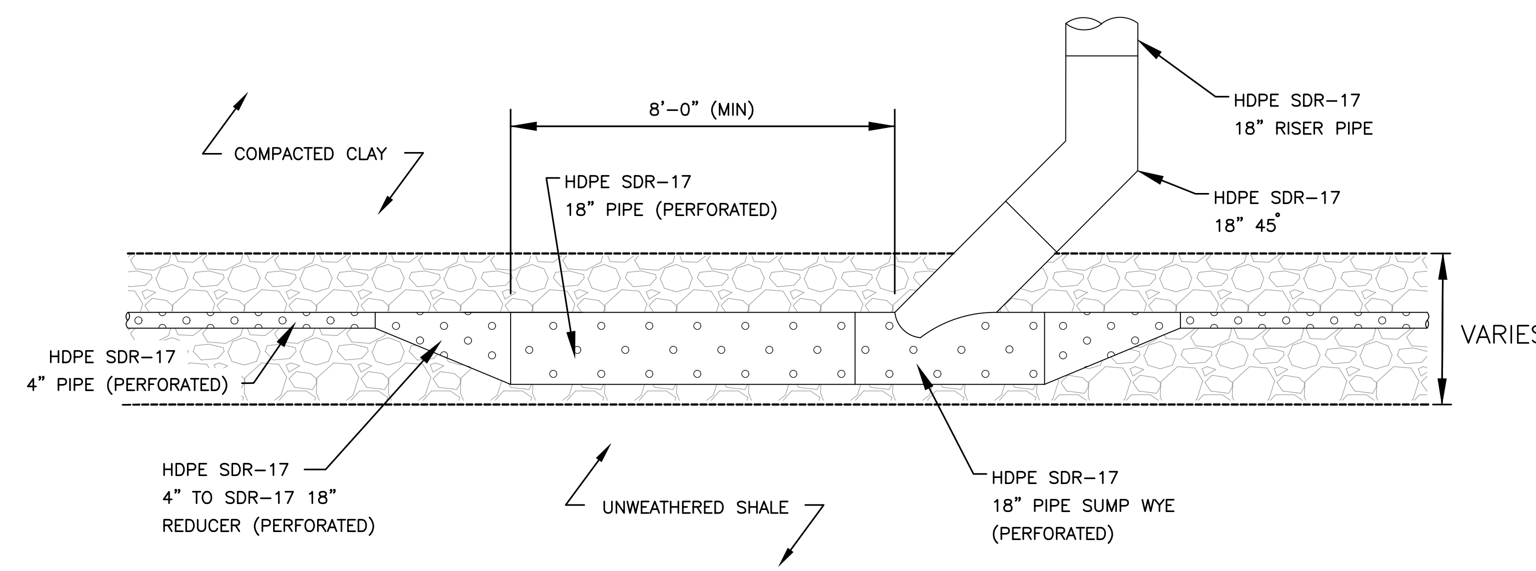
TOP OF UNDERDRAIN TRENCH PROFILE (TYP) U4 6

0 3 6
SCALE IN FEET



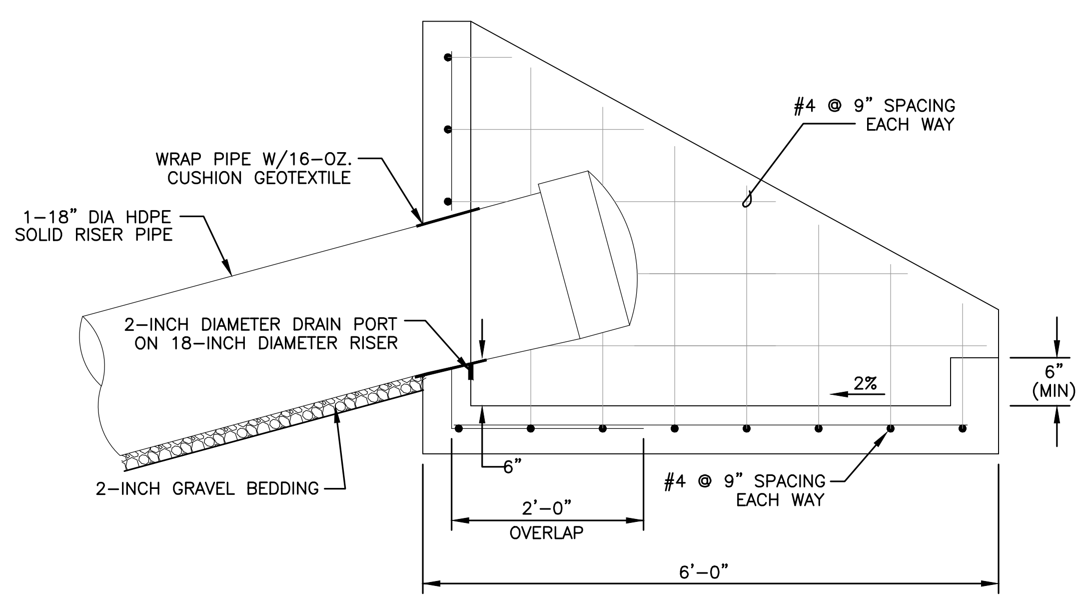
UNDERDRAIN RISER PIPE U5 2,3 6

0 3 6



UNDERDRAIN SIDEWALL SUMP(TYP) U7 2,3 6

0 2 4
SCALE IN FEET



SIDWALL RISER HEADWALL SECTION U6 2,3,6 6

0 1 2
SCALE IN FEET

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DATE: 08/2022 FILE: 0771-356-11 CAD: FIGURE IIID-C-6.DWG	DRAWN BY: SRF DESIGN BY: DEP REVIEWED BY: NT	REVISIONS NO. DATE DESCRIPTION	FORT WORTH C&D LANDFILL TARRANT COUNTY, TEXAS
Weaver Consultants Group TBPE REGISTRATION NO. F-3727		WWW.WCGRP.COM	FIGURE IIID-C-6

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O:\0771\356\EXPANSION 2022\PART III\IIID\FIGURE IIID-C-6.dwg, jwilson, 1:2

UNDERDRAIN DEWATERING SYSTEM GEOCOMPOSITE AND PIPING ANALYSIS (SECTORS 4 AND 5)

FORT WORTH C & D LANDFILL
APPENDIX IIID-C
UNDERDRAIN DEWATERING SYSTEM
GEOCOMPOSITE AND
PIPING ANALYSIS (SECTORS 4 AND 5)

Required

The purpose of these calculations is to demonstrate the adquacy of the sidewall underdrain dewatering systems proposed for Sectors 4 and 5. Future Sector 6 is not analyzed for this demonstration, as it is intended that the sidewall liner will provide sufficient ballasting of the groundwater potentiometric pressures at the time the recompacted clay liner is installed.

The underdrain systems are designed to provide hydrostatic pressure relief below the Type IV liner system, for areas not founded in the native shale formation.

Assumptions

For the 3H:1V cell sideslopes the calculations were performed assuming a 20 foot (vertical) perched groundwater table acting on the the 3H:1V sidewalls. For the analysis, the 20-foot-thick water bearing formation was used for the geocomposite and drainage pipe calculations.

The analysis assumes that the high hydraulic conductivity of the alluvium stratum layer will result dewatering of the formation at the time of excavation, prior to installation of the underdrain system or sidewall clay liner, resulting in a reduction of the length of sidewall subjected to groundwater uplift with time. At the time of construction, the the location of the underdrain installed on the sideslopes will be a minimum 8 feet vertical (or approximately 25 feet of 3H:1V slope length) up the sidewall beyond the shale/alluvium contact, as shown on Figures IIID-C-5 and 6.

The trench drain for the underdrain will be installed at the alluvium/shale interface in Sectors 4 and 5. A geotechnical engineer representing the design engineer will observe the excavated sidewalls (with alluvium removed in preparation of the the underdrain and 3-foot-thick recompacted clay liner installation) and the sidewall pipe trench and sidewall sumps will be located in the field based on observations of the POR. Multiple sidewall sumps may be required.

The overburden pressure causing compression of the geocomposite layer for the sidewall analysis was limited to 2 times the 20-foot-high perched groundwater hydrostatic pressure acting on the sidewall that requires ballasting. Additional compression of the geocomposite resulting from overburden pressure greater than the required ballasting pressure was not considered in calculating the geocomposite flow capacity, as the groundwater potentiometric pressures will be sufficiently ballasted at higher loading.

Demonstration of the structural stability of the underdrain dewatering piping (4 and 18-inch diameter PE pipe) is provided separately in this appendix.

Method

1. Estimate the hydraulic conductivity of the alluvium stratum identified in the Appendix IIIG - Geology Report
2. Estimate the flow into the geocomposite drainage layer.
3. Determine the flow capacity of the geocomposite drainage layer.
4. Compare geocomposite flow capacity with inflow to determine suitability of selected geocomposite.
5. Estimate the flow into the 4-inch-diameter dewatering pipe.
6. Determine the flow capacity of the dewatering pipe.
7. Determine required pipe perforation based on characteristics of the surrounding drainage media.
8. Evaluate the storage capacity and pump cycling for the sump.

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* , second edition, Prentice Hall, Inc., 1990.
4. GSE Drainage Design Manual, May 2004.
5. *Dewatering and Groundwater Control,* TM5-818-5, November 1983.
6. Phillips 66 Driscopipe, System Design, 1991.
7. Acar, Yalcin B.& Daniel, David E., *Geoenvironment 2000 Characterization, Containment, Remediation, and Performance in Environmental Geotechnics, Volume 2, American Society of Civil Engineers,* 1995.
8. Gray, Donald H., Koerner, Robert M., Qian, Xuede, Geotechnical Aspects of Landfill Design and Construction, 2002.
9. Geosynthetic Institute, GRI Standard GC-8, 2001.

Solution

1. **Estimate the flow into the geocomposite drainage layer - Landfill Sidewalls (Sectors 4 and 5)**

The maximum flow length of dewatering geocomposite on the sideslope to a toe trench is approximately 25 feet (based on the geocomposite intercepting the lower 8 feet of the 20-foot-thick water-bearing alluvium on the 3H:1V sidewall). This assumes that the sidewall underdrain dewatering layer discharges into a toe drain installed at the base of the alluvium stratum (i.e., at the alluvium/shale interface).

$Q = kiA$

where:

Q =	groundwater inflow rate into geocomposite (cfs/ft)
k =	For the cell sideslope analysis it was assumed that a 20-foot (vertical) layer of alluvial is exposed in the excavation, with flow into the underdrain system calculated based on a horizontal hydraulic conductivity of the formation times the height of the water bearing formation. Hydraulic conductivity for the alluvium stratum based on geometric mean reported in the Appendix IIIG - Geology Report.
i =	gradient (ft/ft). For the sideslope calcuations and perched groundwater conditions, it was assumed that the gradient is represented the measured hydraulic gradient of the water bearing formation, or 0.009 ft/ft as discussed in the Appendix IIIG - Geology Report.
L=	Height of water bearing formation, or 20 feet for this analysis.
A =	inflow area (sf) (area per unit width of dewatering)

k =	4.62E-03	cm/s =	1.52E-04	ft/s
i =	0.009	ft/ft		
L=	20	ft (vertical height of formation subjected to horiz. flow)		
A =	20	sf' (Height of water bearing formation flowing horizontally multiplied by a unit width of 1 foot)		

	$Q_{max, sidewall}$ =	2.73E-05	cfs/ft width of geocomposite

2. **Determine the flow capacity of the geocomposite drainage layer - Landfill Sidewalls (Sectors 4 and 5)**

Assume the geocomposite leachate collection layer will undergo compression due to the weight of liner, protective cover, and waste.

Unloaded Geocomposite Thickness (200 mil) =	0.20	in
Unit Weight of Soil =	120	pcf

FORT WORTH C & D LANDFILL
APPENDIX IIID-C
UNDERDRAIN DEWATERING SYSTEM
SIDESLOPE TYPICAL GEOCOMPOSITE AND PIPING ANALYSIS (SECTORS 4 AND 5)

Table 1 - Geocomposite Thickness

Fill Condition	d_W[1] (ft)	d_S[2] (ft)	γ[3] (pcf)	P[4] (psf)	t[5] (in)	t[5] (m)
Liner and Protective Cover Layers Installed	0	4.5	120	540	0.199	0.0051
Waste Thickness - Sidewall (see Note 4 below)	25	4.5	80	2,540	0.192	0.0049

1 d_W is the depth of waste and daily cover soil above the geocomposite underdrain collection layer.

2 d_S is the depth of soil (protective cover, intermediate cover, and final cover) above the geocomposite underdrain collection layer.

3 The unit weight of waste/soil is selected at the midpoint of the waste column thickness using the Unit Weight Profile for MSW graph provided in Ref 5.

4 P is the pressure on the geocomposite underdrain collection layer due to the weight of the waste and soil. This value has been back-calculated into a value representative of approximately 40 feet hydraulic uplift (2X actual value of 20 feet) acting on sideslope liner system.

5 t is the thickness of the geocomposite underdrain collection layer after being subjected to compression based on the chart below adapted from Reference 7.

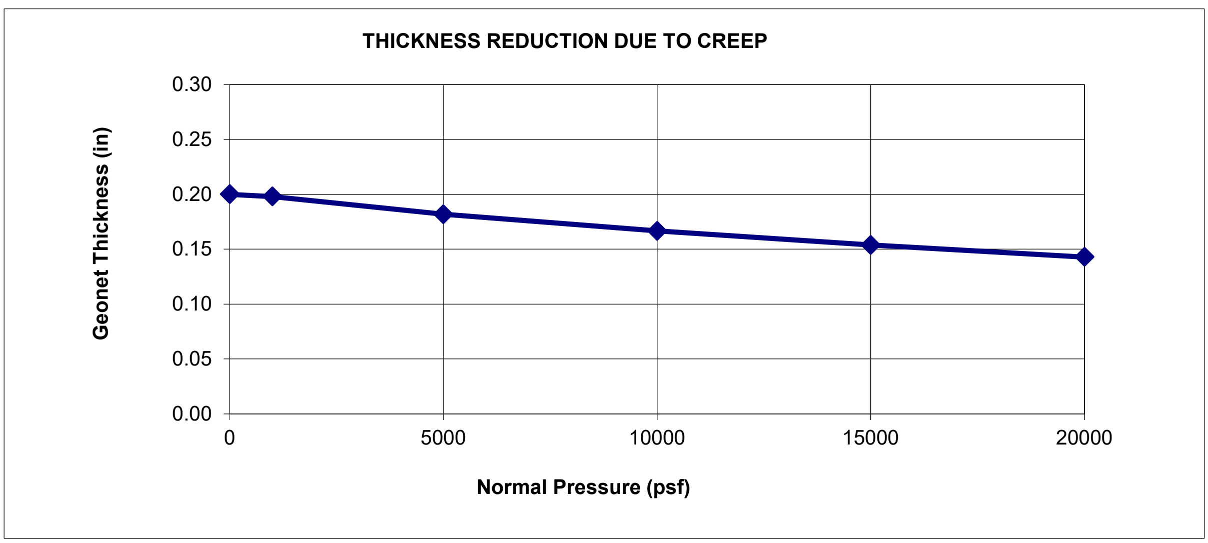


Table 2 - Reduction Factors and Factor of Safety (Sidewalls)

Reduction Factors[1]		Fill Condition: Liner Protective Cover Installed	Fill Condition: Maximum Waste Column in Place
RF_{IN}	Delayed Intrusion	1.0	1.2
RF_{CC}	Chemical Clogging	1.0	1.2
RF_{BC}	Biological Clogging	1.0	1.0
Total Reduction Factor[2]		1.00	1.44
Overall Factor of Safety to Account For Uncertainties		2.0	2.0
FS Factor[3]		2.00	2.88

[1] Values are obtained from References 3, 8, and 9.
[2] The Total Reduction Factors are a product of all the reduction factors for each fill condition.
[3] The FS Factor is a product of the Total Reduction Factor and Overall Factor of Safety to Account For Uncertainties for each fill condition.
[4] Chemical and biological clogging are assumed neglible due to short time underdrain utilized prior to ballasting. Some minor chemical clogging may occur over time due to groundwater mineralization.

Manufacturer's Transmissivity Data

The required minimum transmissivity for the 200-mil-thick double-sided geocomposite with a 6 oz/sy geotextile is shown in table below. These values are developed based on engineering judgment and experience with similar geocomposite products at numerous MSW sites evaluated by WCG in the US.

Compute the design transmissivity (T) of the geocomposite.

Table 3 - Design Transmissivity (Sidewalls)

Fill Condition	t[1] (in)	T[2] (m^2/s)	FS Factor[3]	T_{DES}[4] (m^2/s)	T_{DES} (cfs/ft)
Liner and Protective Cover Installed	0.199	2.50E-03	2.00	1.25E-03	1.35E-02
Maximum Waste Thickness as Ballast	0.192	2.30E-03	2.88	7.99E-04	8.60E-03

[1] t is the calculated geocomposite thickness from Table 1.

[2] T is the transmissivity values obtained from review of representative geocomposite products similar to proposed for project. Representative transmissivity values for 200-mil geocomposite shown on Sheet IIID-C-10.

[3] FS Factor is the product of the factors of safety from Table 2.

[4] T_{DES} is the design transmissivity value calculated using the following equation:

$$T_{DES} = T / (\text{FS Factor})$$

Design Flow Capacity

Unit Width of Geocomposite in dewatering: 1 ft

From Tables 3A and 3B above, the minimum design transmissivity of the geocomposite drainage layer is:

$Q_{design,\ sideslope}$ = 8.60E-03 cfs/ft

$Q_{max,\ sideslope}$ = 2.73E-05 cfs/ft

The flow capacity of the 200 mil geocomposite ($Q_{design,\ sideslope}$) is greater than the estimated flow of groundwater into the geocomposite ($Q_{max,\ sideslope}$). Therefore the design use of 200 mil for the sideslope installation is acceptable.

3. Estimate the flow into the dewatering pipe - Sidewall (Sectors 4 and 5)

$Q = kiA$

where:
- Q = inflow rate (cfs/ft)
- k = hydraulic conductivity (cm/s)
- i = gradient (ft/ft)
- A = largest area flowing to a dewatering pipe (sf)

k =	4.62E-03	cm/s	
	1.52E-04	ft/s	
i =	0.009	ft/ft	
A =	36,000	sf	(1,800 lineal ft slope length x 20 ft (Sector 4) waterbearing formation thickness)

$Q_{max,\ sideslope}$ = 4.91E-02 cfs

4. Determine the flow capacity of the dewatering pipe (Sector 4 and 5 Sidewall Toe Drain).

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

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 4-inch SDR 17 pipe:

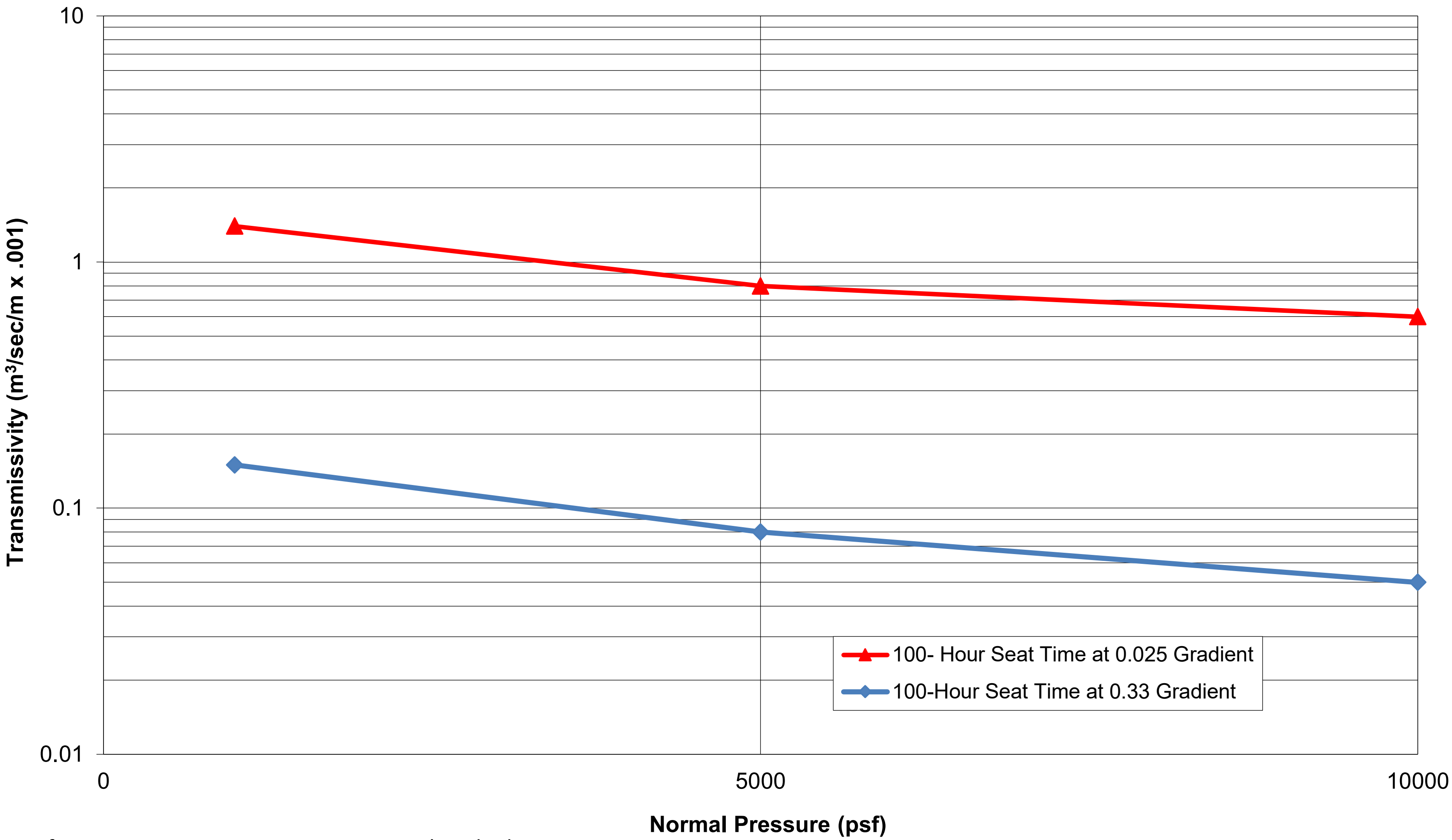
		ID =	3.97	in
		=	0.331	ft
A =	$\frac{(\pi \times d^2)}{4}$	A =	0.086	sq ft
R =	d / 4	R =	0.083	ft
S =	Design slope of pipe (0.5% min)	S =	0.005	ft / ft
n =	Manning's number	n =	0.009	from Ref. 6

Q_{full} = 0.19 cfs
= 86 gpm

$Q_{max,\ sideslope}$ = 0.05 cfs (from Step 3)
= 22.0 gpm

The flow capacity of the 4-inch-diameter pipe (86 gpm) is significantly larger than the maximum calculated flow from the geocomposite (22 gpm for 1,800 lineal foot of piping) into the toe dewatering pipe (or a calculated 1.2 gpm per 100 feet of pipe). Note also that these calculations do not account for the future continued dewatering of the alluvium, which will further reduce flow into the underdrain system.

TRANSMISSIVITY OF DOUBLE-SIDED GEOCOMPOSITE
6/8 oz/sy Polypropylene Geotextile with 200 mil Drainage Net
(Soil/Geocomposite/Geomembrane)



Reference: GSE Drainage Design Manual, 3rd Edition, 2004

UNDERDRAIN SUMP PUMP DEMONSTRATION

FORT WORTH C & D LANDFILL
APPENDIX IIID-C
UNDERDRAIN DEWATERING SYSTEM
UNDERDRAIN PUMP DEMONSTRATION

REQUIRED: Calculate pump size for inflow conditions at Sector 4. Pipe assumes marginal groundwater storage within the sump, and also assumes that smaller pumps may be used in event actual field conditions dictate lower flows.

METHOD: A. Use groundwater production rates from calculations and the sump drainage area for Sector 4 (with approximately 1,800 lineal feet of sideslope draining into the sump at a calculated rate of 1.2 gpm/100 lineal feet of sideslope (as presented on Figure IIID-C-2). Underdrain piping and sump details are provided on Figures IIID-C-4 and 5.

REFERENCES:

1. Bass, J., *Avoiding Failure of Leachate Collection and Cap Drainage Systems* , Pollution Technology Review No. 138, Noyles Data Corporation, 1986.

2. Phillips 66 Driscopipe, *System Design* , 1991.

3. Heisler, Sanford I., P.E., Wiley Engineer's Desk Reference, John Wiley & Sons, Inc., New York, 1998.

FORT WORTH C & D LANDFILL
APPENDIX IIID-C
UNDERDRAIN DEWATERING SYSTEM
UNDERDRAIN PUMP DEMONSTRATION

SOLUTION:

A. Average Groundwater Flow Rate into Sump

Determine the per lineal foot of collection pipe flow rate for a typical underdrain sump.

The following table presents an estimate of the flow into a sump based on the caculations presented in this appendix.

Calculations performed for Sector 4 as representative of both Sector 4 and 5, the remaining sectors requiring underdrains.

Condition	Length Constructed Underdrain Slope (lineal ft)	Underdrain Seepage (gpm/100 lineal ft slope)	Total Flow	
			gpm	gpd
Sector 4	1800	1.2	21.6	31,104

B. Estimated Rate of Underdrain Groundwater Removal

Submersible pump capacity = 30 gpm

Groundwater Production (gpd)	Pump Rate (gpm)	Average Pump Time	
		(min/day)	(hr/day)
31,104	30	1,037	17.3

Average pump time is less than 24 hours per day, therefore the design is acceptable. A pump with less capacity may also be used if it can be demonstrated (based on field records) that the actual underdrain groundwater flow rate is less than the design flow. Alternatively landfill operator may elect to periodically pump sumps using a submersible pump versus a dedicated pump.

PIPE STRUCTURAL STABILITY – 4 INCH TRENCH PIPE

REQUIRED: Analyze structural stability of the 4-inch-diameter groundwater dewatering system pipe.

METHOD:

A. Determine the critical load and calculate stress under the following two conditions:

1. Construction loading
2. Overburden loading

B. Use the critical loading pressure to analyze pipe stability under the following three possible failure conditions:

1. Wall crushing
2. Wall buckling
3. Ring deflection

NOTE: Typical groundwater underdrain dewatering system details are shown on Figures IIID-C-1 through IIID-C-6, and are for illustration purposes only. Additional Groundwater dewatering system details can be found in Appendix IIIA.

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. Phillips 66 Driscopipe, *System Design* , 1991.

4. Landfill Design Series, *Leachate Gas Management Systems Design, Volume 5, Leachate Management and Storage,* Appendix A, 1993.

5. Caterpillar Tractor Company*, Caterpillar Performance Handbook,* Edition 27, October 1996.

6. Quian, Xuede, R.M. Koerner, D. H. Gray, "Geotechnical Aspects of Landfill Design and Construction." Prentice-Hall, Inc., New Jersey, 2002.

SOLUTION:

A. Determine the critical load and stress:

A.1. Maximum construction loading:

Assume: CAT 637E Series II scraper with an even load distribution

Loaded weight = 190,500 lb
Tire pressure = 80 psi
Number of tires = 4

For a circular tire imprint:

$$F = \frac{\text{Loaded Weight}}{\text{Number of Tires}}$$

Where: F = Force exerted by one tire (lb)

F =	47,625	lb

Determine area of contact for circular tire imprint:

$$r = (F/\pi p)^{1/2}$$

Where:
r = Radius of contact (in)
F = Force exerted by one tire (lb)
p = Tire pressure (psi)

r =	13.8	in

Use Boussinesq's solution to find the stress at a point below a uniformly loaded circular area:

$$y = p\,(1 - ((r/z)^2+1)^{-3/2})$$

Where:
y = Change in vertical stress (psi)
p = Tire pressure (psi)
r = Radius of contact (in)
z = Protective cover thickness (in)

z = 24 in

y =	27.8	psi

Assume only one wheel load on pipe and add 50% for impact loading:

P_L = 1.5y

Where: P_L = Maximum live load (psi)

P_L =	41.7	psi

P_D = (zw)/1728

Where: P_D = Maximum dead load (psi)
z = Protective cover thickness (in)
w = Unit weight of protective cover (pcf)

z =	24	in
w =	120	pcf

P_D =	1.67	psi

P_T = $P_L + P_D$

Where: P_T = Maximum construction load (psi)

P_T =	43.3	psi

A.2. Overburden loading (postclosure load):

For maximum fill load on pipe:

1.0	ft protective cover @	120	pcf =	120	psf	
4.0	soil liner & prot. cover @	120	pcf =	480	psf	
100.0	ft C&D waste/soil @	80	pcf =	8,000	psf	Highest waste column thickness
			Σ =	8,600	psf	over a 4" LCS pipe.

P_T =	59.7	psi

Determine critical loading condition:

Construction loading:	P_T =	43.3	psi
Overburden loading:	P_T =	59.7	psi

Overburden loading is most critical to the structural stability of the pipe and will be used to determine the design pipe stress.

Determine design stress:

1. Adjust critical stress to account for loss of strength in the pipe due to perforations:

$P_{DES1} = 12P_T / (12 - l_p)$

Where:

l_p	= Cumulative length of perforations per foot of pipe
P_T	= Critical pipe stress (psi)
P_{DES1}	= Pipe stress adjusted for loss of strength (psi)

	6	holes / foot
	0.5	in / hole
l_p =	3.0	in/ft

From determination of critical loading:

P_T =	59.7	psi
P_{DES1} =	79.6	psi

Adjust pipe stress determined above to account for effects of soil arching:

2. The design pipe stress is estimated by accounting for the soil structure interaction between the buried groundwater dewatering system pipe and its backfill to obtain a realistic loading condition on the pipe.

2a. For the burial conditions shown on Figure 1 (sheet IIID-C-26), the pipe may be classified as a positive projecting conduit.

2b. Because the pipe is flexible and will deflect in the vertical plane as shown on Figure 2 (sheet IIID-C-27), the pipe will experience a reduction in loading due to soil arching. Soil arching is present when the soil column over the pipe settles and creates shear stresses in the surrounding soil. Those shear stresses will support the soil column, thereby reducing the load experienced by the pipe (see Figure 3, sheet IIID-C-27).

2c. The load on the pipe will be estimated using Marston's Formula:

$$W_c = \gamma C_c B_c^{\,2} \qquad (1)$$

$$C_c = \frac{e^{\pm 2k\mu\left(H_e/B_c\right)} - 1}{\pm 2k\mu} + \left(\frac{H}{B_c} - \frac{H_e}{B_c}\right) e^{\pm 2k\mu\left(H_e/B_c\right)} \qquad (2)$$

Where:	W_c	= Load per unit length of conduit (lb/ft)
	γ	= Unit weight of soil above conduit (pcf)
	B_c	= Outer diameter of conduit (ft)
	H	= Height of fill above conduit (ft)
	H_e	= Height of plane of equal settlement above critical plane (ft)
	k	= Lateral pressure ratio (earth pressure coefficient)
	μ	= tan ϕ
	ϕ	= Angle of internal friction of pipe-zone backfill (PZB) (degrees)

$$H_e = \pm r_{sd}\, p\left(\frac{H}{B_c}\right) \qquad (3)$$

Where:	r_{sd}	= Settlement ratio
	p	= Ratio of the conduit projection above the compacted soil liner to its diameter

$$r_{sd} = \frac{\left(S_m + S_g\right) - \left(S_f + dc\right)}{S_m} \qquad (4)$$

Where:	S_m	= Compression deformation of soil column adjacent to conduit
	S_g	= Settlement of natural ground adjacent to conduit
	S_f	= Settlement of conduit into foundation material
	dc	= Vertical deflection of the conduit

It is assumed that for a groundwater dewatering system pipe S_g and S_f are equivalent. The equation settlement ratio, therefore, reduces to the following:

$$r_{sd} = \frac{S_m - dc}{S_m} \qquad (5)$$

Since the trench aggregate (PZB) is much stiffer than the pipe, dc is larger than S_m implying that r_{sd} will be negative. Because r_{sd} is negative, the pipe is categorized as an incomplete ditch as specified by Marston. Note that in the above equations, where a + and a - sign are used together, the upper sign corresponds to a positive r_{sd} and a the lower sign to a negative r_{sd}.

2d. Load analysis solution by trial and error

Step 1: Assume a value for the settlement ratio, r_{sd}.

r_{sd} = -0.58

Step 2: Calculate S_m based on the estimated vertical stress at the level of the pipe and the deformation modulus E of the PZB.

$S_m = P_{DES1} D / E_s$

Where:
- P_{DES1} = Pipe stress adjusted for loss of strength (psi)
- D = Pipe diameter (in)
- E_s = PZB soil modulus (psi)

P_{DES1} =	79.6	psi
D =	4.5	in
E_s =	3,000	psi

S_m =	0.119	in

Step 3: Calculate dc using Equation (5):

$dc = S_m (1 - r_{sd})$

dc =	0.188	in

Step 4: Use the Iowa Formula (provided below) to calculate load per unit length (W_c).

$$W_c = \frac{dc}{(DL)k}\left(\frac{EI}{r^3} + 0.061E'\right)$$

Where:
- DL = Deflection lag factor
- k = Bedding factor
- E = Young's modulus for pipe material (psi)
- I = Moment of inertia for pipe wall = $t^3/12$ (in^4/in)
- r = Pipe radius (in)
- E' = Modulus of soil reaction (psi)

DL =	2.5	(Ref 6)
k =	0.1	(Ref 6)
E =	33,000	psi (refer to chart 25 on page IIID-C-58, based on P_{DES1} above)
t =	0.390	in (SDR 17 pipe)
I =	0.005	in^4/in
r =	2.3	in
E' =	3,000	psi

W_c =	149	lb/in

Step 5: Calculate C_c using Equation 1:

$$C_c = \frac{W_c}{\gamma B_c^{\ 2}}$$

Composite unit weight for waste and soil:

5.0	ft soil @	120	pcf =	600	psf
100.0	ft waste @	80	pcf =	8,000	psf
			Total =	8,600	psf

γ =	81.90	pcf (weighted average based on above table)
B_c =	4.5	in

C_c =	154.9	(unitless)

Step 6: Solve for H_e/B_c using Equation 2 in an iterative manner:

H =	100	ft
H/B_c =	266.7	

Assume: H_e/B_c = 2.06

kμ =	0.13	(Ref 4)
$e^{-2k\mu(He/Bc)}-1$ =	-0.42	
-2kμ =	-0.26	
$(H/B_c - H_e/B_c)$ =	264.6	
$e^{-2k\mu(He/Bc)}$ =	0.58	

Left-hand-side of equation (LHS) = 155
Right-hand-side of equation (RHS) = 156

Step 7: Substitute H_e/B_c into equation given below to determine if proper value for r_{sd} was used.

$$\left[\frac{1}{2k\mu} \pm \left(\frac{H}{B_c} - \frac{H_e}{B_c}\right) \pm \frac{r_{sd}\,p}{3}\right]\frac{e^{\pm 2k\mu(H_e/B_c)} - 1}{\pm 2k\mu} \pm \frac{1}{2}\left(\frac{H_e}{B_c}\right)^2$$
$$\pm \frac{r_{sd}\,p}{3}\left(\frac{H}{B_c} - \frac{H_e}{B_c}\right)e^{\pm 2k\mu(H_e/B_c)} - \frac{1}{2k\mu}\left(\frac{H_e}{B_c}\right) \mp \left(\frac{H}{B_c}\right)\left(\frac{H_e}{B_c}\right) = \pm r_{sd}\,p\left(\frac{H}{B_c}\right)$$

Because r_{sd} is negative for the incomplete ditch condition, the lower signs in the above equation are used.

p =	1
kμ =	0.13
H/B_c =	266.7
H_e/B_c =	2.06
r_{sd} =	-0.58
LHS =	154
RHS =	154

If LHS is not approximately equal to RHS, adjust value for r_{sd} in Step 1 and repeat solution procedure.

2e. Once the solutions to the above equations are determined, the design pipe stress may be calculated and the deflection of the pipe determined.

$P_{DES2} = W_c / D$

Where: P_{DES2} = Load on pipe adjusted to account for effects of soil arching (psi)

W_c =	149	lb/in
D =	4.5	in
P_{DES2} =	33	psi

A summary table for the structural stability analysis is provided on sheet IIID-C-25 for the 4-inch-diameter groundwater dewatering system pipe. A pipe will be selected from this table for use in the groundwater dewatering system based on the calculated factors of safety for each possible failure condition. An example calculation is provided below that outlines the procedures used to determine the factors of safety for all pipe SDR sizes shown in the summary table.

B. Use the critical loading pressure to analyze pipe stability:

Example pipe structural stability calculations:

SDR	= Standard dimension ratio	=	17	
S_Y	= compressive yield strength	=	1,500	psi
RD_{all}	= allowable ring deflection	=	4.2	%

1. Wall crushing (Ref 3)

$S_A = P_{DES2} (SDR - 1) / 2$ FS $= S_Y / S_A$

Where:
- S_A = Actual compressive stress (psi)
- SDR = Standard dimension ratio
- P_{DES2} = Load pipe adjusted to account for effects of soil arching (psi)
- S_Y = Compressive yield strength (psi)
- FS = Factor of safety against wall crushing

P_{DES2} = 33 psi

S_A =	264.3	psi
FS =	5.7	

Compare calculated and suggested factor of safety:	5.7	> 1.0

2. Wall buckling (Ref 3)

$P_{cb} = 0.8 (E' (2.32E / SDR^3))^{1/2}$ FS $= P_{cb} / P_{DES2}$

Where:
- P_{cb} = Critical buckling pressure at top of pipe (psi)
- E' = Soil modulus (psi)
- E = Stress/time dependent tensile modulus for design loading conditions (psi)
- P_{DES2} = Load pipe adjusted to account for effects of soil arching (psi)
- FS = Factor of safety against wall buckling

E' =	3,000	psi
E =	27,000	psi for 50 years based on S_A above (see chart page IIID-C-28)
P_{DES2} =	33	psi

P_{cb} =	156.5	psi
FS =	4.7	

Compare calculated and suggested factor of safety:	4.7	> 1.0

3. Ring deflection (Ref 3)

$E_S = P_{DES2} / E'$

Where:
E_S = Soil strain (%)
P_{DES2} = Load pipe adjusted to account for effects of soil arching (psi)
E' = Soil modulus (psi)

P_{DES2} = 33 psi
E' = 3,000 psi

E_S = 1.1 %

Ring deflection for buried HDPE pipe is conservatively the same (no more than) the vertical compression of the soil envelope around the pipe. Therefore, assumed actual ring deflection (RD_{act}) is equal to soil strain.

RD_{act} = 1.1 %

Allowable ring deflection, RD_{all} = 4.20 %

$RD_{act} < RD_{all}$, design is acceptable

FORT WORTH C & D LANDFILL
APPENDIX IIID-C
UNDERDRAIN DEWATERING SYSTEM
PIPE STRUCTURAL STABILITY - 4" DIA PIPE

Adjusted load to account for soil arching = 33 psi

	Wall Crushing			Wall Buckling				Ring Deflection			
SDR	S_Y	S_A	FS_{WC}	E^2	E'	P_{cb}	FS_{WB}	RD_{all}	E'	RD_{act}	FS_{RD}
32.5	1,500	520.3	2.9	20,000	3,000	50.9	1.5	8.1	3,000	1.1	7.4
26.0	1,500	412.9	3.6	22,000	3,000	74.7	2.3	6.5	3,000	1.1	5.9
21.0	1,500	330.3	4.5	25,000	3,000	109.7	3.3	5.2	3,000	1.1	4.7
19.0	1,500	297.3	5.0	26,000	3,000	129.9	3.9	4.7	3,000	1.1	4.3
17.0 [1]	1,500	264.3	5.7	27,000	3,000	156.5	4.7	4.2	3,000	1.1	3.8
15.5	1,500	239.5	6.3	28,000	3,000	183.0	5.5	3.9	3,000	1.1	3.5
13.5	1,500	206.6	7.3	29,000	3,000	228.9	6.9	3.4	3,000	1.1	3.1
11.0	1,500	165.2	9.1	30,000	3,000	316.9	9.6	2.7	3,000	1.1	2.5

denotes standard size

[1] Select 4-inch-diameter HDPE SDR 17.0 pipe for use in the groundwater dewatering system based on the calculated factors of safety.

[2] Values for the modulus of elasticity were selected from the attached chart (sheet IIID-C-29), Reference 3, using the calculated stress in the pipe wall (S_A under the wall crushing heading in the above table) for a 50 year duration (maximum loading is the overburden load on the pipe).

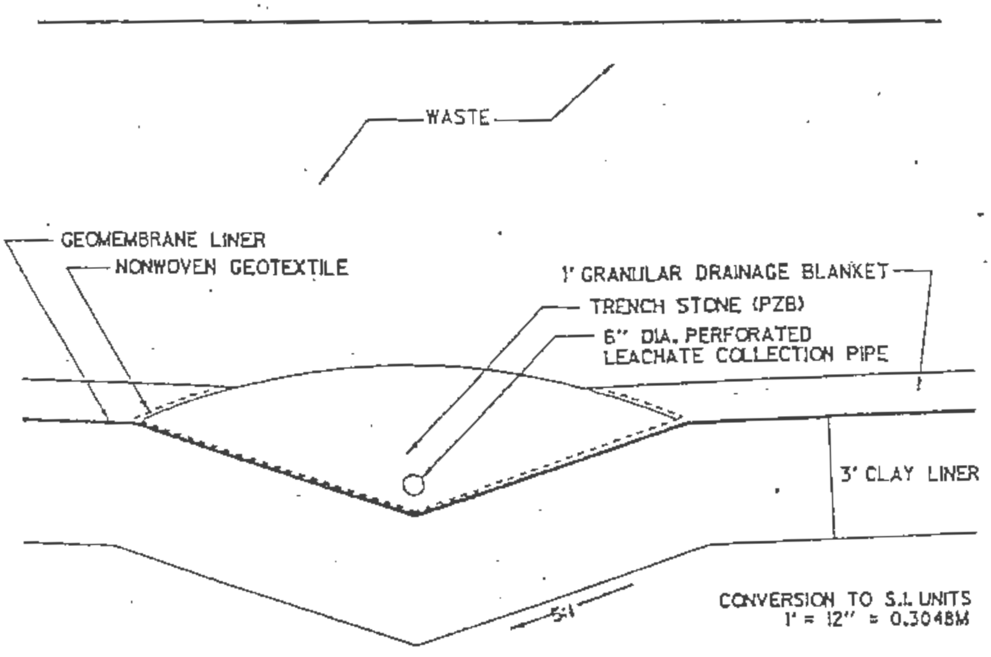


FIGURE 1: TYPICAL V-SHAPED TRENCH
FOR LANDFILLS WITH COMPOSITE LINERS

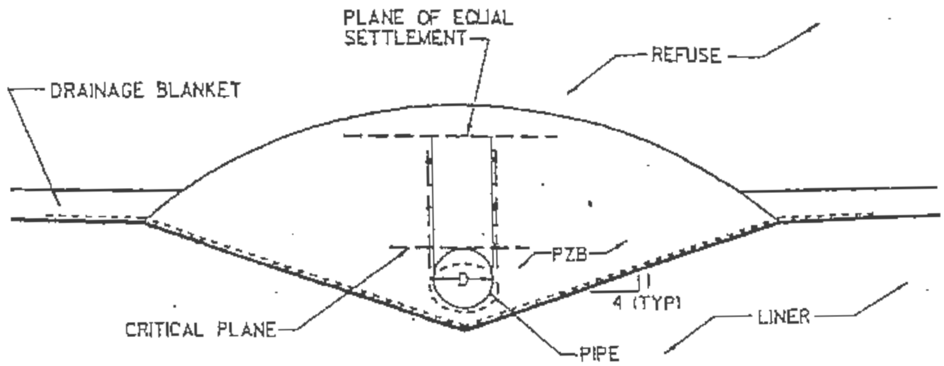


FIGURE 2: SETTLEMENT OF LEACHATE PIPE INDUCING SHEAR STRESSES IN PZB

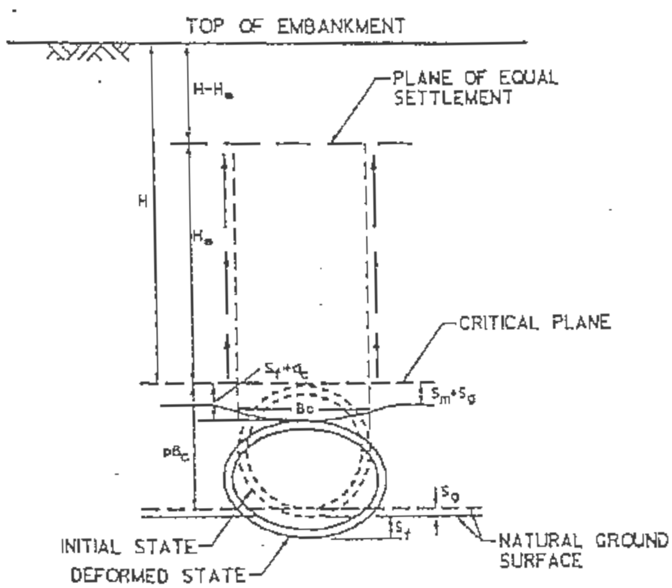


FIGURE 3: CASE OF AN INCOMPLETE DITCH CONDITION FOR A POSITIVE PROJECTING CONDUIT

/dl/users/jow/prgetran.dgn

here: S_A = Actual compressive stress, psi
SDR = Standard Dimension Ratio
P_T = External Pressure, psi

Safety Factor = 1500 psi ÷ S_A where 1500 psi is the Compressive Yield Strength of Driscopipe.

Design by Wall Buckling: Local wall buckling is a longitudinal wrinkling of the pipe wall. Tests of *non-pressurized* Driscopipe show that buckling and collapse do not occur when the soil envelope is in full contact with the pipe and is compacted to a dense state. However, it can be forced to occur over the long term in *non-pressurized* pipe if the total external soil pressure, P_t, is allowed to exceed the pipe-soil system's critical buckling pressure, P_{cb}. If $P_t > P_{cb}$, gradual collapse may occur over the long term. A calculated, conservative value for the critical buckling pressure may be obtained by the following approximate formula. All pipe diameters with the same SDR in the same burial situation have the same critical collapse and critical buckling endurance

$$P_{cb} = 0.8\sqrt{E' \times P_c}$$

Where:

P_t = Total vertical soil pressure at the top of the pipe, psi

P_{cb} = Critical buckling soil pressure at the top of the pipe, psi

E' = Soil modulus in psi calculated as the ratio of the vertical soil pressure to vertical soil strain at a specified density

P_c = Hydrostatic, critical-collapse differential pressure, psi

$$P_c = \frac{2E\,(t/D)^3\,(D_{MIN}/D_{MAX})^3}{(1-\mu^2)}$$

$$P_c = \frac{2.32\,E}{(SDR)^3}$$

Where: (D_{MIN}/D_{MAX}) = .95
μ = Poission's Ratio
μ = .45 for Driscopipe
E = stress and time dependent tensile modulus of elasticity, psi

In a direct burial pressurized pipeline, the internal pressure is usually great enough to exceed the external critical-buckling soil pressure. When a pressurized line is to be shut down for a period, wall buckling should be examined.

Design by Wall Buckling Guidelines:
Although wall buckling is seldom the limiting factor in the design of a Driscopipe system, a check of non-pressurized pipelines can be made according to the following steps to insure $P_t < P_{cb}$.

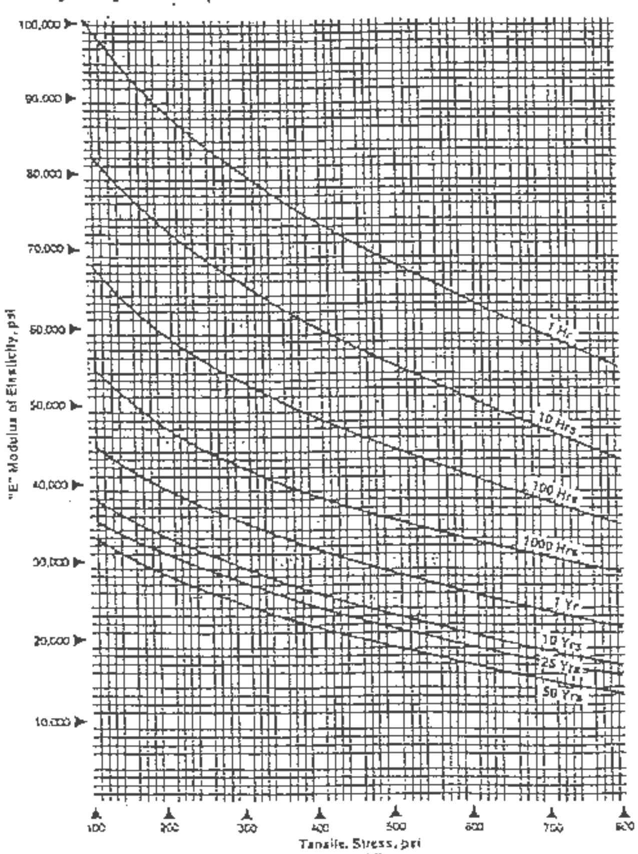
1. Calculate or estimate the total soil pressure, P_t, at the top of the pipe.
2. Calculate the stress "S_A" in the pipe wall according to the formula:

$$S_A = \frac{(SDR - 1)\,P_t}{2}$$

3. Based upon the stress "S_A" and the estimated time duration of non-pressurization, use Chart 25 to find the value of the pipe's modulus of elasticity, E, in psi.

Chart 25

Time Dependent Modulus of Elasticity for Polyethylene Pipe vs. Stress Intensity (73.4°F)



NOTE: The short term modulus of elasticity of Driscopipe per ASTM D 638 is approximately 100,000 psi. Due to the cold flow (creep) characteristic of the pipe material, this modulus is dependent upon the stress intensity and the time duration of the applied stress.

DRISCOPIPE

Simplified Burial Design: A conservative estimate of the ability of Driscopipe pipelines to perform in a buried environment is found in Chart 24. It is based on a minimum 2:1 safety factor and 50 year design service life. A detailed burial design starts on page 37. The detailed design should be used for critical or marginal applications or whenever a more precise solution is desired.

Detailed Burial Design:
Design by Wall Crushing: Wall crushing would theoretically occur when the stress in a pipe wall, due to the external vertical pressure, exceeded the long-term compressive strength of the pipe material. To ensure that the Driscopipe wall is strong enough to endure the external pressure the following check should be made:

$$S_A = \frac{(SDR-1)}{2}P_T$$

Values of E′

Based on Soil Type (ASTM D2321) and Degree of Compaction

Soil Type of Initial Backfill Embedment Material	Description	E′ (psi) for Degree of Compaction (Proctor Density, %) Loose	Slight (70-85%)	Moderate (85-95%)	High (95%)
I	Manufactured angular, granular materials (crushed stone or rock, broken coral, cinders, etc.)	1,000	3,000	3,000	3,000
II	Coarse grained soils with little or no fines	N.R.	1,000	2,000	3,000
III	Coarse grained soils with fines	N.R.	N.R.	1,000	2,000
IV	Fine-grained soils	N.R.	N.R.	N.R.	N.R.
V	Organic soils (peat, muck, clay, etc.)	N.R.	N.R.	N.R.	N.R.

N.R. = Not Recommended for use by ASTM D2321 for pipe wall support

Chart 24

SDR	Maximum Burial Depth, ft. in dry soil of 100 lbs/cu. ft. Soil Modulus, psi* 1000	2000	3000	Maximum External Pressure psi Soil Modulus, psi* 1000	2000	3000	Maximum Deflection, % after installation Soil Modulus, psi* 1000	2000	3000
32.5	25	32	37	17	22	26	1.7	0.9	0.6
26	33	45	52	23	31	36	2.3	1.2	0.8
21	46	61	71	32	42	49	3.2	1.6	1.1
19	52	69	81	36	48	56	3.6	1.8	1.2
17	61	121	181	42	84	126	4.2	2.1	1.4
15.5	56	112	168	39	78	117	3.9	2.0	1.3
13.5	49	98	147	34	68	102	3.4	1.7	1.1
11	39	78	117	27	54	81	2.7	1.4	0.9
9.3	33	68	101	23	47	70	2.3	1.2	0.8
8.3	30	61	89	21	42	62	2.1	1.1	0.7
7.3	26	52	79	18	36	55	1.8	0.9	0.6

*assumes no external loads

PIPE STRUCTURAL STABILITY – 18 INCH SUMP RISER PIPE

FORT WORTH C & D LANDFILL
APPENDIX IIID-C
UNDERDRAIN DEWATERING SYSTEM
PIPE STRUCTURAL STABILITY - 18" DIA PIPE

REQUIRED: Analyze structural stability of the 18-inch-diameter groundwater dewatering system pipe.

METHOD:

A. Determine the critical load and calculate stress under the following two conditions:

1. Construction loading
2. Overburden loading

B. Use the critical loading pressure to analyze pipe stability under the following three possible failure conditions:

1. Wall crushing
2. Wall buckling
3. Ring deflection

NOTES: The groundwater dewatering system details shown on Figures IIID-C-1 through IIID-C-6 are for illustration purposes only to show parameters used in the following calculations. Groundwater underdrain dewatering system details can also be found in Appendix III. The calculations assume 100-feet of waste placement over pipe, which is conservative considering the underdrain system will be ballasted and the underdrain system abandoned prior to placement of 100 feet of waste over system.

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. Phillips 66 Driscopipe, *System Design* , 1991.

4. Landfill Design Series,*Leachate Gas Management Systems Design, Volume 5, Leachate Management and Storage,* Appendix A, 1993.

5. Caterpillar Tractor Company, *Caterpillar Performance Handbook,* Edition 27, October 1996.

6. Quian, Xuede, R.M. Koerner, D. H. Gray, "Geotechnical Aspects of Landfill Design and Construction." Prentice-Hall, Inc., New Jersey, 2002.

SOLUTION:

A. Determine the critical load and stress:

A.1. Maximum construction loading

Assume: CAT 637E Series II scraper with an even load distribution

Loaded weight =	190,500	lb
Tire pressure =	80	psi
Number of tires =	4	

For a circular tire imprint:

$$F = \frac{\text{Loaded Weight}}{\text{Number of Tires}}$$

Where: F = Force exerted by one tire (lb)

F =	47,625	lb

Determine area of contact for circular tire imprint:

$$r = (F/\pi p)^{1/2}$$

Where:
r = Radius of contact (in)
F = Force exerted by one tire (lb)
p = Tire pressure (psi)

r =	13.8	in

Use Boussinesq's solution to find the stress at a point below a uniformly loaded circular area:

$$y = p\,(1 - ((r/z)^2+1)^{-3/2})$$

Where:
y = Change in vertical stress (psi)
p = Tire pressure (psi)
r = Radius of contact (in)
z = Protective cover thickness (in)

z = 24 in

y =	27.8	psi

Assume only one wheel load on pipe and add 50% for impact loading:

$P_L = 1.5y$

Where: P_L = Maximum live load (psi)

P_L =	41.7	psi

$P_D = (zw)/1728$

Where:
P_D = Maximum dead load (psi)
z = Protective cover thickness (in)
w = Unit weight of protective cover (pcf)

z =	24	in
w =	120	pcf

P_D =	1.67	psi

$P_T = P_L + P_D$

Where: P_T = Maximum construction load (psi)

P_T =	43.3	psi

A.2. Overburden loading (postclosure load):

For maximum fill load on pipe:

1.0	ft intermediate cover @	120	pcf =	120	psf
4.0	soil liner & prot. cover @	120	pcf =	480	psf
100	ft C&D waste/soil @	80	pcf =	8,000	psf
			Σ =	8,600	psf

P_T =	59.7	psi

Determine critical loading condition:

Construction loading:	P_T =	43.3	psi
Overburden loading:	P_T =	59.7	psi

Overburden loading is most critical to the structural stability of the pipe and will be used to determine the design pipe stress.

Determine Design Stress:

1. Adjust critical stress to account for loss of strength in the pipe due to perforations:

$$P_{DES1} = 12P_T / (12 - l_p)$$

Where:

l_p = Cumulative length of perforations per foot of pipe
P_T = Critical pipe stress (psi)
P_{DES1} = Pipe stress adjusted for loss of strength (psi)

6	holes / foot
0.5	in / hole

l_p =	3.0	in/ft

From determination of critical loading:

P_T =	59.7	psi

P_{DES1} =	79.6	psi

Adjust pipe stress determined above to account for effects of soil arching:

2. The design pipe stress is estimated by accounting for the soil structure interaction between the groundwater dewatering system pipe and its backfill to obtain a realistic loading condition on the pipe.

2a. For the burial conditions shown on Figure 1 (sheet IIID-C-26), the pipe may be classified as a positive projecting conduit.

2b. Because the pipe is flexible and will deflect in the vertical plane as shown on Figure 2 (sheet IIID-C-27), the pipe will experience a reduction in loading due to soil arching. Soil arching is present when the soil column over the pipe settles and creates shear stresses in the surrounding soil. Those shear stresses will support the soil column, thereby reducing the load experienced by the pipe (see Figure 3, sheet IIID-C-27).

2c. The load on the pipe will be estimated using Marston's Formula:

$$W_c = \gamma C_c B_c^{\,2} \qquad (1)$$

$$C_c = \frac{e^{\pm 2k\mu(H_e/B_c)} - 1}{\pm 2k\mu} + \left(\frac{H}{B_c} - \frac{H_e}{B_c}\right) e^{\pm 2k\mu(H_e/B_c)} \qquad (2)$$

Where:
- W_c = Load per unit length of conduit (lb/ft)
- γ = Unit weight of soil above conduit (pcf)
- B_c = Outer diameter of conduit (ft)
- H = Height of fill above conduit (ft)
- H_e = Height of plane of equal settlement above critical plane (ft)
- k = Lateral pressure ratio (earth pressure coefficient)
- μ = tan ϕ
- ϕ = Angle of internal friction of pipe-zone backfill (PZB) (degrees)

$$H_e = \pm r_{sd}\, p\left(\frac{H}{B_c}\right) \qquad (3)$$

Where:
- r_{sd} = Settlement ratio
- p = Ratio of the conduit projection above the compacted soil liner to its diameter

$$r_{sd} = \frac{\left(S_m + S_g\right) - \left(S_f + dc\right)}{S_m} \qquad (4)$$

Where:
- S_m = Compression deformation of soil column adjacent to conduit
- S_g = Settlement of natural ground adjacent to conduit
- S_f = Settlement of conduit into foundation material
- dc = Vertical deflection of the conduit

It is assumed that for a groundwater dewatering system pipe S_g and S_f are equivalent. The equation settlement ratio, therefore, reduces to the following:

$$r_{sd} = \frac{S_m - dc}{S_m} \qquad (5)$$

Since the trench aggregate (PZB) is much stiffer than the pipe, dc is larger than S_m implying that r_{sd} will be negative. Because r_{sd} is negative, the pipe is categorized as an incomplete ditch as specified by Marston. Note that in the above equations, where a + and a - sign are used together, the upper sign corresponds to a positive r_{sd} and a the lower sign to a negative r_{sd}.

2d. Load analysis solution by trial and error

Step 1: Assume a value for the settlement ratio, r_{sd}.

r_{sd} = -0.68

Step 2: Calculate S_m based on the estimated vertical stress at the level of the pipe and the deformation modulus E of the PZB.

$S_m = P_{DES1}\, D / E_s$

Where:
- P_{DES1} = Pipe stress adjusted for loss of strength (psi)
- D = Pipe diameter (in)
- E_s = PZB soil modulus (psi)

P_{DES1} =	79.6	psi
D =	18	in
E_s =	3,000	psi

S_m =	0.478	in

Step 3: Calculate dc using Equation (5):

$dc = S_m\,(1 - r_{sd})$

dc =	0.802	in

Step 4: Use the Iowa Formula (provided below) to calculate load per unit length (W_c).

$$W_c = \frac{dc}{(DL)k}\left(\frac{EI}{r^3} + 0.061E'\right)$$

Where:
- DL = Deflection lag factor
- k = Bedding factor
- E = Young's modulus for pipe material (psi)
- I = Moment of inertia for pipe wall = $t^3/12$ (in^4/in)
- r = Pipe radius (in)
- E' = Modulus of soil reaction (psi)

DL =	2.5	(Ref 6)
k =	0.1	(Ref 6)
E =	33,000	psi (refer to chart 25 on page IIID-C-58, based on P_{DES1} above)
t =	1.059	in (SDR 17 pipe)
I =	0.099	in^4/in
r =	9.0	in
E' =	3,000	psi

W_c =	602	lb/in

Step 5: Calculate C_c using Equation 1:

$$C_c = \frac{W_c}{\gamma B_c^{\ 2}}$$

Composite unit weight for waste and soil:

5.0	ft soil @	120	pcf =	600	psf
100.0	ft waste/soil @	80	pcf =	8,000	psf
			Total =	8,600	psf

γ =	81.9	pcf (weighted average based on above table)
B_c =	18	in

C_c =	39.2	(unitless)

Step 6: Solve for H_e/B_c using Equation 2 in an iterative manner:

H =	105	ft
H/B_c =	70.0	

Assume: H_e/B_c = 2.28

$k\mu$ =	0.13	(Ref 4)
$e^{-2k\mu(He/Bc)}-1$ =	-0.45	
$-2k\mu$ =	-0.26	
$(H/B_c - H_e/B_c)$ =	67.7	
$e^{-2k\mu(He/Bc)}$ =	0.55	

Left-hand-side of equation (LHS) = 39
Right-hand-side of equation (RHS) = 39

<u>Step 7:</u> Substitute H_e/B_c into equation given below to determine if proper value for r_{sd} was used.

$$\left[\frac{1}{2k\mu} \pm \left(\frac{H}{B_c} - \frac{H_e}{B_c}\right) \pm \frac{r_{sd}\,p}{3}\right] \frac{e^{\pm 2k\mu\left(H_e/B_c\right)} - 1}{\pm 2k\mu} \pm \frac{1}{2}\left(\frac{H_e}{B_c}\right)^2$$
$$\pm \frac{r_{sd}\,p}{3}\left(\frac{H}{B_c} - \frac{H_e}{B_c}\right) e^{\pm 2k\mu\left(H_e/B_c\right)} - \frac{1}{2k\mu}\left(\frac{H_e}{B_c}\right) \mp \left(\frac{H}{B_c}\right)\left(\frac{H_e}{B_c}\right) = \pm r_{sd}\,p\left(\frac{H}{B_c}\right)$$

Because r_{sd} is negative for the incomplete ditch condition, the lower signs in the above equation are used.

p =	1
$k\mu$ =	0.13
H/B_c =	70.0
H_e/B_c =	2.28
r_{sd} =	-0.68
LHS =	47
RHS =	48

If LHS is not approximately equal to RHS, adjust value for r_{sd} in Step 1 and repeat solution procedure.

2e. Once the solutions to the above equations are determined, the design pipe stress may be calculated and the deflection of the pipe determined.

$P_{DES2} = W_c / D$

Where: P_{DES2} = Load on pipe adjusted to account for effects of soil arching (psi)

W_c =	602	lb/in
D =	18.0	in
P_{DES2} =	33	psi

A summary table for the structural stability analysis is provided on sheet IIID-C-41 for the 18-inch-diameter groundwater dewatering system pipe. A pipe will be selected from this table for use in the groundwater dewatering system based on the calculated factors of safety for each possible failure condition. An example calculation is provided below that outlines the procedures used to determine the factors of safety for all pipe SDR sizes shown in the summary table.

B. Use the critical loading pressure to analyze pipe stability:

Example pipe structural stability calculations:

SDR	= Standard dimension ratio	=	17	
S_Y	= compressive yield strength	=	1,500	psi
RD_{all}	= allowable ring deflection	=	4.2	%

1. Wall crushing (Ref 3)

$S_A = P_{DES2}\,(SDR - 1) / 2$ FS $= S_Y / S_A$

Where:
- S_A = Actual compressive stress (psi)
- SDR = Standard dimension ratio
- P_{DES2} = Load pipe adjusted to account for effects of soil arching (psi)
- S_Y = Compressive yield strength (psi)
- FS = Factor of safety against wall crushing

P_{DES2} = 33 psi

S_A =	267.4	psi
FS =	5.6	

Compare calculated and suggested factor of safety:	5.6	> 1.0

2. Wall buckling (Ref 3)

$P_{cb} = 0.8\,(E'\,(2.32E / SDR^3))^{1/2}$ FS $= P_{cb} / P_{DES2}$

Where:
- P_{cb} = Critical buckling pressure at top of pipe (psi)
- E' = Soil modulus (psi)
- E = Stress/time dependent tensile modulus for design loading conditions (psi)
- P_{DES2} = Load pipe adjusted to account for effects of soil arching (psi)
- FS = Factor of safety against wall buckling

E' =	3,000	psi
E =	26,000	psi for 50 years based on S_A above (see chart sheet IIID-C-28)
P_{DES2} =	33	psi

P_{cb} =	153.5	psi
FS =	4.6	

Compare calculated and suggested factor of safety:	4.6	> 1.0

3. Ring deflection (Ref 3)

$E_S = P_{DES2} / E'$

Where:
E_S = Soil strain (%)
P_{DES2} = Load pipe adjusted to account for effects of soil arching (psi)
E' = Soil modulus (psi)

P_{DES2} =	33	psi
E' =	3,000	psi
E_S =	1.1	%

Ring deflection for buried HDPE pipe is conservatively the same (no more than) the vertical compression of the soil envelope around the pipe. Therefore, assumed actual ring deflection (RDact) is equal to soil strain.

RD_{act} = 1.1 %

Allowable ring deflection, RD_{all} = 4.20 %

$RD_{act} < RD_{all}$, design is acceptable

Adjusted load to account for soil arching = 33 psi

	Wall Crushing			Wall Buckling				Ring Deflection			
SDR	S_Y	S_A	FS_{WC}	E^2	E'	P_{cb}	FS_{WB}	RD_{all}	E'	RD_{act}	FS_{RD}
32.5	1,500	526.4	2.8	20,000	3,000	50.9	1.5	8.1	3,000	1.1	7.3
26.0	1,500	417.8	3.6	22,000	3,000	74.7	2.2	6.5	3,000	1.1	5.8
21.0	1,500	334.2	4.5	24,000	3,000	107.4	3.2	5.2	3,000	1.1	4.7
19.0	1,500	300.8	5.0	25,000	3,000	127.4	3.8	4.7	3,000	1.1	4.2
17.0 [1]	1,500	267.4	5.6	26,000	3,000	153.5	4.6	4.2	3,000	1.1	3.8
15.5	1,500	242.3	6.2	27,000	3,000	179.7	5.4	3.9	3,000	1.1	3.5
13.5	1,500	209.0	7.2	28,500	3,000	226.9	6.8	3.4	3,000	1.1	3.1
11.0	1,500	167.1	9.0	30,000	3,000	316.9	9.5	2.7	3,000	1.1	2.4

denotes standard size

[1] Select 18-inch-diameter HDPE SDR 17.0 pipe for use in the groundwater dewatering system based on the calculated factors of safety.

[2] Values for the modulus of elasticity were selected from the attached chart (sheet IIID-C-29), Reference 3, using the calculated stress in the pipe wall (S_A under the wall crushing heading in the above table) for a 50 year duration (maximum loading is the overburden load on the pipe).

VENEER STABILITY OF RECOMPACTED CLAY LINER AND GEOCOMPOSITE

FORT WORTH C & D LANDFILL
APPENDIX IIID-C
0771-356-11-35
VENEER STABILITY OF RECOMPACTED CLAY LINER
AND GEOCOMPOSITE UNDERDRAIN

Required: Evaluate the stability of the sidewall recompacted clay liner system components

Procedure:

A. Sidewall Bottom Liner System Stability

1. Verify that the tensile stress in the liner system will be less than the yield stress of the liner components by using Koerner's method for determination of shear stress in liner systems considering cohesion/adhesion forces of the liner components. Underdrain geocomposite designed to be installed on 3H:1V sidewalls without anchor trenches, based on results of

B. Infinite Slope Stability Analysis

1. Use Duncan and Buchignani's method for infinite stability analyses to evaluate the internal stability of the bottom liner system using peak shear strength values

Contents:

- Verification that the tensile stress in the bottom liner system will be less than yield stress is provided on Sheets IIID-C-44 through IIID-C-46.
- Infinite stability analysis to evaluate the internal stability of the bottom liner system is presented on Sheets IIID-C-47 through IIID-C-49.

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. USACE, *Slope Stability*, Engineering and Design Manual, EM 1110-2-1902, October 31, 2003.
4. Koerner, Robert M., *Analysis and Design of Veneer Cover Soils*, 1998 Sixth International Conference of Geosynthetics.
5. Koerner, George R. and Narejo, Dhani, *Direct Shear Database of Geosynthetic-to-Geosynthetic and Geosynthetic-to-Soil Interfaces,* GRI Report #30, June 14, 2005.
6. Gilbert, Robert B., *Peak Versus Residual Strength for Waste Containment Systems,*
7. Proceedings of the 15th GRI Conference, December 13, 2001.
8. NAVFAC Design Manual 7.01, September 1986.

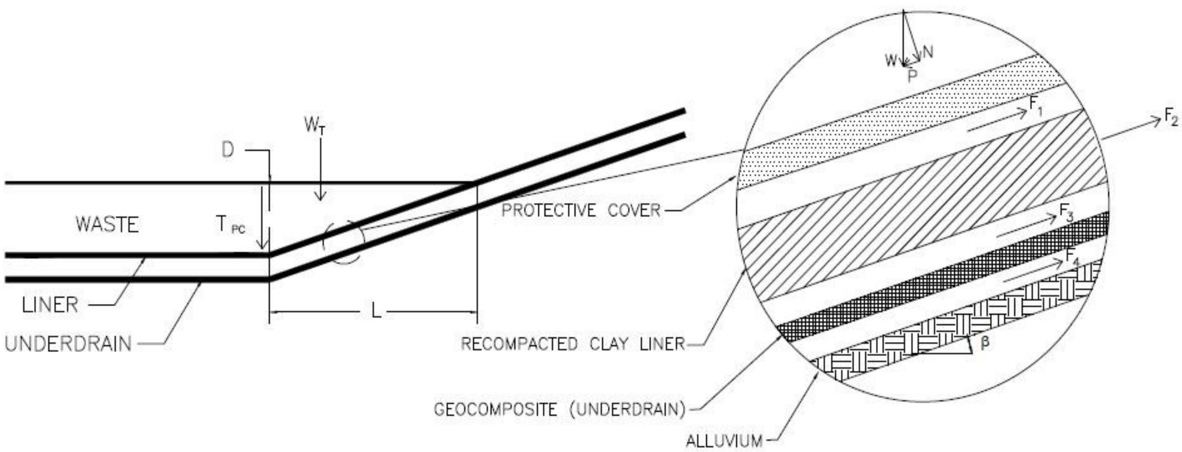
VENEER STABILITY OF RECOMPACTED CLAY LINER
AND GEOCOMPOSITE UNDERDRAIN

A. Liner System Stability

Note: The liner system includes a 1-foot-thick protective cover and a 3-foot-thick recompacted clay liner underlain by 200-mil geocomposi

1. Verify that tensile stress in liner system is less than yield stress for the liner system.

Recompacted Clay Layer (All Areas)



Assume a Caterpillar D8T WH Track-Type Tractor

Operational Weight =	85,150 lb
Number of Tracks =	2
Track Width =	1.84 ft

W_W = Weight of solid waste, lb/ft
W_{PC} = Weight of protective cover, lb/ft
W_T = Combined weight of equipment, solid waste, and protective cover, lb/ft
T_{PC} = Friction force on edge of protective cover, lb/ft
W = Net force of equipment, waste, and protective cover on liner system, lb/ft
N = Normal force on liner system, lb/ft
P = Shearing force on liner system, lb/ft
β = Slope angle, deg
F_n = Resisting force, lb/ft, calculated using the equation:
$(N * \tan(\Delta_n)) + (C_{an} * L / \cos(\beta))$

F_1 = Resistance of protective cover/recompacted clay liner, lb/ft
F_2 = Resistance of internal recompacted clay liner, lb/ft
F_3 = Resistance of recompacted clay liner/geocomposite, lb/ft
F_4 = Resistance of geocomposite/alluvium, lb/ft

FORT WORTH C & D LANDFILL
APPENDIX IIID-C
0771-356-11-35
VENEER STABILITY OF RECOMPACTED CLAY LINER AND GEOCOMPOSITE UNDERDRAIN

Δ_n = Interface friction angle of interface "n", deg
C_{an} = Adhesion of interface "n", psf
ϕ_n = Internal friction angle of material "n", deg
C_n = Cohesion of material "n", psf

γ_{was} = Unit weight of solid waste (including daily cover), pcf
D_{was} = Individual lift height, ft
ϕ_{was} = Internal friction angle of waste, deg
γ_{pc} = Unit weight of protective cover, pcf
D_{pc} = Thickness of protective cover and recompacted clay liner (combined), ft
ϕ_{pc} = Internal friction angle of protective cover/recompacted clay liner, deg
L = Horizontal length of lift, ft

Parameters:

$\beta_{sideslope}$ =	18.43	deg	γ_{was} =	90	pcf
Δ_1 =	19	deg	D_{was} =	10	ft
C_{a1} =	230	psf	ϕ_{was} =	33	deg
Δ_2 =	19	deg	γ_{pc} =	120	pcf
C_{a2} =	230	psf	D_{pc} =	4	ft
Δ_3 =	16	deg	ϕ_{pc} =	19	deg
C_{a3} =	100	psf	L =	30	ft
Δ_4 =	16				
C_{a4} =	100				

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 (or cohesion) and friction angle using project-specific soil and synthetic materials. The interface friction testing will be performed for the specific conditions analyzed. If test results differ from the assumed values, this analysis will be updated for acceptable factor of safety values using the procedure presented in the following sections.

Weight of Equipment

$$W_E = \frac{\text{Operational Weight}}{\text{Number of Tracks x Width of Track}}$$

W_E = 23,139 lb/ft

Weight of Solid Waste

$$W_W = \frac{D_{was} \text{ x } L \text{ x } \gamma_{was}}{2}$$

W_W = 13,500 lb/ft

Weight of Protective Cover

$$W_{PC} = D_{pc} \text{ x } \gamma_{pc} \text{ x } \frac{L}{\cos(\beta_{sideslope})}$$

W_{PC} = 15,178 lb/ft

Combined Weight of Equipment, Solid Waste, and Protective Cover/Recompacted Clay Liner,

$$W_T = W_E + W_W + W_{PC}$$

W_T = 51,817 lb/ft

Friction Force on Edge of Protective Cover

$$T_{PC} = k_o \times \sigma_v \times \tan\phi_{pc} \times D_{pc}$$

where: $k_o = 1 - \sin\phi_{pc}$

$$\sigma_v = \frac{D_{pc} \times \gamma_{pc}}{2}$$

$T_{PC} =$ 223 lb/ft

Net Force of Equipment, Waste, and Protective Cover on Liner System

$W = W_T - T_{PC}$ — $W =$ 51,594 lb/ft

$N = W\cos(\beta)$ — $N =$ 48,948 lb/ft

$P_{sideslope} = W\sin(\beta)$ — $P_{sideslope} =$ 16,311 lb/ft

Recompacted Clay Liner:

Resistance of Protective Cover/Recompacted Clay Liner = $F_1 =$ 24,127 lb/ft

$P_{sideslope} < F_1$ Therefore, protective cover soil is stable on the recompacted clay liner and a driving force equal to P is transferred to the next interface.

Resistance of Internal Recompacted Clay Liner= $F_2 =$ 24,127 lb/ft

$P_{sideslope} < F_2$ Therefore, the recompacted clay liner internally is stable and a driving force equal to P is transferred to the next interface.

Resistance of Recompacted Clay Liner/Geocomposite Interface= $F_3 =$ 17,198 lb/ft

$P_{sideslope} < F_3$ Therefore, recompacted clay liner is stable on the geocomposite and a driving force equal to P is transferred to the next interface.

Resistance of Geocomposite/Alluvium Liner= $F_4 =$ 17,198 lb/ft

$P_{sideslope} < F_4$ Therefore, the geocomposite is stable on the alluvium layer and a driving force equal to P is transferred to the next interface.

B. Infinite Slope Stability Analysis

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 (or cohesion) and friction angle using project-specific soil and synthetic materials. The interface friction testing will be performed for the specific conditions analyzed. If test results differ from the assumed values, this analysis will be updated for acceptable factor of safety values using the procedure presented in the following sections.

LINER SYSTEM

The liner system includes a 1-foot-thick protective cover and a 3-foot-thick recompacted clay liner

1. Use Duncan and Buchignani's method for infinite stability analyses to evaluate the internal stability of the liner, overliner, and final cover systems using peak shear strength values.

The factor of safety is calculated using the following equation:

$$F.S. = A\frac{\tan\Delta}{\tan\beta} + B\frac{C_a}{\gamma H}$$

where:
Δ = Interface friction angle, deg
C_a = Adhesion, psf
β = Slope angle, deg
A = Parameter A from chart on sheet IIID-C-50
B = Parameter B from chart on sheet IIID-C-50
γ = Unit weight of soil, pcf
H = Thickness of material above interface, ft

An example using the recompacted clay liner/geocomposite interface of the liner system is provided below.

A. Define the shear strength parameters (peak shear strength parameters will be used for this example).

Δ = 16 deg
C_a = 100 psf

B. Calculate the pore pressure, r_u, using the following equation:

$$r_u = (T \times \gamma_w \times \cos^2 b) / (H \times \gamma)$$

VENEER STABILITY OF RECOMPACTED CLAY LINER
AND GEOCOMPOSITE UNDERDRAIN

where: H = Thickness of material above interface, ft
γ_w = Unit weight of water, pcf
β = Slope angle, deg
T = Maximum head above interface, ft
γ = Unit weight of soil, pcf

H =	4	ft
γ_w =	62.4	pcf
β =	18.43	deg (3H:1V)
T =	0	ft
γ =	120	pcf

r_u = 0.00

Since T=0, there is no pore pressure build-up in the protective cover. If the soil material is assumed to be saturated, use a unit weight of 125 pcf for soil.

C. Calculate the slope ratio, b.

b = cot β = 3.0

D. Using r_u and b, determine Parameters A and B from the charts on sheet IIID-C-50.

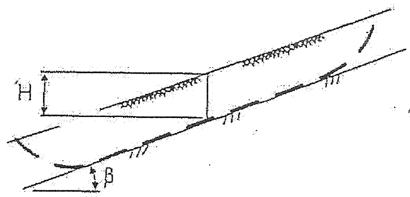
A = 1.0
B = 3.3

E. Calculate the factor of safety and compare against the minimum recommended factor of safety.

F.S. = 1.55 > $F.S._{min}$ = 1.5

FORT WORTH C & D LANDFILL
APPENDIX IIID-C
0771-356-11-35
VENEER STABILITY OF RECOMPACTED CLAY LINER AND GEOCOMPOSITE UNDERDRAIN

Component/Interface	Strength Parameters		H (ft)	γ (pcf)	β (deg)	T (ft)	r_u	b	A	B	Factor of Safety Generated	Recommended Minimum Factor of Safety	Acceptable Factor of Safety
	Cohesion/Adhesion (psf)	Friction Angle (deg)											
	Peak	Peak									Peak	Peak	Peak
Liner System - Reompacted Clay Liner Option (3H:1V Maximum Slope)													
Compacted Clay Liner													
Protective Cover/Recompacted Clay liner	230	19	1	120	18.43	0	0.00	3.0	1.0	3.3	7.36	1.5	YES
Recompacted Clay liner/Geocomposite	100	16	4	120	18.43	0	0.00	3.0	1.0	3.3	1.55	1.5	YES
Geocomposite/Alluvium	100	16	4	120	18.43	0	0.00	3.0	1.0	3.3	1.55	1.5	YES
Recompacted Clay Liner Internal	230	19	2.5	120	18.43	0	0.00	3.0	1.0	3.3	3.56	1.5	YES



γ = total unit weight of soil

γ_w = unit weight of water

c' = cohesion intercept

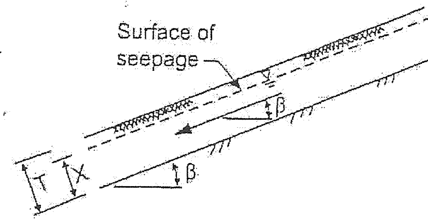
ϕ' = friction angle

r_u = pore pressure ratio = $u/\gamma H$

u = pore pressure at depth H

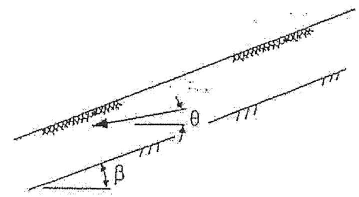
Steps.

1. Determine r_u from measured pore pressure or formulas at right.
2. Determine A and B from charts below.
3. Calculate $F = A\dfrac{\tan\phi'}{\tan\beta} + B\dfrac{c'}{\gamma H}$



Seepage parallel to slope

$$r_u = \frac{X}{T}\frac{\gamma_w}{\gamma}\cos^2\beta$$



Seepage emerging from slope

$$r_u = \frac{\gamma_w}{\gamma}\frac{1}{1+\tan\beta\tan\theta}$$

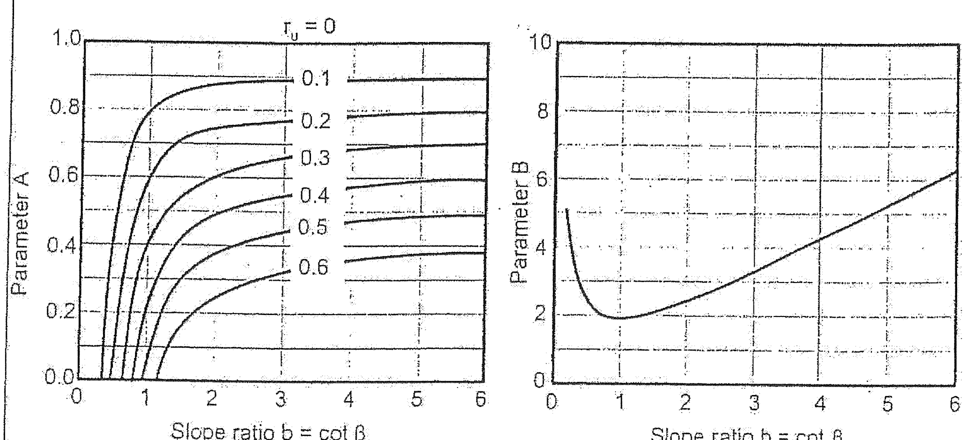


Figure E-7. Slope stability charts for infinite slopes (after Duncan, Buchianani, and DeWet 1987)

EVALUATION OF SIDEWALL BALLAST – SECTOR 6

Required: Provide calculations demonstrating that the recompacted clay liner and protective cover provide sufficient ballasting during construction of the Sector 6 sidewall liner system.

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

Definition of terms/variables:

H = Maximum groundwater head above top of clay liner, ft
P_{H20} = Maximum uplift pressure created by groundwater head, psf
$R_{pc + rcl , v}$ = Counteracting ballast pressure from protective cover and recompacted clay liner - vertical, psf
$R_{pc + rcl , n}$ = Counteracting ballast pressure from protective cover and recompacted clay liner - normal, psf
E_{H20} = Highest potentiometric surface elevation, ft-msl
E_{rcl} = Elevation of top of recompacted clay liner, ft-msl
γ_{H20} = Unit weight of water, pcf
$\gamma_{pc,rcl}$ = Unit weight of protective cover and recompacted clay liner, pcf
$T_{pc+rcl , v}$ = Thickness of protective cover and recompacted clay liner as ballast - vertical, ft
$T_{pc+rcl , n}$ = Thickness of protective cover and recompacted clay liner as 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
$FS_{pc+ccl , v}$ = Calculated factor of safety with protective cover and recompacted clay liner installed - vertical
$FS_{pc+ccl , n}$ = Calculated factor of safety with protective cover and recompacted clay liner installed - normal

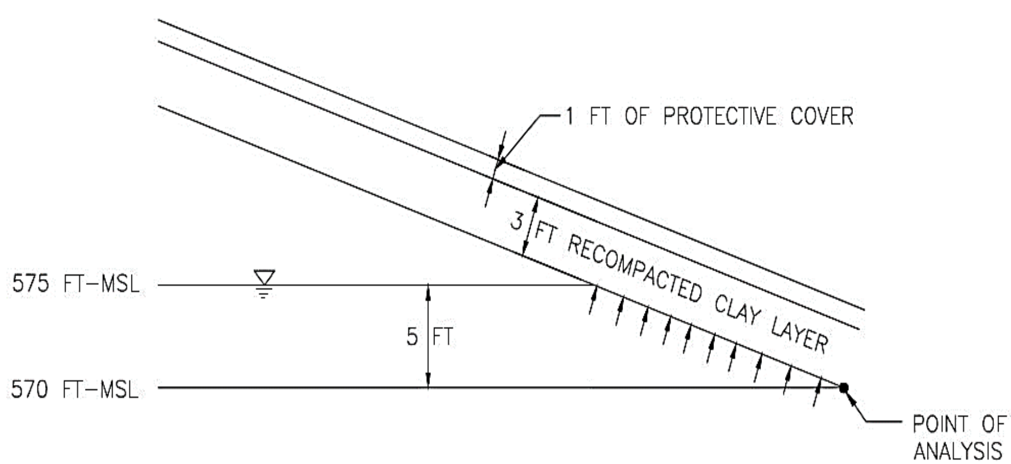


Diagram for Ballast Analysis in Sector 6

calculation using Evaluation Point No. 1:

Parameters:

E_{H20} =	575.0	ft-msl
E_{rcl} =	570.0	ft-msl
γ_{H20} =	62.4	pcf
b = side slope angle =	18.43	
cos b =	0.9487	
$T_{pc+rcl,v}$ =	4.2	ft ($T_{pc,v}$/cos β)
$T_{pc+rcl,n}$ =	4.0	ft
$\gamma_{pc,rcl}$ =	120	pcf

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

$H = E_{H20} - E_{rcl}$
H = 5.0 ft

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

$P_{H2O} = (\gamma_{H2O} \times H)$
P_{H2O} = 312 psf

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

$R_{pc+rcl,v} = (\gamma_{pc,rcl} \times T_{pc+rcl,v})$
$R_{pc+rcl,v}$ = 504 psf

$R_{pc+rcl,n} = (\gamma_{pc,rcl} \times T_{pc+rcl,n})$
$R_{pc+rcl,n}$ = 480 psf

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

$FS_{pc+rcl,v} = R_{pc+rcl,v}/P_{H2O}$ = 1.6

$FS_{pc+rcl,n} = R_{pc+rcl,n}/P_{H2O}$ = 1.5

The minimum required factor of safety for protective cover as ballast is 1.2 when using soil as ballast. Since the factor of safety against uplift is higher than 1.2 no additional ballast would be necessary indicating that the protective cover and recompacted clay liner provide enough ballast to counteract the hydrostatic uplift pressure acting at the top of protective cover and recompacted clay liner. The depth of alluvium requiring ballasting will be confirmed in the field during construction by the POR.

APPENDIX IIID-D

BALLAST EVALUATION REPORT FORMS

Texas Commission on Environmental Quality
Municipal Solid Waste Landfill
Ballast Evaluation Report

TCEQ

Part A: Facility Identification

Permittee: ______________________________

Permit No.: __________ Operational Classification Type: ______________________________

County: ______________________________

Part B: General Information

1. Describe liner system cross-section in bottom, sidewalls, leachate collection trenches, and sumps. ______________________________
____
____
____

2. Does the SDP require an active or passive dewatering system for this liner system?
____
____

3. Which cell, area, or sector does the BER represent? ______________________________
____
____

4. Date of the current LQCP that was used to develop this BER? ______________________________
   a. Was this plan followed? ______________________________
   b. If not followed, why not? ______________________________

5. Dates the certifying engineer and the technician visited the site (other than previously reported in SLER/SLER).
____
____

Part C: Groundwater and Ballast Data

1. Attach to this report a map(s) of the area under evaluation showing the site grid system and elevation contours of seasonal high groundwater level, liner system, and top of ballast. Also include actual groundwater elevation contours if lower than seasonal high groundwater levels due to dewatering or other causes if these lower groundwater levels are being used to demonstrate uplift stability during construction or during waste-as-ballast placement.

2. Attach instrumentation data (from piezometers, pneumatic pore pressure cells, etc.) taken during liner construction and since the end of construction or last BER.

3. Attach surveyed elevations of top of ballast. Was all surveying performed under the supervision of a registered surveyor? ____________________

4. Attach any test or other documentation of unit weights of soil materials used as ballast.

5. If waste was used as ballast, submit Waste-as-Ballast Placement Record (attached) with authorized signature of facility operator or permittee. Does the record indicate that the waste ballast is in accordance with the LQCP? ____________________

   If not, provide explanation. ____________________

   Does the record indicate that a minimum 40,000-pound wheeled compactor was used throughout the period covered by this BER? _____. If not, indicate the following:

   Time period covered? ____________________

   Approximate volume of airspace consumed during period? ____________________

   Tons of waste from landfill gate records during period? ____________________

   Approximate percentage of daily/intermediate cover? ____________________

   Unit weight of waste (attach calculations)? ____________________

   (Note: Ballast calculations must not use unit weight of waste greater than 1,200 lbs/yd^3).

Part D: Calculations of Uplift Stability

1. Provide calculated factors of safety against uplift for all critical locations in the area covered by this BER (see attached table). The factors of safety must be checked at critical points in the liner system (i.e. at bottom of geomembrane, bottom of compacted clay, etc.). The factors of safety must cover stability using the appropriate piezometric heads after completion of waste-as-ballast placement. Include sample uplift stability calculation(s).
2. Do the analyses conducted in D.1 indicate adequate factors of safety against uplift (1.2 if only soil is used as ballast and 1.5 if waste is used as ballast from the seasonal high groundwater level?

   ____

Part E: Engineer Certification

I certify that the liner has been constructed as designed in accordance with the issued permit and in general compliance with the regulations.

Affix Professional Engineer's Seal (Date & Sign)

[seal]

_

(date signed)

(typed or printed name)

(phone number)

(fax number)

(company or business name)

(address, city, zip code)

Note: A professional engineer must be registered in Texas.

Part F: Signature of Permittee

1. I have read and fully understand the findings of the BER submittal.
2. 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 there are significant penalties for submitting false information, including the possibility of fine and imprisonment for knowing violations.

(signature)

(typed or printed name)

(title)

(date signed)

(phone number)

(fax number)

(company or business name)

(*address, city, state, zip code*)

APPENDIX IIID-E

2021 BALLAST EVALUATION REPORT APPROVAL LETTER

Jon Niermann, *Chairman*
Emily Lindley, *Commissioner*
Bobby Janecka, *Commissioner*
Toby Baker, *Executive Director*

TEXAS COMMISSION ON ENVIRONMENTAL QUALITY

Protecting Texas by Reducing and Preventing Pollution

June 11, 2021

Mr. Brett O'Connor, P.E.
Senior Engineer
Texas Regional Landfill Company, LP
3 Waterway Square Place, Suite 550
The Woodlands, Texas 77380

Subject: Fort Worth C&D Landfill – Tarrant County
Municipal Solid Waste – Permit No. 1983D
Ballast Evaluation Report – Sectors 1, 2A, 2C, 2D, 2E, 3A, 3B, 3C, and 3D – Acceptance of Report
Tracking No. 26141317; CN601668486/RN101478790

Dear Mr. O'Connor:

We received a ballast evaluation report (BER) dated June 2, 2021 for Sectors 1, 2A, 2C, 2D, 2E, 3A, 3B, 3C, and 3D at the referenced landfill facility. The BER was prepared by Weaver Consultants Group, LLC and was signed and sealed by Mr. Nevjat Turan, as the Professional of Record, on June 2, 2021. The report documents thicknesses of waste and provides ballast calculations at critical locations in Sectors 1, 2A, 2C, 2D, 2E, 3A, 3B, 3C, and 3D.

The BER is accepted as the documentation submitted by Mr. Nevjat Turan, as the Professional of Record, indicates that the sufficient ballast has been placed in Sectors 1, 2A, 2C, 2D, 2E, 3A, 3B, 3C, and 3D to offset the seasonal high groundwater level in compliance with the issued permit and State of Texas municipal solid waste rules.

If you have any questions regarding this letter, please contact me at (512) 239-6727, or in writing at the address on our letterhead (please include mail code MC 124 on the first line).

Sincerely,

Chandra S. Yadav, P.E.
Municipal Solid Waste Permits Section
Waste Permits Division

CY/tw

cc: Mr. Nevjat Turan, P. E., Weaver Consultants Group, Fort Worth

P.O. Box 13087 • Austin, Texas 78711-3087 • 512-239-1000 • tceq.texas.gov

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FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS
TCEQ PERMIT NO. MSW-1983E

MAJOR PERMIT AMENDMENT APPLICATION

PART III – SITE DEVELOPMENT PLAN
APPENDIX IIIE
FINAL COVER SYSTEM QUALITY CONTROL PLAN

Prepared for

Texas Regional Landfill Company, LP

February 2023

STATE OF TEXAS
NEVZAT TURAN
84059
LICENSED
PROFESSIONAL ENGINEER
02/09/2023

STATE OF TEXAS
DAVID E. POE
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02/09/2023

Prepared by

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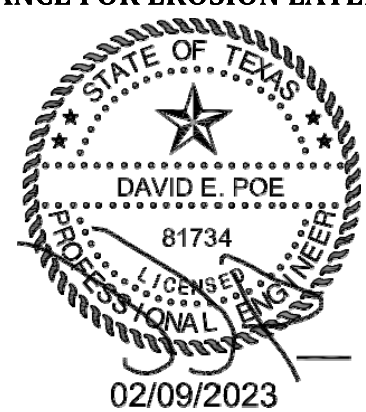
Fort Worth, TX 76109
817-735-9770

WCG Project No. 0771-356-11-35

This document is intended for permitting purposes only.

CONTENTS



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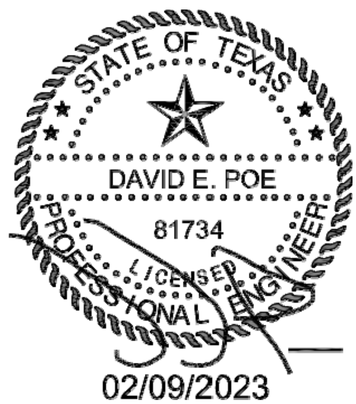
TABLES



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DAVID E. POE
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02/09/2023

1 INTRODUCTION

1.1 Purpose

This Final Cover System Quality Control Plan (FCSQCP) has been prepared to provide the Owner, 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 Regulations (MSWR). More specifically, the FCSQCP addresses the material requirements and installation of the final cover system.

This FCSQCP is divided into the following parts:

- Section 1 – Introduction
- Section 2 – Construction Quality Assurance for Soil Infiltration Layer
- Section 3 – Construction Quality Assurance for Erosion Layer
- Section 4 – Documentation

1.2 Definitions

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

ASTM

American Society for Testing and Materials.

Atterberg Limits

A series of six "limits of consistency" of fine-grained soils defined by Swedish soil scientist Albert Atterberg, two of which are frequently used today to establish a soil's physical boundaries dealing with its plasticity characteristics. These soil boundaries or limits used most frequently in geotechnical engineering are based upon the numerical difference of the Liquid Limit and the Plastic Limit 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.
- 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.
- 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 and the greater it's plasticity characteristics.

Compactive Effort

The amount of compaction energy held constant, and usually transferred into a soil sample with a compaction hammer device, used on soil samples in various laboratory test procedures to establish a soil's density at various moisture contents.

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 (EPA, 1986). 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 plans and specifications for a project.

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 two or higher for soils and FML 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) under the direction of the POR. Additional CQA monitors may be used. If working under the direction of a CQA monitor, the second CQA monitor will have a minimum of one year of directly related experience.

Construction Quality Assurance Professional of Record (POR)

The POR is an authorized representative of the permittee and has overall responsibility for construction quality assurance and confirming that the facility was constructed in general accordance with plans and specifications approved by the permitting agency. The POR must be licensed 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, 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. The POR or his designated representative will be on-site during all final cover 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).

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 drawings and specifications and Section 2 of this plan.

Final Cover System Evaluation Report (FCSER)

Upon completion of closure activities, the certification will be in the form of the FCSER which will be signed by the POR and include all the documentation necessary for certification of closure.

Independent Testing Laboratory

A laboratory that is independent of ownership or control by the permittee or any party to the construction of the final cover.

Nonconformance

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

Permittee

Waste Connections (i.e., Texas Regional Landfill Company, LP) is the permittee, owner, and operator of the facility. Permittee, owner, and operator refer to the same entity throughout this plan.

Permittee's Representative

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

Permeant Fluid

Fluid used in a laboratory coefficient of permeability test and limited to tap water or 0.005 Normal solution of $CaSO_4$. Distilled water will not be used in these test procedures.

Quality Assurance

This is a planned and systematic pattern of procedures and documentation to ensure that items of work or services meet the requirements of the contract documents. Quality assurance includes quality control. Quality assurance will be performed by the POR and CQA monitor.

Quality Control

These actions provide a means to measure and regulate the characteristics of an item or service to comply with the requirements of the contract documents. Quality control will be performed by the contractor.

Registered Surveyor

Registered surveyor in this plan means an individual who, during the entire duration of surveying work, holds a valid registration from the Board of Professional Engineers and Land Surveyors.

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 FCSQCP and requires a separate soil test series.

Soil Test Series

Tests performed to determine a soil's physical characteristics and to document its ability to satisfy the MSWR soil infiltration layer requirements. These tests include sieve analysis (gradation), Atterberg Limits, moisture/density, and coefficient of permeability.

2 CONSTRUCTION QUALITY ASSURANCE FOR SOIL INFILTRATION LAYER

2.1 Introduction

This section of the FCSQCP addresses the construction of the soil infiltration layer component of the final cover system and outlines the FCSQCP program to be implemented with regard to materials selection and evaluation, laboratory test requirements, field test requirements and treatment of problems.

The scope of soil infiltration layer related construction quality assurance includes the following elements:

- Subgrade preparation
- Soil infiltration layer stockpile
- Soil infiltration layer placement
- General fill

2.2 Soil Final Cover

The landfill is designed to include a soil final cover system over the waste fill footprint as discussed in Section 2.2 of the Closure Plan (Appendix IIIJ). Details for the final cover for the landfill are shown in Appendix IIIA – Landfill Unit Design. The final cover will be comprised of the following (from bottom to top):

- 18-inch-thick compacted soil infiltration layer composed of clay or clayey soil, as classified by the Unified Soil Classification System (USCS) as SC (clayey sand/sandy clay), CL (lean clay) or CH (fat clay). The soil infiltration layer will be compacted to a minimum 90 percent (refer to Section 2.3.2) of Standard Proctor density (ASTM D 698) and have a laboratory permeability of not greater than 1 x 10^{-5} cm/s.
- Topsoil layer capable of sustaining native plant growth and seeded or sodded immediately after installation. The topsoil layer will be 6-inch-thick for SC and CL infiltration layer soils and 12-inch-thick for CH infiltration layer soils. Soils with USCS classifications other than those listed above may be used in

the final cover at the discretion of the POR, provided the maximum permeability requirements are met.

2.3 Soil Infiltration Layer Construction

Sections 2.3.1 and 2.3.2 describe general construction procedures to be used for the soil infiltration layer and the preparation of the intermediate cover layer. Construction must be conducted in accordance with the project construction drawings, which will be developed in accordance with this FCSQCP and the Closure Plan (Appendix IIIJ) at the time of each final cover construction.

2.3.1 Soil Infiltration Layer Subgrade

Before soil infiltration layer construction, the vegetation on the intermediate cover will be removed. The surface of the intermediate cover will be prepared to establish the soil infiltration layer subgrade that is a working surface for the first lift of infiltration layer soil. The CQA monitor will visually inspect and approve the prepared subgrade prior to the placement of the soil infiltration layer or structural fill. Approval will be based on a review of test information, if applicable, and CQA monitoring of the intermediate cover preparation.

Surveying will be performed to verify that the finished intermediate cover is completed consistent with the lines and grades specified in the design.

2.3.2 Soil Infiltration Layer

The soil infiltration layer will consist of a minimum 18-inch-thick compacted soil barrier (measured perpendicular to the subgrade surface) that will extend along the sideslopes and topslopes of the landfill. The POR will evaluate soil borrow material prior to commencement of final cover construction. All borrow soils used in soil infiltration layers will have the following minimum values verified by testing in a third party soil laboratory:

- Soil Type – SC, CL, or CH
- Percent Passing the 1-inch Screen – 100 percent passing.
- Remolded Hydraulic Conductivity – less than or equal to $1x10^{-5}$ cm/s at 90 percent.

The soil infiltration layer material will consist of relatively homogeneous clay and clayey soils. The soil will be free of debris, rock greater than 1 inch in diameter, vegetative matter, frozen materials, foreign objects, and organics. Testing will be performed in accordance with Section 2.4 (refer to Table 2-1 for test methods) for each borrow source. A permeability test will be conducted on samples from each borrow source. The permeability test specimens will be prepared by laboratory compaction to a dry density of approximately 90 percent of the Standard Proctor (ASTM D 698) maximum dry density at a moisture content at or above the optimum moisture content. One Proctor moisture-density relationship and remolded permeability test will be required for each different material as determined by a change in the liquid limit or plasticity index of more than 10 points.

The lift thickness will be controlled so that there is total penetration through the loose lift under compaction into the top of previously compacted lift; therefore, the compacted lift thickness will not be greater than the pad or prong length of the compaction equipment. The material will be compacted to a minimum of 90 percent (refer to page IIIE-13) of the maximum dry density determined by Standard Proctor (ASTM D 698) at a moisture content between the Standard Proctor optimum and 5 percentage points above optimum. 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.

Because of possible variability of the available clay materials, additional stockpile testing will be performed if different physical properties of the borrow soil (color, texture, etc.) are observed by the CQA monitor, and the materials vary by more than ten points in either liquid limit or plasticity index from previously evaluated materials.

The clay materials to be used for infiltration layer may require processing to achieve the required moisture content for compaction. The physical characteristics of the clay materials will be evaluated through visual observation before and during construction. To add moisture to the material properly, the clod sizes will first be crushed into manageable sizes of 1 inch in diameter or less. Rocks within the infiltration layer should be less than 1 inch in diameter and will not total more than 10 percent by weight.

Clod-size reduction, if necessary, may be achieved using a disc harrow, soil pulverizer, or other method acceptable to the POR. In order to efficiently break down the clods and pieces of shale, multiple passes of the processing equipment in two directions are recommended. 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 infiltration layer to allow the soil adequate time to hydrate. Water used for the soil infiltration layer must be clean and not contaminated by waste or any objectionable material. Collected onsite stormwater may be utilized to moist-condition soils if it has not come into contact with the solid waste.

The soil infiltration layer must be compacted with a pad/tamping-foot 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 compacted lift thickness must not be greater than the pad or prong length. The top of intermediate cover will be scarified a minimum of two inches prior to placement of the first lift of soil infiltration layer. Use of pad/tamping foot or prong-foot rollers will provide sufficient roughening of soil infiltration layer lift's surface for bonding between 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 penetration during initial 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, and a minimum of four passes are recommended for the compaction process. A pass is defined as one pass (1 direction) of the compactor, not just an axle, over a given area. The recommended minimum of four passes is for a vehicle with front and rear drums. The Caterpillar 815B and 825C are examples of equipment typically used to achieve satisfactory results. The soil infiltration layer will not be compacted solely with a bulldozer or any track-mobilized equipment unless it is used to pull a pad-footed roller.

CQA testing of the soil infiltration layer will be performed as the infiltration layer is being constructed. Testing procedures, frequency, and passing criteria will be in accordance with Section 2.4 (Table 2-2).

Soil infiltration layer construction and testing will be conducted in a systematic and timely fashion on each lift. In general, delays will be avoided in infiltration layer construction (typically no more than 14 days). Reasons for any delays in infiltration layer construction (greater than 14 days) should be fully explained in the FCSER submittal.

The finished top surface of the soil infiltration layer will be uniformly graded by back-dragging to allow accurate measurement of the infiltration layer thickness by surveying. Smooth drum rolling is not required, as this would reduce the interface strength of the infiltration layer and the overlying topsoil.

Surveying will be performed to document that the finished soil infiltration layer has been constructed to a minimum thickness of 18 inches. Thickness verification may be performed by comparison of survey data from control points or by using settlement plates (e.g., plywood sheet or similar material) on a 8,000 square foot grid. The infiltration layer will be surveyed as indicated in Table 2-2 to verify that a minimum 18-inch-thick soil layer is present at each location.

A typical settlement plate diagram is shown on Figure IIIE.1. The location of the settlement plates will be established by a Texas registered surveyor on a 8,000 square foot grid (refer to page IIIE-14). The shaft extending upward from the base will be marked to indicate the minimum required thickness of the infiltration layer.

The infiltration layer will be constructed to the minimum thickness marked on the shaft of the settlement plate. The POR and CQA monitor will verify that the infiltration layer is placed uniformly between each settlement plate.

An infiltration layer thickness certification drawing at each of the survey measurement grid points will be provided. Coordinates defining the perimeter of the final cover system will be called out on the final drawings. The infiltration layer thickness certification drawing will be sealed by a Texas registered surveyor. After the construction of the infiltration layer is complete, the Texas registered surveyor will survey the final elevation of the infiltration layer. The infiltration layer certification drawing will be included in the FCSER. In addition, the elevations obtained for the top of the infiltration layer will be used to verify that the as-built slopes are consistent with the approved landfill completion plan (refer to Appendix IIIA – Landfill Unit Design). A statement that confirms that the as-built slopes are consistent with the approved landfill completion plan will be included in the FCSER.

Once the survey is complete, the settlement plate shafts (if used) will be removed and the resulting holes will be backfilled with bentonite or a bentonite/infiltration layer soil mixture consisting of at least 20 percent bentonite.

Testing and evaluation of the soil infiltration layer during construction will be in accordance with this FCSQCP. The construction methods and test procedures documented in the FCSER will be consistent with the FCSQCP.

The soil infiltration layer will be prevented from losing excessive moisture prior to placement of the topsoil layer. Preserving the moisture content of the installed soil infiltration layer will be dependent on the earthwork contractor's means and methods and is subject to POR approval.

Sections of the soil infiltration layer which do not pass both the density and moisture requirements will be reworked with additional passes of the compactor until the section in question passes. All field density test results will be incorporated into the FCSER.

Hydraulic conductivity samples will be obtained by pushing a sampler through the constructed infiltration layer. 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 (e.g., samples for hydraulic conductivity) will be backfilled with bentonite or a bentonite/infiltration layer soil material mixture consisting of at least 20 percent bentonite.

If the integrity of the "A" sample appears to have been compromised during the 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 maximum 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 FCSER.

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 is determined, the constructed soil infiltration layer 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 per lift will be performed for each repair area, given that the reconstructed soil infiltration layer area is not larger than one acre. The reconstructed soil infiltration layer area will be tied into the currently constructed soil infiltration layer. Repair area lifts will be tied to previously installed areas per 2.3.4. The reconstructed soil infiltration layer 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). The testing frequency for reconstructed areas will be in accordance with Table 2-2.

Reconstruction activities, including additional testing and surveying, will be incorporated into the FCSER.

2.3.3 Surface Water Removal

The prepared intermediate cover or infiltration layer which is under construction may encounter water from storm events. Prior to placement of the soil infiltration layer, intermediate cover will be graded to provide positive drainage for the base grades of the soil infiltration layer. The soil infiltration layer will not be placed in standing water and water will not be allowed to accumulate over constructed infiltration layer. The construction area will be graded to provide for positive drainage. Temporary diversion berms will be constructed as needed to divert surface flow away from the construction area.

2.3.4 Infiltration Layer Tie-In Construction

Newly constructed infiltration layer will be tied-in with any adjoining existing infiltration layers. Additionally, terminations will be constructed for future tie-ins along edges where the infiltration layer will be extended in the future. During the construction of continuous infiltration layers, the new infiltration layer segment will not be constructed by "butting" the entire thickness of the new infiltration layer directly against the edge of the old infiltration layer. The tie-in will be constructed either by a sloped transition (typically 5H:1V) or a stair-stepped transition (typically 1 lift thickness per step). The length of the tie-in should be at least 5 feet per foot of infiltration layer thickness. The tie-ins with existing clay infiltration layer will be constructed utilizing a sloped or stair-stepped transition. In general, terminations for future tie-ins will be constructed by extending the infiltration layer approximately 7.5 feet past the limits for the final cover area under construction.

2.4 Construction Testing

2.4.1 Standard Operating Procedures

CQA monitors will perform field and laboratory tests in accordance with applicable standards specified in this FCSQCP. Sampling will be performed by using standard ASTM practices for recovering thin-walled tube samples (ASTM D 1587). The sampling holes (i.e., samples collected for hydraulic conductivity testing) will be backfilled with bentonite or bentonite/infiltration layer soil material mixture consisting of at least 20 percent bentonite.

2.4.2 Test Frequencies

The test frequencies of borrow soils for the infiltration layer are listed in Table 2-1. The testing frequencies required during construction of the infiltration layer are listed in Table 2-2. Additional testing must be conducted whenever work or materials are suspect, marginal, or of poor quality. Further 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.

2.5 Reporting

The POR on behalf of the Operator will submit to the TCEQ a FCSER for approval of each final cover area. Section 6 describes the documentation requirements.

Table 2-1
Standard Tests Soil Borrow for Infiltration Layer Soils

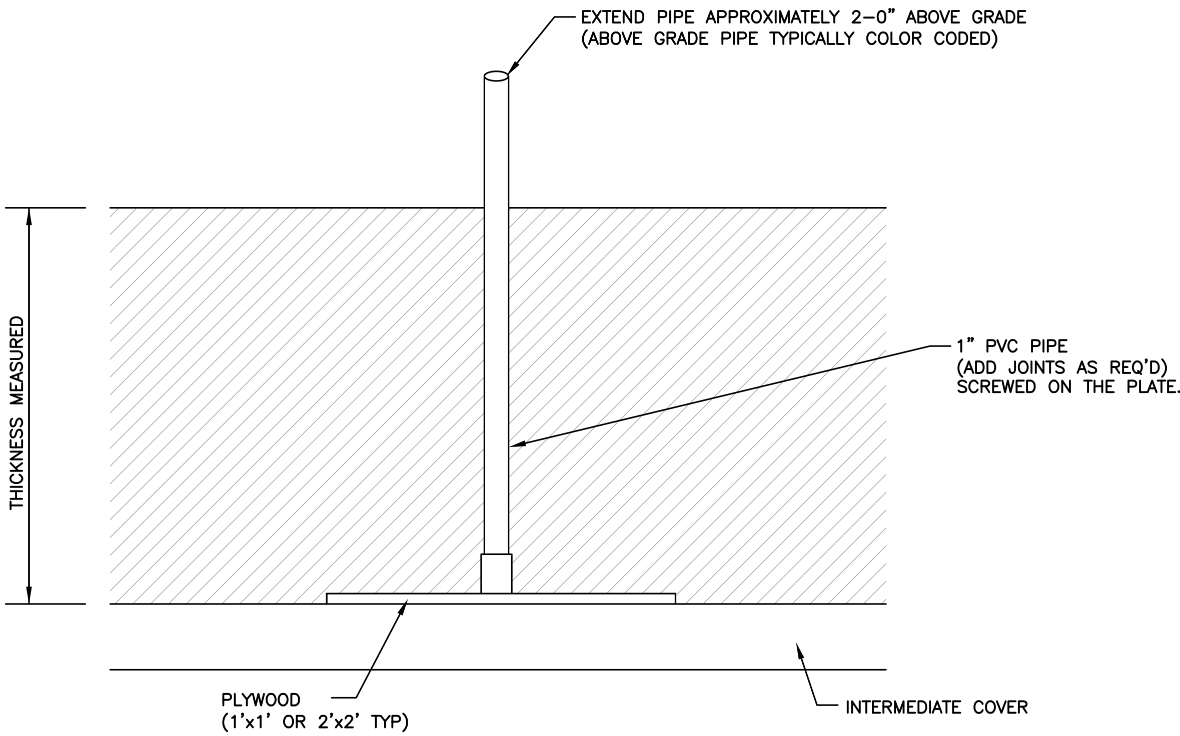
Test[1]	Specification	Standard	Frequency
Moisture/Density Relationship	Determine moisture/density curve using a minimum of four data points	ASTM D 698	One per soil type
Coefficient of Permeability (Remolded Sample)[2]	$1.0x10^{-5}$ cm/s or less	COE EM1110-2-1906	
Plasticity Index	No specified requirement	ASTM D 4318	
Liquid Limit, percent	No specified requirement	ASTM D 4318	
Percent Passing No. 200 Mesh Sieve	No specified requirement	ASTM D 1140	
Percent Passing 1-inch Sieve	100	ASTM D 448	
Unified Soil Classification	Reported in moisture/density test for soils meeting liquid limit, elastic limit, and percent passing -200	ASTM D 2487	

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 90% of the maximum dry density (determined using Moisture/density test) at or above the optimum moisture content. The POR may require 95% of the maximum dry density based on the hydraulic conductivity test results for the 90% remolded soil sample.

Table 2-2
Required Tests and Observations for Infiltration Layer and Final Cover

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 [2]	90% Maximum Standard Proctor Dry Density. Standard Proctor optimum moisture content or greater 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	N/A
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	N/A
Coefficient Permeability (Hydraulic Conductivity) [1]	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^{-5} cm/s or less
Thickness Verification [3]	1 each 8,000 square feet with a minimum of 2 reference points by a licensed Texas land surveyor	Survey subgrade and top of infiltration layer and erosion layer	18-inch minimum compacted clay thickness and 6 or 12 inch minimum erosion layer thickness

[1] Field permeability testing in accordance with Title 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.
[2] This method is not applicable if the field nuclear gauge reads both density and moisture.
[3] The infiltration layer will be constructed in parallel lifts.



SETTLEMENT PLATE (TYP)

NTS

NOTES:

1. PIPE MAYBE MARKED OR COLOR CODED TO CLEARLY INDICATE THE THICKNESS MEASURED. UPON SURVEY AND CERTIFICATION BY SURVEYOR PER SECTION 2.3.2, PIPE WILL BE UNSCREWED AND REMOVED IN ACCORDANCE WITH SECTION 2.3.2.

STATE OF TEXAS
DAVID E. POE
81734
LICENSED
PROFESSIONAL ENGINEER
02/09/2023

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TYPICAL SETTLEMENT PLATE

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

Weaver Consultants Group
REGISTRATION NO. F-3727

DRAWN BY: JDW	DATE: 12/2022	FILE: 0771-356-00
REVIEWED BY: DEP	CAD: FIG IIIJ-A.1.DWG	FIGURE IIIE.1

O:\0771\356\EXPANSION 2022\PART III\IIIE\III-E.1-SETTLEMENT PLATE.dwg, jwilson, 1:1

3 CONSTRUCTION QUALITY ASSURANCE FOR EROSION LAYER

The erosion layer will consist of a minimum of 6 or 12 inches of earthen material and will be capable of sustaining native and introduced vegetative growth and must be seeded immediately after completion of the final cover. A minimum 6-inch-thick erosion layer will be required over SC and CL infiltration layer soils, and a minimum 12-inch-thick erosion layer will be required over CH infiltration layer soils. Temporary or permanent erosion control materials may be used to minimize erosion and aid establishment of vegetation. The physical characteristics of the erosion layer will be evaluated through visual observation (and laboratory testing if deemed necessary by the POR) before construction and visual observation during construction. Additional testing during construction will be at the discretion of the POR.

The erosion layer may be placed using any appropriate equipment capable of completing the work and should only receive minimal compaction required for stability.

The thickness of the erosion layer will be verified with surveying procedures at a minimum of one survey point per 10,000 square feet of constructed area by a licensed Texas surveyor with a minimum of one reference point. The survey results for the erosion layer will be included in the FCSER.

During construction the CQA monitor will:

- Verify that grade control is performed prior to work.
- The POR will coordinate with the project surveyor to perform a thickness verification survey of the erosion layer materials upon completion of placement operations. Verify corrective action measures as determined by the verification survey. Thickness surveying to determine minimum erosion layer thickness will be performed similar to the infiltration layer thickness verification shown in Table 2-2.

4 DOCUMENTATION

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

4.1 Preparation of FCSER

The POR, on behalf of the Operator, will submit to the TCEQ a FCSER for approval of each portion of final cover system constructed.

Testing, evaluation, and submission of the FCSER for the final cover system during construction will be in accordance with this FCSQCP. The construction methods and test procedures documented in the FCSER will be consistent with this FCSQCP.

At a minimum, the FCSER will contain:

- A summary of all construction activities.
- All laboratory and field test results.
- Documentation of thickness of the infiltration and erosion layers by a Texas registered Surveyor.
- Sampling and testing location drawings.
- A description of significant construction problems and the resolution of these problems.
- As-built record drawings, including all previous FCSER submittals and dates of TCEQ approval.
- A statement of compliance with the permit FCSQCP and construction plans.
- The reports will be signed and sealed by a professional engineer(s) licensed in the State of Texas.

The as-built record drawings will accurately site the constructed location of work items. The POR will review and verify that as-built drawings are correct. As-built drawings will be included in the FCSER.

4.2 Reporting Requirements

The FCSER will be signed and sealed by the POR, signed by the Operator, and submitted to the MSW Permits Section of the Waste Permits Division of the TCEQ for approval.

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS
TCEQ PERMIT NO. MSW-1983E

MAJOR PERMIT AMENDMENT APPLICATION

PART III – SITE DEVELOPMENT PLAN
APPENDIX IIIF
SURFACE WATER DRAINAGE PLAN

Prepared for

Texas Regional Landfill Company, LP

February 2023

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WCG Project No. 0771-356-11-35

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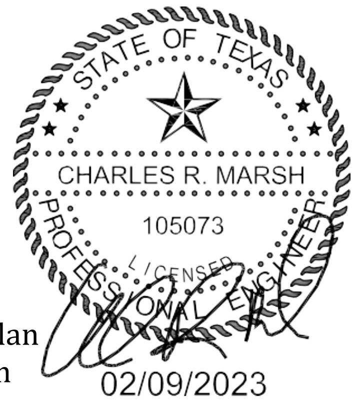
CONTENTS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CONTENTS (Continued)

CONTENTS (Continued)

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 Fort Worth C&D 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 the proposed landfill development will not adversely alter the existing permitted drainage patterns. As noted in Section 4, the proposed condition represents the final configuration of the site after the 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.

To provide a complete and relevant comparison between the permitted and post-development conditions, the existing permitted landfill layout was evaluated using the latest precipitation data, different hydrograph methodology, and updated offsite drainage area information. These updates are discussed further in Section 4.

1.3 Floodplain

As a part of the proposed expansion, a CLOMR was prepared for the landfill area as the proposed development areas include the 100-year floodplain. The current effective FEMA Flood Insurance Rate Map for the area of the landfill is provided on Figure 4.6 and excerpts from the approved CLOMR and the FEMA approval letter are included in Appendix IIIF-G. The 100-year floodplain related design and demonstrations developed as part of this application meet the requirements set forth in 30 TAC §330.307. As shown in Appendix IIIF-G, the 100-year floodplain will be contained around the landfill footprint and will not encroach on the limit of waste.

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 channels. The perimeter channels collect runoff from drainage letdowns and convey the runoff to stormwater detention ponds. The perimeter channels will be constructed incrementally as the site develops. The perimeter drainage system will be constructed in the general sequence shown on Parts I/II Drawings I/IIA-4 through I/IIA-6. As shown on Drawing IIIF.1 – Drainage Structure Plan, runoff generated from the developed areas will be conveyed through perimeter channels on the south and east sides of the fill areas and a series of detention ponds on the north side to be attenuated before being discharged into Village Creek.

In the permitted condition, runoff from the areas east of the permit boundary is routed around the north or south sides of the landfill through the perimeter channels on the south side of the landfill or through perimeter channels on the east side that flow to a series of four detention ponds north of the landfill. Both the southern channels and northern ponds outfall to an open area west of the landfill before being discharged to Village Creek, which flows in a south to north direction on the west side of the landfill property.

As a part of the proposed landfill development, the area west of the fill area will be excavated to provide overbank floodplain storage for Village Creek, as shown on Drawing IIIF.1. The floodplain storage area will effectively act as a large vegetated open space to collect and eventually discharge flows from the upstream perimeter channels and ponds. The floodplain storage area does not have an outlet structure; therefore, it is not considered a pond that will detain water. Additionally, the series of ponds along the northern edge of the landfill have been reconfigured to attenuate the flow before discharging into the floodplain storage area. An additional pond has also been added west of the entrance facilities to provide additional storage capacity.

Due to the 100-year floodplain being located above the culvert outlet of the last detention pond (P5) in the series of ponds on the north side of the landfill, a flap gate will be installed on the outlet side of the culvert. A flap gate will only allow flow in the downstream direction during flooding events. For more information about detention pond and culvert design, see Appendix IIIF-B.

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. Fort Worth C&D 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 – Contaminated Water Management Plan.

2.2 Erosion and Sedimentation Control Plan

The Fort Worth C&D 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 north 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 – 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), Fort Worth C&D Landfill 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 can be 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 Village 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-417, May 2018) and the TxDOT Bridge Division Hydraulic Design Manual, September 2019.

Water surface profiles were determined for the perimeter channels using the Channel Analysis Program (HYDROCALC HYDRAULICS Version 2.0.1 for Windows, Dodson & Associates, 1996-2010) 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 before exiting offsite. Manning's *"n"* values for the channels and culverts were taken from the TxDOT Bridge Division Hydraulic Design Manual (Table 4-7, page 4-43; and Table 4-9, page 4-46), September 2019.

3.2 Hydrologic Analysis

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 hydrologic analysis for the post-development condition is presented in Appendix IIIF-A. The hydrologic analysis for the permitted landfill completion condition is included in Appendix IIIF-E.

3.2.2 Watershed Subareas and Schematization

The landfill areas that contribute flow to each 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 as well as in Appendix IIIF-E for the permitted completion condition.

Offsite areas (areas outside the permit boundary) incorporated into the hydrologic analyses as appropriate have been delineated using topography obtained from the United States Geological Survey 7.5-Minute Quadrangle for Alvarado, Texas and Grandview, Texas. The offsite drainage area delineation is shown on Figure 4.3 for the post-development discharge analysis. The offsite areas are also included in the hydrologic analysis for the permitted landfill completion condition, as shown in Appendix IIIF-E.

3.2.3 Time Step

The time step, or the program computation interval, is the time interval at which the flow rates for the hydrographs are generated by the program. Time step used for a design storm event hydrograph generation is 5 minutes.

3.2.4 Hypothetical Precipitation

The hypothetical storm data used in the post-project analysis was obtained from the NOAA Atlas 14 for the project area and is consistent with the existing permitted data. 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 area modeled 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 86 was selected to represent the final cover sideslopes, and a CN of 84 was selected for final cover top dome surfaces. A CN of 99 was used for the detention pond areas. Further discussion on selection of CN values is provided in Appendices IIIF-A and IIIF-E for post-development and permitted landfill completion conditions, respectively.

3.2.6 Hydrograph Information

Two different types of hydrograph generation methods have been used in the drainage analyses: distributed runoff methods and the Snyder unit hydrograph method using the Espey "10-Minute" method for parameter estimation. Muskingum-Cunge and pond storage discharge 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.

Distributed Runoff Methods

The distributed runoff method (e.g., kinematic wave method) is applicable to small stormwater catchments with uniformly sloped overland flow plains that drain into channels. Landfill final cover areas consist of relatively short (typically 100 feet on 3H: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 of final cover that drains to a swale.
- These methods were developed for a network of relatively small drainage areas. Typically, to design the various perimeter channels, landfill drainage areas need to be subdivided to determine a peak flow at several points.
- These methods are also inherently conservative because it is based on watershed dimensions as opposed to other methods that use empirical information. 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., offsite drainage areas). The method is applicable to drainage areas with a wide range of characteristics. 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 areas.

Hydrograph Routing

The Muskingum-Cunge 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 storage/elevation relationships for each pond by defining pond surface area versus depth. Additionally, discharge structure (low level outlet and spillway) characteristics of each 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.7 through IIIF.12. The swales and channels are designed to convey the peak flow rate generated by the 25-year storm event with at least one foot of freeboard from the top of the channel. The additional foot of freeboard will be used to convey the flows generated by the 100-year storm event. These swales and channels will also reduce maintenance at the site after closure by minimizing erosion.

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 2019 TXDOT *Bridge Division Hydraulic Design Manual* (“Suggested Manning’s Roughness Coefficients” Table, Chapter 6, Section 1). A Manning’s “n” value of 0.030 is used for swales, a value of 0.040 for gabion-lined chutes, a value of 0.010 for FML-lined chutes, and a value of 0.030 for vegetation-lined or turf reinforcement mat-lined perimeter channels. These values represent typical roughness coefficients to the proposed drainage structures, after vegetation has become established.

3.3.2 Drainage Letdown Structure (or Chute) Analysis

A typical chute detail is illustrated on Drawing IIIF.9. 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 dissipators controls 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 dissipators.

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 landfill development will not adversely alter the existing permitted landfill completion condition drainage patterns. The appendices containing the two drainage conditions analyzed are listed below.

- Appendix IIIF-A (Post-Development Condition Hydrologic Calculations) – This appendix contains analysis and supporting calculations for the proposed configuration of the site after development of the expanded landfill is complete.
- Appendix IIIF-E (Updated Permitted Condition Hydrologic Calculations) – This appendix contains excerpts from the 2021 expansion permit document that establish the currently-permitted drainage patterns and peak flow rates for the permit boundary area. Section 4.3.1 includes a discussion of analyses performed to facilitate a comparison between the existing permitted and post-development conditions.

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 Fort Worth C&D is located in the Village Creek watershed, approximately one-mile south of Lake Arlington. All of the site drains to Village Creek, which flows adjacent to the majority of the western permit boundary.

The total drainage area of the Village Creek watershed is approximately 123 square miles. The site comprises approximately 0.3 percent of the Village Creek watershed. The Village Creek watershed is shown on Figure 4.2.

4.2 Site Drainage Patterns

The existing permitted, updated permitted, and post-development site drainage patterns are detailed on Figures 4.4 and 4.5. As shown on Figure 4.5, the post-development drainage patterns are consistent with the currently permitted and updated permitted drainage patterns. This facility includes one outfall at the permit boundary and multiple runon locations (locations where upstream offsite areas flow onto the permit boundary) for both the updated permitted and proposed conditions.

As shown on Figure 4.4, the total drainage area of the permit boundary is unchanged by the proposed expansion. However, the change in hydrologic methodology and offsite drainage area delineation led to the development of the updated permitted condition. Supplementing the existing permitted condition analysis with an updated hydrologic model allows for a direct comparison to be made between the permitted and post-project conditions. As shown in the onsite drainage area information on Figures 4.4 and 4.5, the updated permitted and proposed onsite drainage delineations are consistent.

The total drainage area to the single outfall location (DP1) is consistent between the updated permitted and post-project conditions (218 acres). In the existing permitted drainage analysis, the offsite areas included approximately 10 fewer areas of runon. The updated permitted and post-development conditions were adjusted to better model offsite conditions using contours from the USGS 7.5-minute quadrangle topographic map.

4.3 Effect of Site Development on Drainage from the Site

The purpose of this section is to evaluate the peak flow rates, runoff volumes, and peak flow velocities of the updated permitted and post-development hydrologic conditions. A summary of peak flow rates, runoff volumes, and peak flow velocities entering and exiting the permit boundary is provided in Table 4.1 and Figure 4.5 – Site Drainage Patterns, Runon/Runoff. Section 4.3.1 discusses the updated permitted landfill completion condition drainage analysis and how its input and methodology compares to the post-development condition.

Sections 4.3.2 through 4.3.5 discuss the impact of the proposed landfill conditions on peak flow rates, runoff volumes, and peak flow velocities entering and exiting the permit boundary.

4.3.1 Comparison of Existing Permitted and Updated Permitted Analyses

4.3.1.1 Overview of Updated Permitted Condition

The drainage analysis included in TCEQ Permit No. 1983D (for the purpose of this appendix, this case will be designated the "existing permitted condition") was developed in 2020 by Geosyntec Consultants, Inc. (TCEQ approval May 20, 2021). These documents utilized different hydrologic methodologies, precipitation data, and offsite areas than what is used in the post-development condition. In order to develop a direct comparison between the existing permitted and post-development conditions, a separate HEC-HMS analysis was developed for the existing permitted condition. This analysis is included in Appendix IIIF-E. As noted in Section 1.2, to comply with Title 30 TAC §330.63(c)(1)(C), the proposed landfill completion condition is compared to the existing permitted condition of the landfill to demonstrate that the continued development of the landfill will not adversely alter the existing permitted drainage patterns. This comparison is only meaningful if both the post-development and existing permitted conditions are based on consistent drainage information, including the same permit boundary. A discussion of the model parameters used in the existing permitted condition and the "updated permitted condition" is included in Section 4.3.1.2.

Additionally, runoff volume and velocity calculations for all discharge locations were not included in the existing permitted drainage calculations at all discharge locations. These calculations were prepared as a part of this application and are included in Appendix IIIF-E.

4.3.1.2 Model Parameter Comparison

Updates to the existing permitted condition are listed below.

- In the existing permitted condition, it is unclear what topographic data was used to delineate drainage basins outside of the permit boundary. To make this clear, Offsite Areas O1 through O4 were delineated using topographic information from the 2019 USGS 7.5-minute quadrangle topographic map, which led to slightly more area (10 acres) discharging onto the permit boundary.
- To be consistent with methods utilized in recently approved TCEQ applications, precipitation loss, hydrograph development, channel routing, and pond storage routing methods were updated as follows:
  - Curve numbers for all drainage methods were updated to 84 for all non-landfill drainage areas, 84 for landfill top dome surfaces, 86 for landfill side slope surfaces, and 99 for ponds based on tabulated curve numbers for the land uses of these areas (see Appendix IIIF-E). Curve

numbers in the existing permitted condition are 71 for most non-landfill drainage areas, and 85 for landfill drainage areas.

- Hydrographs are developed in the updated permitted landfill completion condition using distributed runoff methods or Snyder's unit hydrograph, as discussed in Section 3.2.6. The existing permitted condition utilizes the SCS unit dimensionless hydrograph for all drainage areas.
- The channel routing mechanism was updated to the Muskingum-Cunge Method for all channels in the HEC-HMS Model.
- Initial abstraction parameters were considered 0 in the updated permitted condition to provide a conservative analysis. The initial abstraction in the existing permitted condition was 0.35 for all drainage areas, and supporting information was not cited.

- The drainage area delineation for the currently permitted landfill areas has been updated to model letdown structures as single drainage areas instead of modeling drainage areas for individual swales. This update provides simplified but accurate flow rates for the drainage letdown structures.

4.3.1.3 Comparison of Peak Flows at the Permit Boundary

As shown in Figure 4.4, flow discharges to the northwest side of the permit boundary to Village Creek at location DCP1. The existing and updated permitted conditions are different due to the re-delineation of offsite drainage areas and updated methodology as discussed on Section 4.3.1.2. The 25-year peak flow rate at location DCP1 is lower for the updated permitted condition.

4.3.2 Peak Flow Rates

As shown in Table 4.1, post-development peak flow rates for the 25-year frequency storm (design storm) are lower than the existing permitted design peak flow rates at the stormwater discharge location at the permit boundary. The major discharge location from the site occurs at discharge location DCP1. The reconfiguration of the series of detention ponds on the north side of the landfill as well as the addition of another detention pond west of the entrance facility results in lower post-development peak flow rates at the northwest discharge of the site.

4.3.3 Discharge Volumes

The total volume of runoff discharged from the site is increases slightly at DCP1. The increase in DCP1 is mainly due to the increased developed landfill final cover areas, sideslopes, and perimeter channels and the proposed stormwater detention ponds. The increased volume of runoff generated by the proposed development is mitigated by the drainage improvements proposed to be constructed that release at lower peak rates than the permitted condition.

4.3.4 Discharge Velocities

Consistent with the decreased flow rates at the permit boundary, the velocities at the permit boundary for the post-development condition are lower than the currently permitted condition. Since the post-development peak design storm discharge rates are lower at the permit boundary and the geometry changed at the discharge location due to the floodplain storage area, the velocities are lower compared to the currently permitted condition.

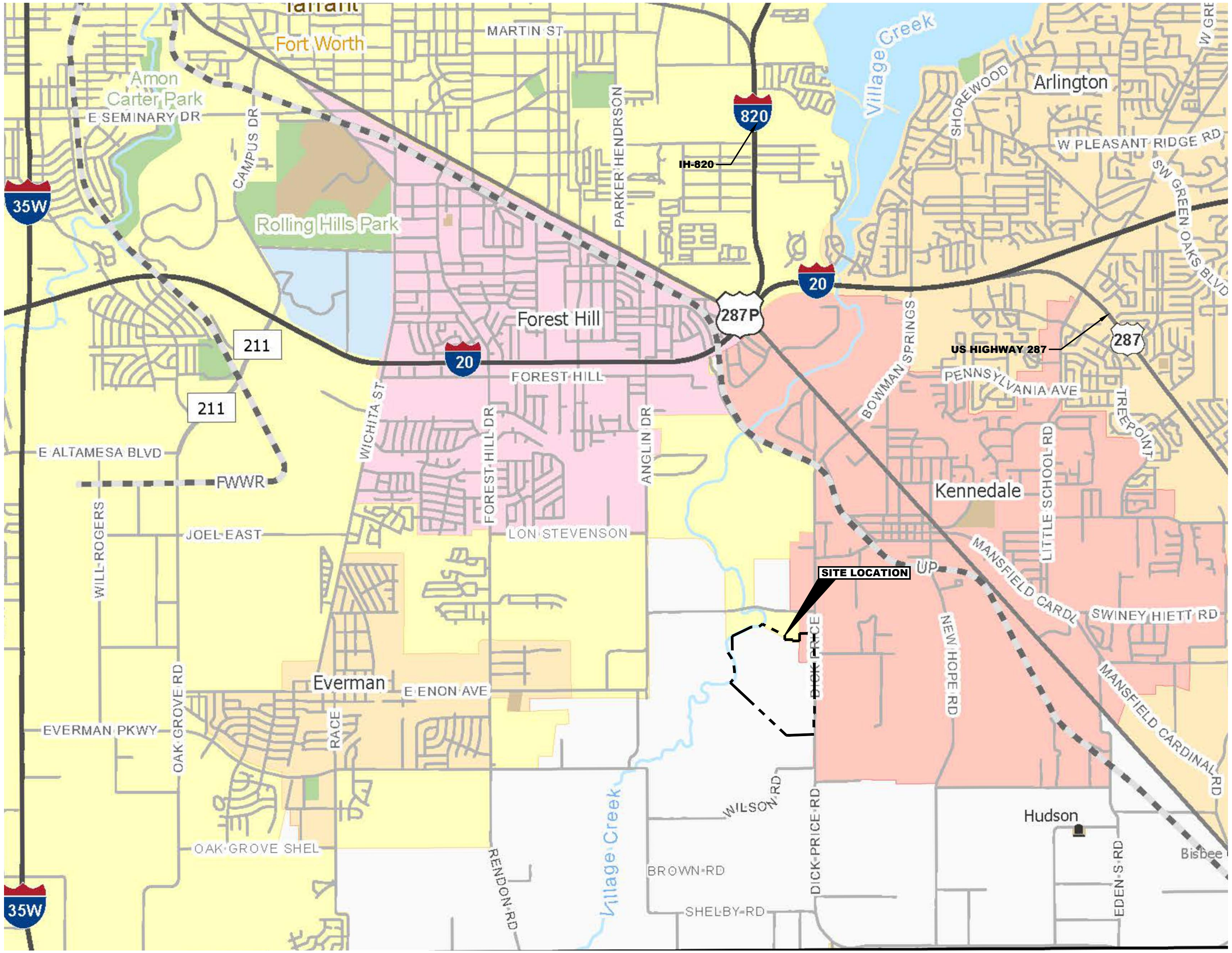
4.4 Summary

From the hydrological evaluations of the permitted and post-development conditions, the permitted 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 increased 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
Flow Rates, Drainage Areas, Hydrograph Time to Peak Values, Runoff Volumes, and Velocities for the 25-Year Design Storm Event

Stormwater Discharge Point[1]	Existing Permitted Condition[3]					Updated Permitted Condition[3]					Post-Development Condition				
	Flow Rate (cfs)	Drainage Area (acres)	Time to Peak (hrs)	Runoff Volume (ac-ft)	Velocity at Permit Boundary (fps)	Flow Rate (cfs)	Drainage Area (acres)	Time to Peak (hrs)	Runoff Volume (ac-ft)	Velocity at Permit Boundary[2] (fps)	Flow Rate (cfs)	Drainage Area (acres)	Time to Peak (hrs)	Runoff Volume (ac-ft)	Velocity at Permit Boundary[2] (fps)
DCPO1 (DO1)	9.9	1.74	--	0.56	--	21.5	5.11	13.20	2.3	2.51	21.5	5.11	13.20	2.3	2.51
DCPO2 (CO2)	26.7	4.85	--	1.56	--	20.3	4.56	13.15	2.0	2.50	20.3	4.56	13.15	2.0	2.50
DCPO3	--	--	--	--	--	83.7	17.39	13.15	7.7	7.60	83.7	17.39	13.15	7.7	7.60
DCPO4 (AO13)	39.8	6.81	--	2.20	--	30.5	6.25	13.15	2.8	6.16	30.5	6.25	13.15	2.8	6.16
DCP1	797.1	207.43	--	82.0	5.00	533.1	217.66	13.30	98.9	4.67	424.5	217.66	13.20	99.4	4.61

[1] Stormwater discharge points are shown on Figure 4.5. The volume shown is the total volume of runoff for the hydrograph duration. Discharge points in parentheses are the Basin ID's in the existing permitted condition.
[2] Runoff volume and velocity calculations are provided in Appendix IIIF-A and IIIF-E.
[3] Refer to Section 4.3.1.1 for a discussion on the existing permitted condition and updated permitted condition.



N

0 2000 4000
SCALE IN FEET

LEGEND

- PERMIT BOUNDARY
- Unincorporated Community
- County Seat
- Border Crossing
- Cemetery
- Cemetery (Inside City)
- Deep Draft Port
- Shallow Draft Port
- Railroad
- Dam
- River or Stream
- TXDOT District
- Lakes
- Education
- Military
- Airport Runway
- Airport
- Prison
- Parks and Other Public Land
- CITY OF FORT WORTH CITY LIMITS

NOTES:

1. ADAPTED FROM TEXAS DEPARTMENT OF TRANSPORTATION HIGHWAY MAY, 2018.

STATE OF TEXAS
CHARLES R. MARSH
105073
LICENSED
PROFESSIONAL ENGINEER
02/09/2023

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MAJOR PERMIT AMENDMENT SITE LOCATION MAP

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

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FIGURE 4.1

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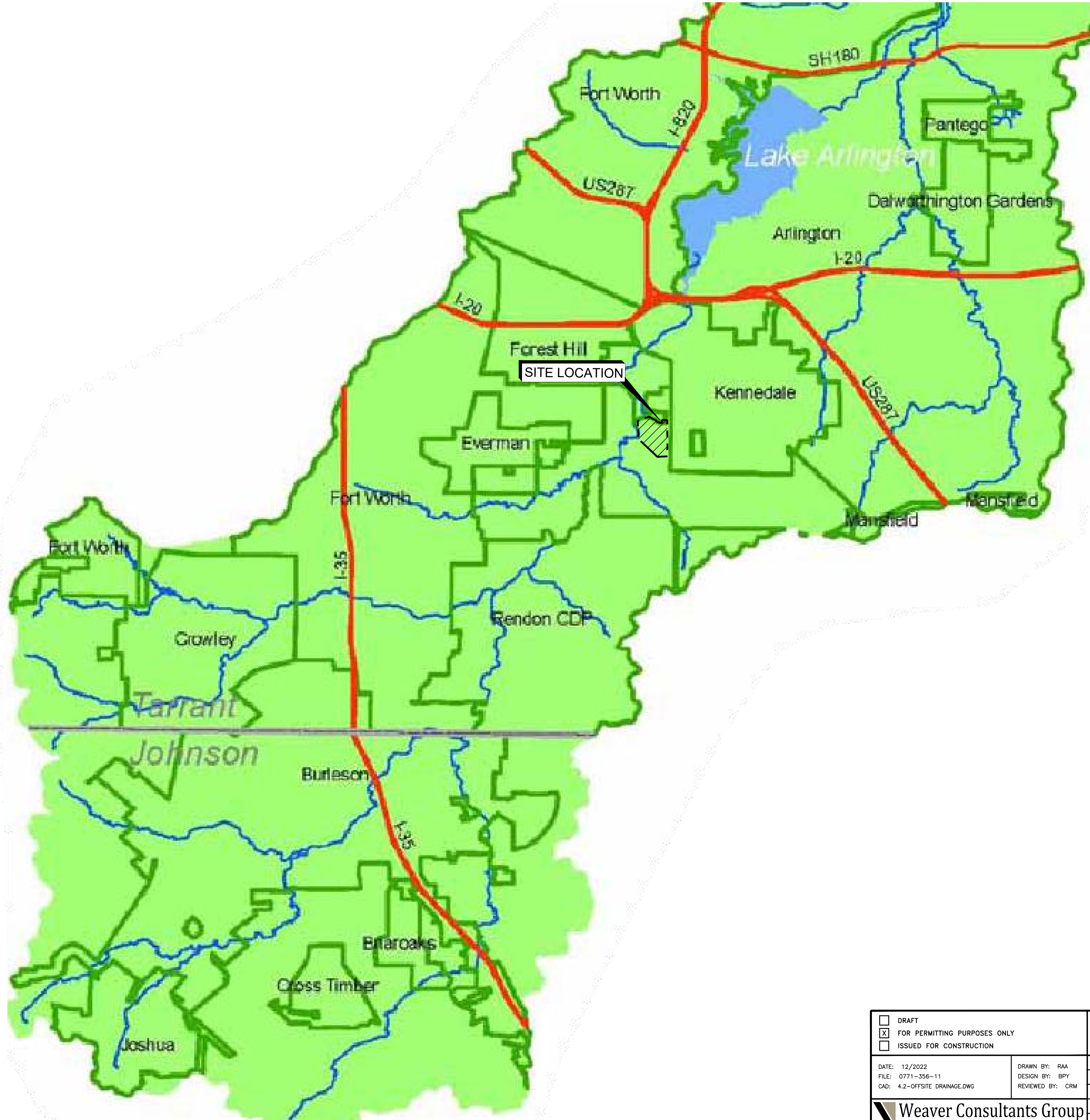
O:\0771\356\EXPANSION 2022\PART III\IIF\FIGURES\4.1-SITE LOCATION MAP.dwg, jwilson, 1:2



NOTES:

1. VILLAGE CREEK WATERSHED MAP REPRODUCED FROM CITY OF ARLINGTON, LAKE ARLINGTON MASTER PLAN, APRIL 2011.

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MAJOR PERMIT AMENDMENT
VILLAGE CREEK DRAINAGE BASIN

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

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FIGURE 4.2

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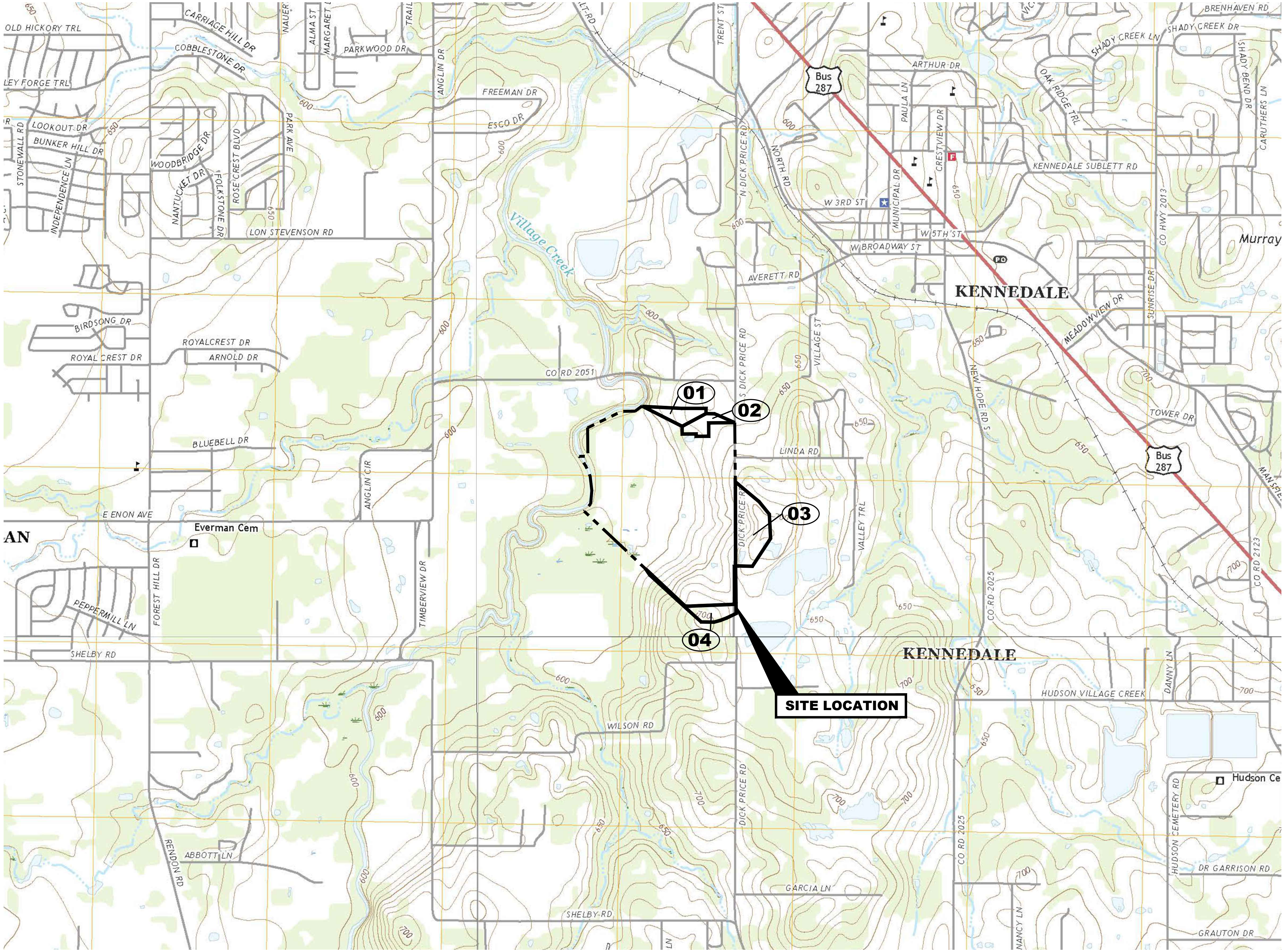
O:\0771\356\EXPANSION 2022\PART III\HIF\FIGURES\4.2-VILLAGE CREEK DRAINAGE BASIN.dwg, jwilson, 1:2



N

SCALE IN FEET: 0, 1000, 2000

LEGEND

- PERMIT BOUNDARY
- DRAINAGE AREA BOUNDARY
- 02 DRAINAGE AREA LABEL

ROAD CLASSIFICATION

- Expressway
- Secondary Hwy
- Ramp
- Local Connector
- Local Road
- 4WD
- Interstate Route
- US Route
- State Route

DRAINAGE AREA NO.	AREA (ACRES)
01	5.11
02	4.56
03	17.39
04	6.25
TOTAL	33.32

NOTES:

1. ADAPTED FROM USGS 7.5 MINUTE QUADRANGLE TOPOGRAPHIC MAP (FORT WORTH, KENNEDALE, BURLESON AND MANSFIELD, TEXAS, 2019).
2. DRAINAGE AREA DELINEATION WITHIN THE PERMIT BOUNDARY IS INCLUDED ON DRAWING IIIF–E–26.

FORT WORTH, TX 2019

KENNEDALE, TX 2019

BURLESON, TX 2019

MANSFIELD, TX 2019

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105073
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MAJOR PERMIT AMENDMENT
OFFSITE DRAINAGE AREA MAP

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

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FIGURE 4.3

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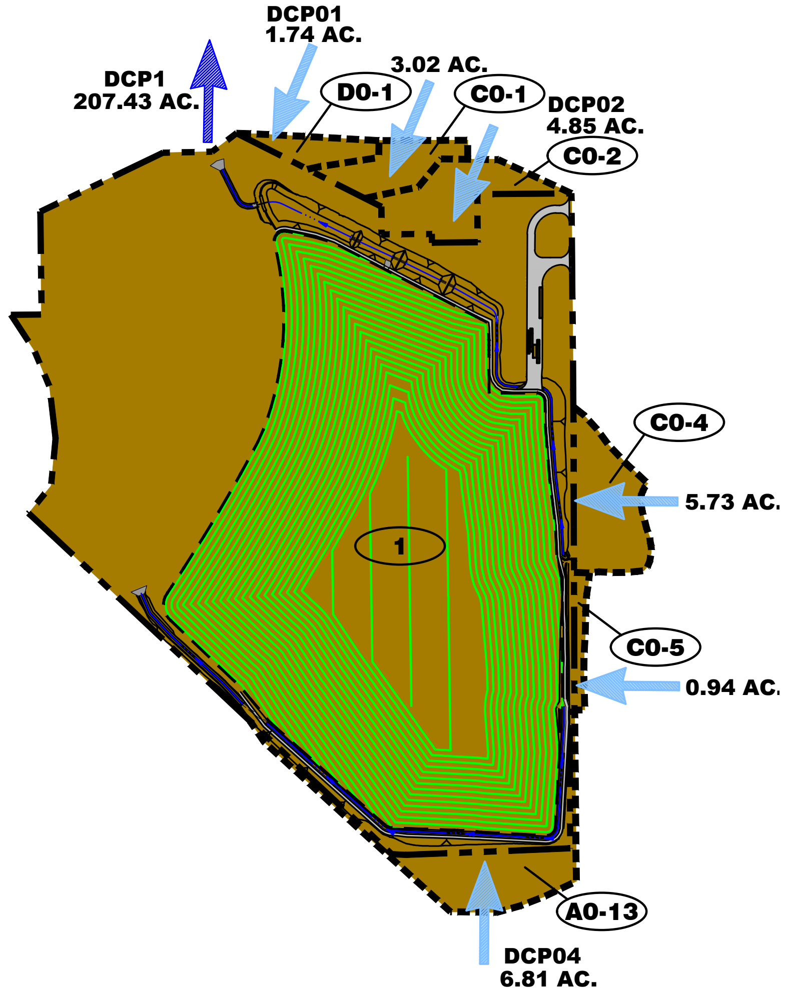
O:\0771\356\EXPANSION 2022\PART III\IIIF\FIGURES\4.3–OFFSITE DRAINAGE.dwg, jwilson, 1:2



EXISTING PERMITTED DRAINAGE PATTERS

DISCHARGE POINTS TOTAL = 207.43 ACRES

OFF-SITE AREAS

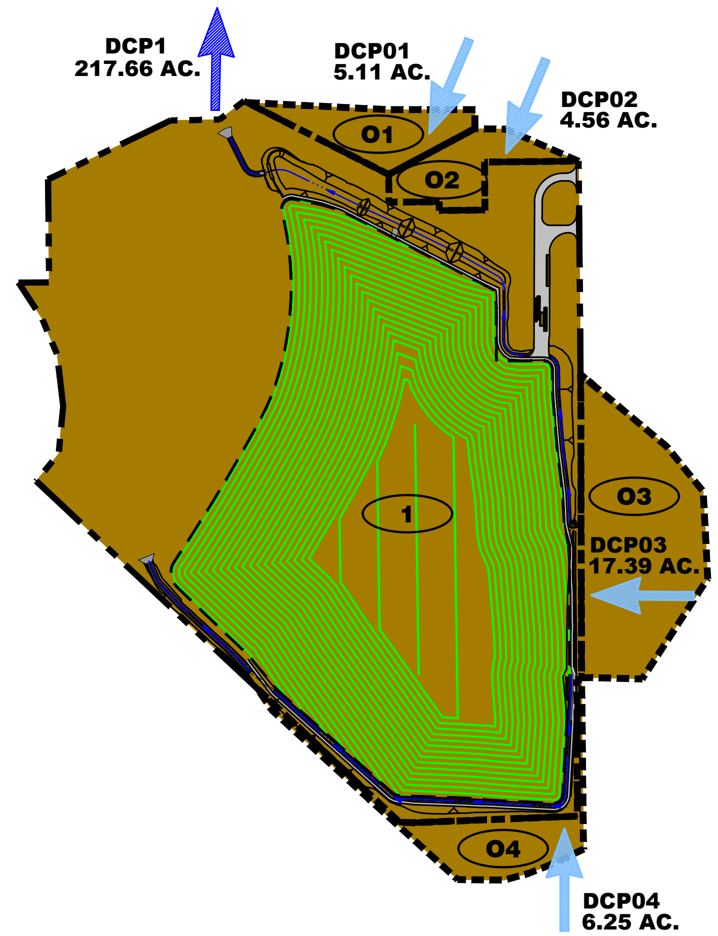
DO-1 = 1.74 ACRES
CO-1 = 3.02 ACRES
CO-2 = 4.85 ACRES
CO-4 = 5.73 ACRES
CO-5 = 0.94 ACRES
AO-13 = 6.81 ACRES

OFF-SITE TOTAL = 23.09 ACRES

ON-SITE AREAS

1 = APPROX. 184.34 ACRES

PERMIT BOUNDARY TOTAL = 184.34 ACRES
GRAND TOTAL = APPROX. 207.43 ACRES



UPDATED PERMITTED DRAINAGE PATTERS

DISCHARGE POINTS TOTAL = 217.66 ACRES

OFF-SITE AREAS

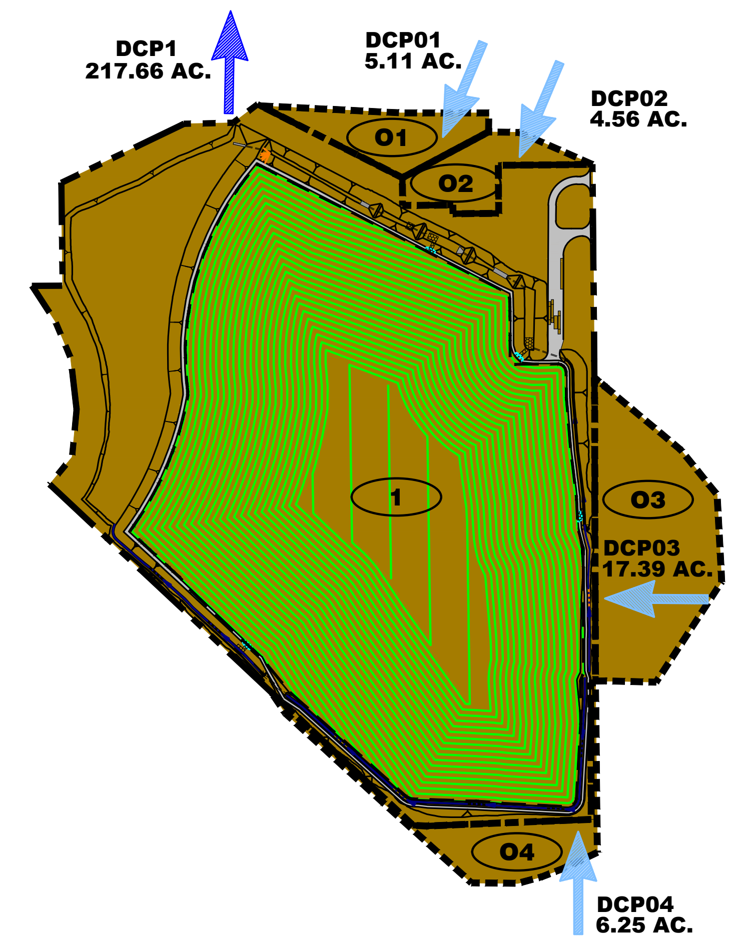
01 = 5.11 ACRES
02 = 4.56 ACRES
03 = 17.39 ACRES
04 = 6.25 ACRES

OFF-SITE TOTAL = 33.32 ACRES

ON-SITE AREAS

1 = APPROX. 184.34 ACRES

PERMIT BOUNDARY TOTAL = 184.34 ACRES
GRAND TOTAL = APPROX. 217.66 ACRES



POST-PROJECT DRAINAGE PATTERNS

DISCHARGE POINTS TOTAL = 391.76 ACRES

OFF-SITE AREAS

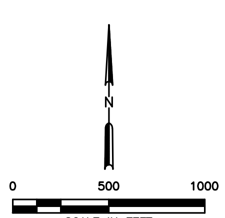
01 = 5.11 ACRES
02 = 4.56 ACRES
03 = 17.39 ACRES
04 = 6.25 ACRES

OFF-SITE TOTAL = 33.32 ACRES

ON-SITE AREAS

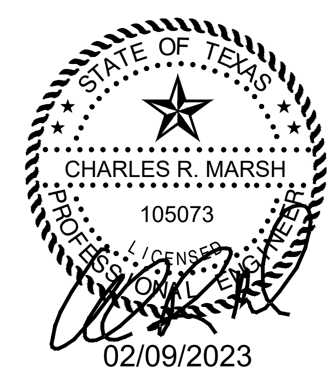
1 = APPROX. 184.34 ACRES

PERMIT BOUNDARY TOTAL = 184.34 ACRES
GRAND TOTAL = APPROX. 217.66 ACRES



LEGEND

- PROPERTY BOUNDARY
- LIMITS OF WASTE
- DRAINAGE DIVIDE
- PERMITTED/PROPOSED FINAL COVER CONTOUR
- 04 DRAINAGE AREA LABEL
- UPLAND DRAINAGE ENTERING SITE
- STORMWATER DISCHARGE POINT



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MAJOR PERMIT AMENDMENT
SITE DRAINAGE PATTERNS

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

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FIGURE 4.4

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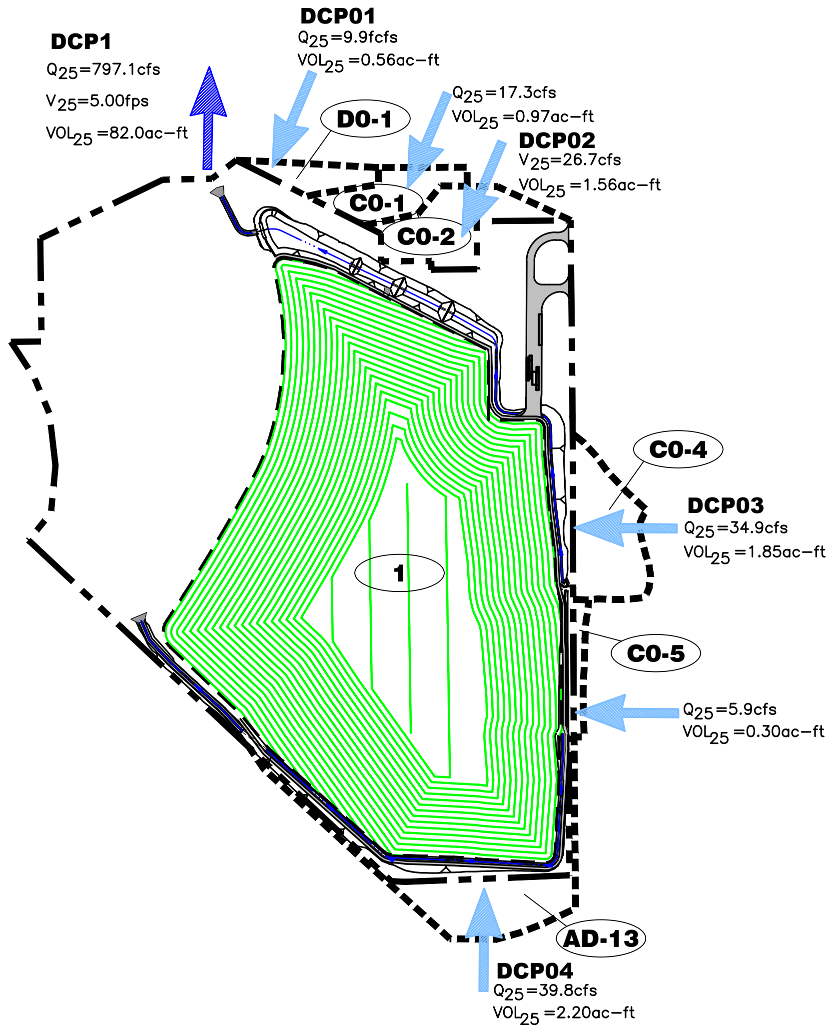
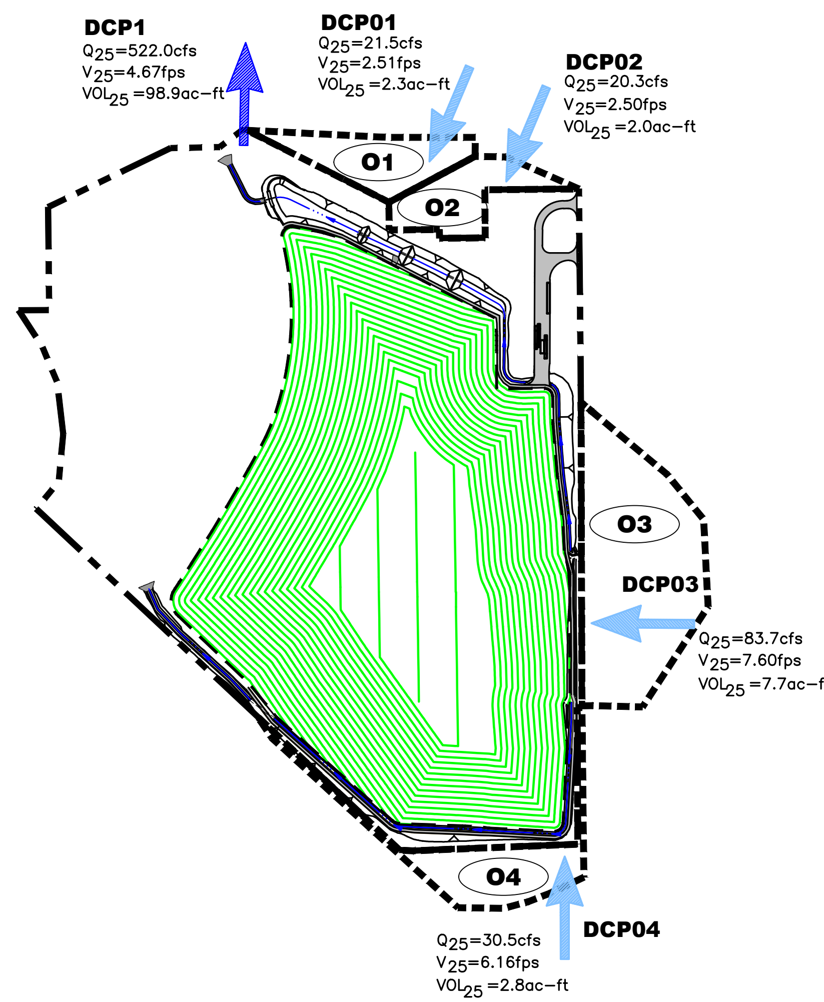
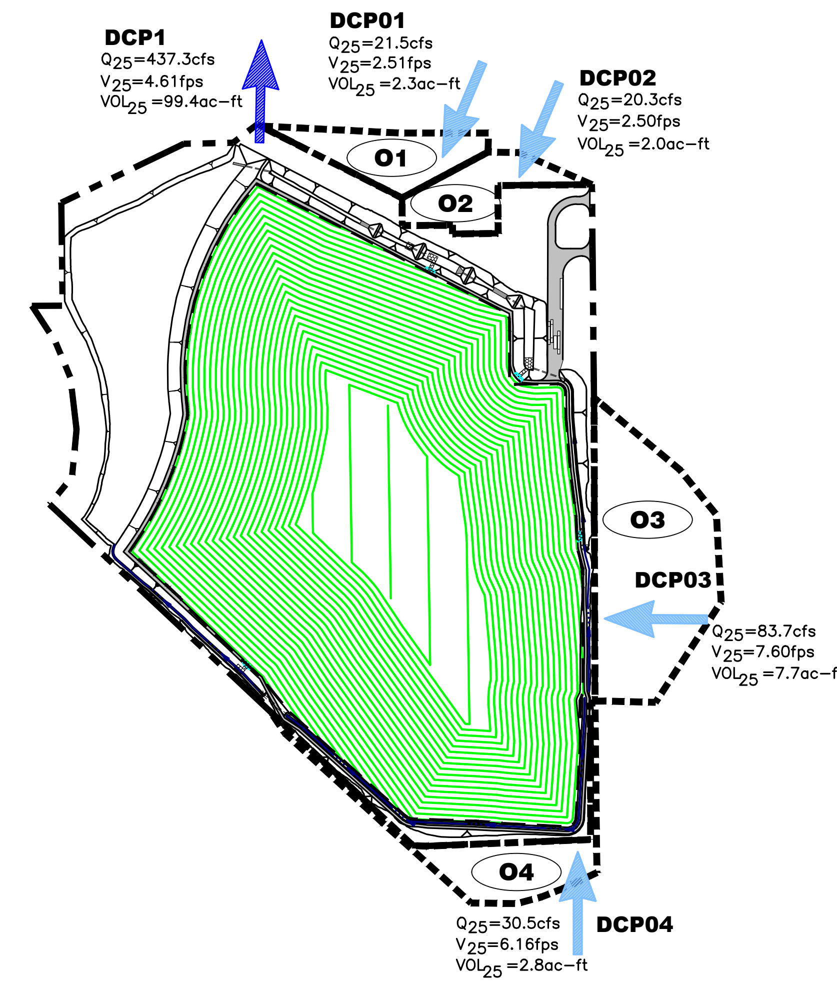


EXISTING PERMITTED DRAINAGE PATTERS



UPDATED PERMITTED DRAINAGE PATTERS



POST–PROJECT DRAINAGE PATTERNS

N

0 500 1000

SCALE IN FEET

LEGEND

PROPERTY BOUNDARY

LIMITS OF WASTE

DRAINAGE DIVIDE

PERMITTED/PROPOSED FINAL COVER CONTOUR

O4 DRAINAGE AREA LABEL

UPLAND DRAINAGE ENTERING SITE

STORMWATER DISCHARGE POINT

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105073
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MAJOR PERMIT AMENDMENT
SITE DRAINAGE PATTERNS
RUNON/RUNOFF

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

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FIGURE 4.5

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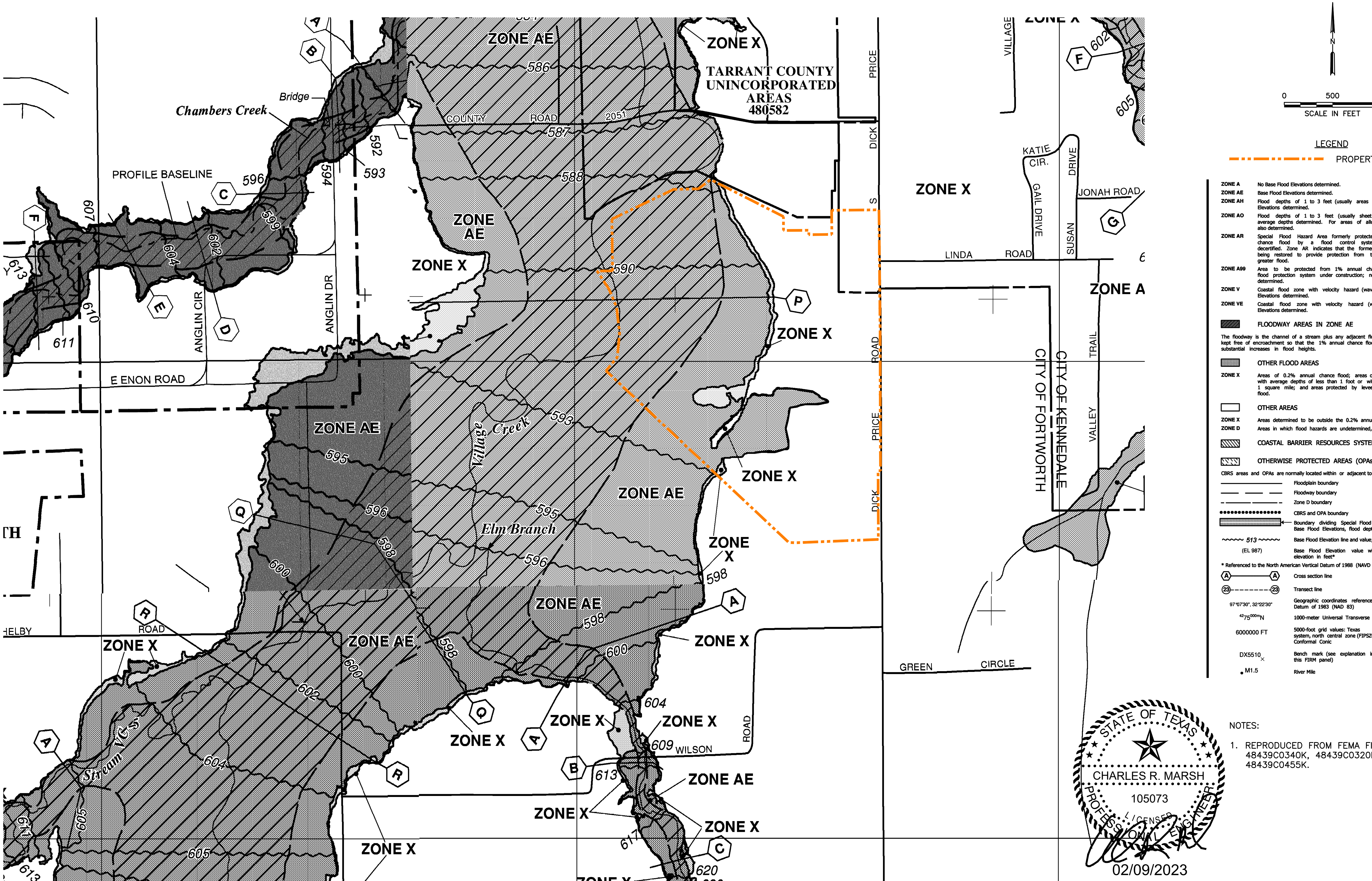
O:\0771\356\EXPANSION 2022\PART III\HF\FIGURES\4.5- RUNON RUNOFF.dwg, jwilson, 1:2



LEGEND

PROPERTY BOUNDARY

ZONE A	No Base Flood Elevations determined.
ZONE AE	Base Flood Elevations determined.
ZONE AH	Flood depths of 1 to 3 feet (usually areas of ponding); Base Flood Elevations determined.
ZONE AO	Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain); average depths determined. For areas of alluvial fan flooding, velocities also determined.
ZONE AR	Special Flood Hazard Area formerly protected from the 1% annual chance flood by a flood control system that was subsequently decertified. Zone AR indicates that the former flood control system is being restored to provide protection from the 1% annual chance or greater flood.
ZONE A99	Area to be protected from 1% annual chance flood by a Federal flood protection system under construction; no Base Flood Elevations determined.
ZONE V	Coastal flood zone with velocity hazard (wave action); no Base Flood Elevations determined.
ZONE VE	Coastal flood zone with velocity hazard (wave action); Base Flood Elevations determined.

FLOODWAY AREAS IN ZONE AE

The floodway is the channel of a stream plus any adjacent floodplain areas that must be kept free of encroachment so that the 1% annual chance flood can be carried without substantial increases in flood heights.

OTHER FLOOD AREAS

ZONE X — Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood.

OTHER AREAS

ZONE X — Areas determined to be outside the 0.2% annual chance floodplain.
ZONE D — Areas in which flood hazards are undetermined, but possible.

COASTAL BARRIER RESOURCES SYSTEM (CBRS) AREAS

OTHERWISE PROTECTED AREAS (OPAs)

CBRS areas and OPAs are normally located within or adjacent to Special Flood Hazard Areas.

Floodplain boundary
Floodway boundary
Zone D boundary
CBRS and OPA boundary
Boundary dividing Special Flood Hazard Areas of different Base Flood Elevations, flood depths or flood velocities.

~~ 513 ~~~~	Base Flood Elevation line and value; elevation in feet*
(EL 987)	Base Flood Elevation value where uniform within zone; elevation in feet*

* Referenced to the North American Vertical Datum of 1988 (NAVD 88)

(A)———(A)	Cross section line
(23)- - - - -(23)	Transect line
97°07'30", 32°22'30"	Geographic coordinates referenced to the North American Datum of 1983 (NAD 83)
$^{42}75^{000m}$N	1000-meter Universal Transverse Mercator grid ticks, zone 14
6000000 FT	5000-foot grid values: Texas State Plane coordinate system, north central zone (FIPSZONE 4202), Lambert Conformal Conic
DX5510 ×	Bench mark (see explanation in Notes to Users section of this FIRM panel)
• M1.5	River Mile

STATE OF TEXAS
CHARLES R. MARSH
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NOTES:

1. REPRODUCED FROM FEMA FIRM PANELS 48439C0340K, 48439C0320L, 48439C0435K, 48439C0455K.

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NO.	DATE	DESCRIPTION

MAJOR PERMIT AMENDMENT
FLOOD INSURANCE RATE MAP (FIRM)

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

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FIGURE 4.6

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O:\0771\356\EXPANSION 2022\PART III\IIIF\FIGURES\4.6-FIRM.dwg, jwilson, 1:2

*

DRAWINGS

IIIF.1 – Drainage Structure Plan

IIIF.2 – Post-Development Drainage Area Plan

IIIF.3 – Post-Development Offsite Drainage Areas

IIIF.4 – Perimeter Drainage Plan

IIIF.5 – Perimeter Channel Profiles

IIIF.6 – Perimeter Channel Profiles

IIIF.7 – Drainage Details

IIIF.8 – Drainage Details

IIIF.9 – Drainage Details

IIIF.10 – Drainage Details

IIIF.11. – Drainage Details

IIIF.12. – Drainage Details

IIIF.13 – Ponds P1 through P5 Plan

N

0 200 400

SCALE IN FEET

LEGEND

- PERMIT BOUNDARY
- LIMIT OF WASTE
- 630 — EXISTING CONTOUR
- N 6915000 — STATE PLANE COORDINATE
- 560 — FINAL COVER CONTOUR
- DRAINAGE LETDOWN
- DRAINAGE SWALE
- PROPOSED PERIMETER CHANNEL
- GABIONS
- MW-7 — EXISTING/PERMITTED GROUNDWATER MONITORING WELL
- MW-9A — APPROVED GROUNDWATER MONITORING WELL
- MW-4 — PERMITTED GROUNDWATER MONITORING WELL (TO BE ABANDONED)
- GMP-4B — PERMITTED GAS MONITORING PROBE

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS PROVIDED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN 02-17-2022.
2. REFER TO APPENDIX IIIF–SURFACE WATER DRAINAGE PLAN FOR DRAINAGE DESIGN INFORMATION.
3. MAXIMUM FINAL COVER ELEVATION 860 FT-MSL.

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CAD: IIIF.1-DRAINAGE PLAN.DWG

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DESIGN BY: BPY
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NO.	DATE	DESCRIPTION

MAJOR PERMIT AMENDMENT FINAL COVER PLAN

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

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DRAWING IIIF.1

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0 200 400
SCALE IN FEET

LEGEND

	PERMIT BOUNDARY
	LIMIT OF WASTE
630	EXISTING CONTOUR
N 6915000	STATE PLANE COORDINATE
560	FINAL COVER CONTOUR
	DRAINAGE LETDOWN
	DRAINAGE SWALE
	PROPOSED PERIMETER CHANNEL
	GABIONS
MW-7	EXISTING/PERMITTED GROUNDWATER MONITORING WELL
MW-9A	APPROVED GROUNDWATER MONITORING WELL
MW-4	PERMITTED GROUNDWATER MONITORING WELL (TO BE ABANDONED)
GMP-4B	PERMITTED GAS MONITORING PROBE
	DRAINAGE DIVIDE
O3	DRAINAGE AREA DESIGNATION

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS PROVIDED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN 02-17-2022.
2. REFER TO APPENDIX IIIF-SURFACE WATER DRAINAGE PLAN FOR DRAINAGE DESIGN INFORMATION.
3. MAXIMUM FINAL COVER ELEVATION 860 FT-MSL.

DRAINAGE AREA NO.	AREA (ACRES)	DRAINAGE AREA NO.	AREA (ACRES)
DA1	35.42	P1	2.16
DA2	32.15	P2	3.77
DA3	8.98	P3	3.37
DA4	33.08	CH1	2.52
S1	9.92	CH2	3.31
S2	35.42	CH3	3.53
S3	2.03	CH4	4.80
O1	5.11	CH5	3.90
O2	4.56		
O3	17.39		
O4	6.25		

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MAJOR PERMIT AMENDMENT POST-DEVELOPMENT DRAINAGE AREA PLAN

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

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DRAWING IIIF.2

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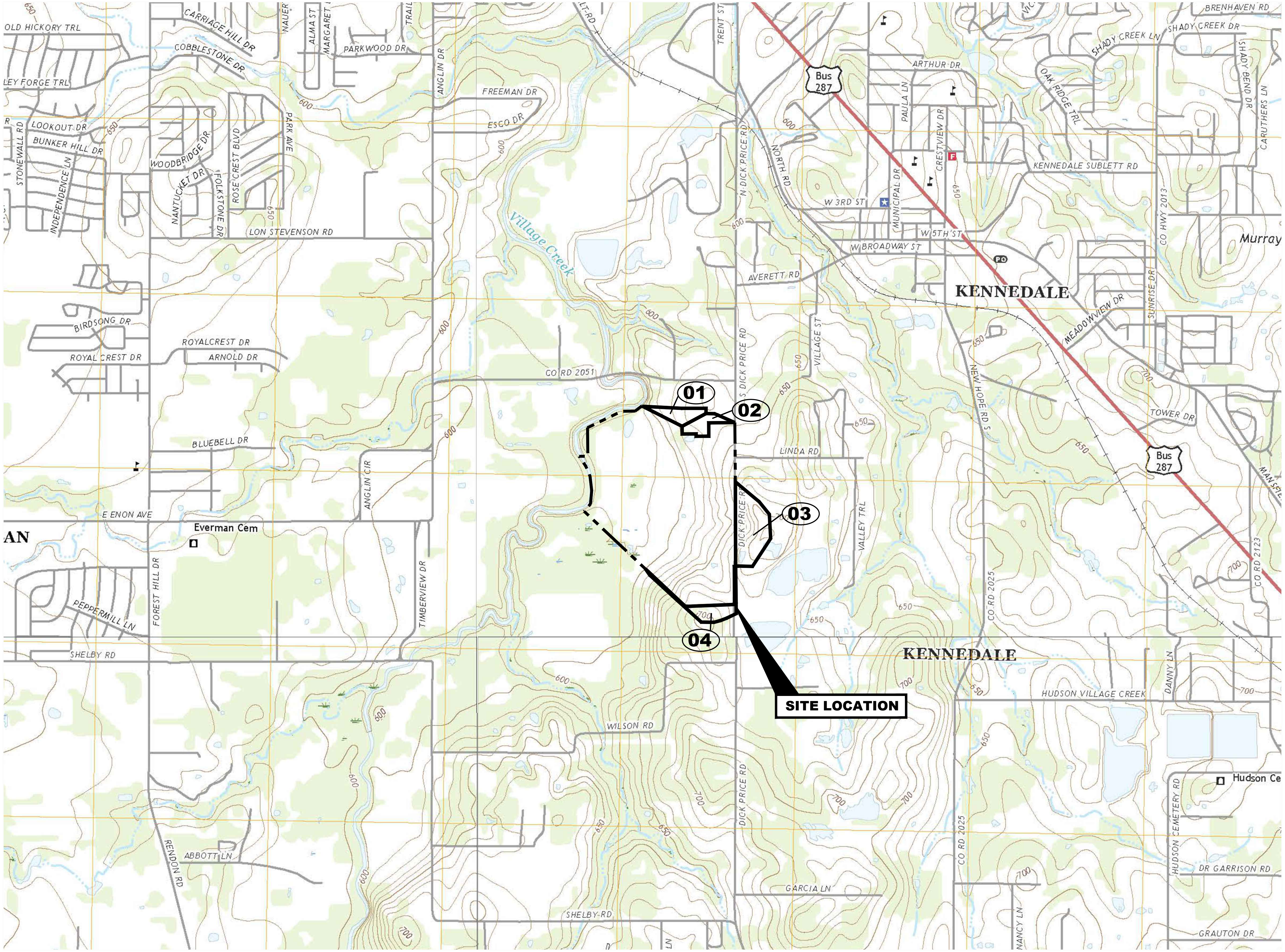

N

0 1000 2000
SCALE IN FEET

LEGEND

PERMIT BOUNDARY

DRAINAGE AREA BOUNDARY

02 DRAINAGE AREA LABEL

ROAD CLASSIFICATION

Expressway — Local Connector
Secondary Hwy — Local Road
Ramp — 4WD
Interstate Route — US Route — State Route

DRAINAGE AREA NO.	AREA (ACRES)
01	5.11
02	4.56
03	17.39
04	6.25
TOTAL	33.32

NOTES:

1. ADAPTED FROM USGS 7.5 MINUTE QUADRANGLE TOPOGRAPHIC MAP (FORT WORTH, KENNEDALE, BURLESON AND MANSFIELD, TEXAS, 2019).
2. DRAINAGE AREA DELINEATION WITHIN THE PERMIT BOUNDARY IS INCLUDED ON DRAWING IIIF–E–26.

FORT WORTH, TX 2019	KENNEDALE, TX 2019
BURLESON, TX 2019	MANSFIELD, TX 2019

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CAD: IIIF.3-OFFSITE DRAINAGE.DWG

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TBPE REGISTRATION NO. F-3727

MAJOR PERMIT AMENDMENT POST–DEVELOPMENT OFFSITE DRAINAGE AREAS

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

WWW.WCGRP.COM

DRAWING IIIF.3

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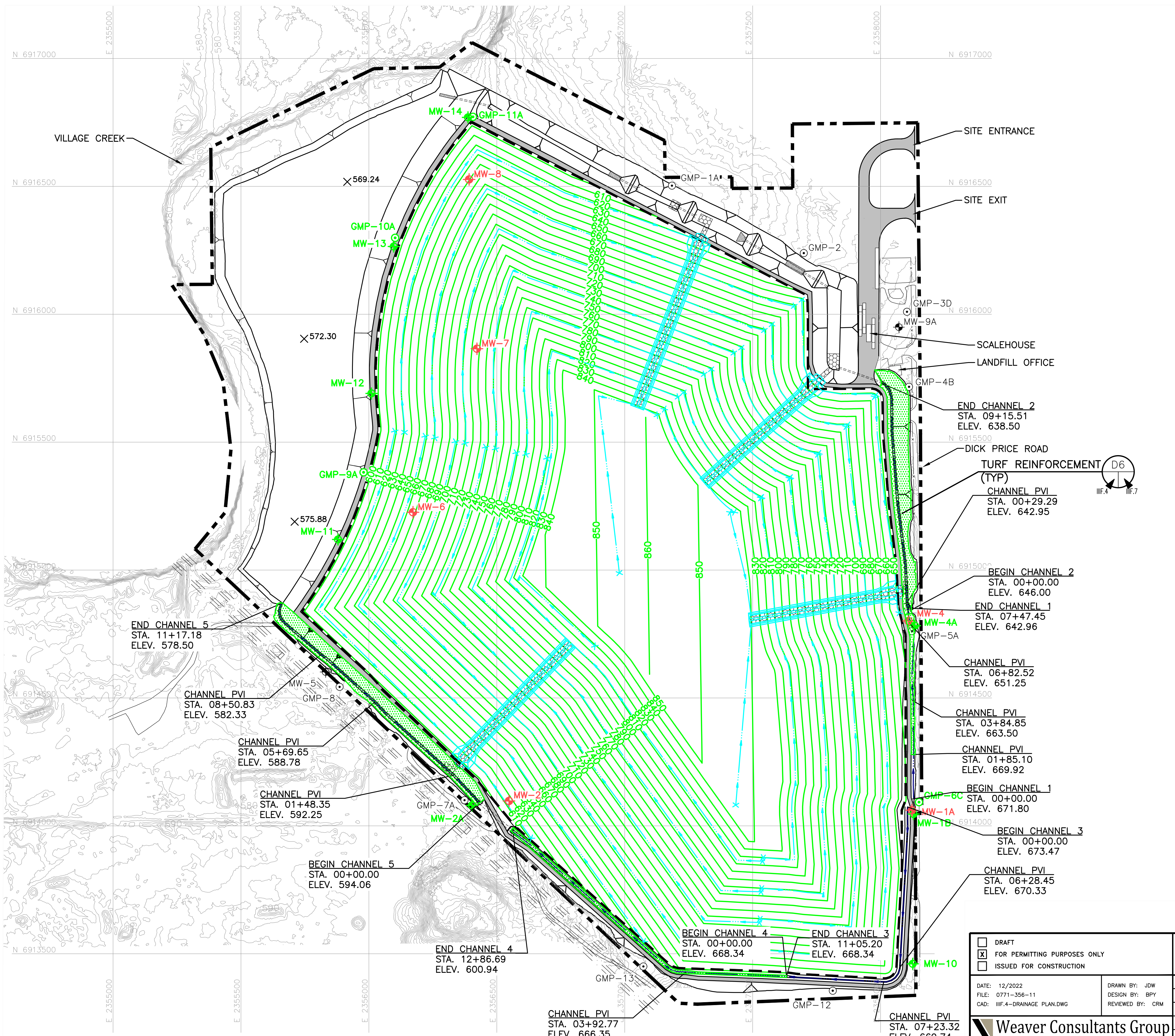
O:\0771\356\EXPANSION 2022\PART III\IIIF\IIIF.3-OFFSITE DRAINAGE.dwg, jwilson, 1:2



N

0 200 400
SCALE IN FEET

LEGEND

- PERMIT BOUNDARY
- LIMIT OF WASTE
- 630 EXISTING CONTOUR
- N 6915000 STATE PLANE COORDINATE
- 560 FINAL COVER CONTOUR
- DRAINAGE LETDOWN
- DRAINAGE SWALE
- PROPOSED PERIMETER CHANNEL
- GABIONS
- TURF REINFORCEMENT MAT
- MW-7 EXISTING/PERMITTED GROUNDWATER MONITORING WELL
- MW-9A APPROVED GROUNDWATER MONITORING WELL
- MW-4 PERMITTED GROUNDWATER MONITORING WELL (TO BE ABANDONED)
- GMP-4B PERMITTED GAS MONITORING PROBE

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS PROVIDED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN 02-17-2022.
2. REFER TO APPENDIX IIIF–SURFACE WATER DRAINAGE PLAN FOR DRAINAGE DESIGN INFORMATION.
3. MAXIMUM FINAL COVER ELEVATION 860 FT-MSL.

STATE OF TEXAS
CHARLES R. MARSH
105073
LICENSED PROFESSIONAL ENGINEER
02/09/2023

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DATE: 12/2022
FILE: 0771-356-11
CAD: IIIF.4-DRAINAGE PLAN.DWG

DRAWN BY: JDW
DESIGN BY: BPY
REVIEWED BY: CRM

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

PREPARED FOR
TEXAS REGIONAL LANDFILL COMPANY, LP

REVISIONS

NO.	DATE	DESCRIPTION

MAJOR PERMIT AMENDMENT PERIMETER CHANNEL PLAN

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

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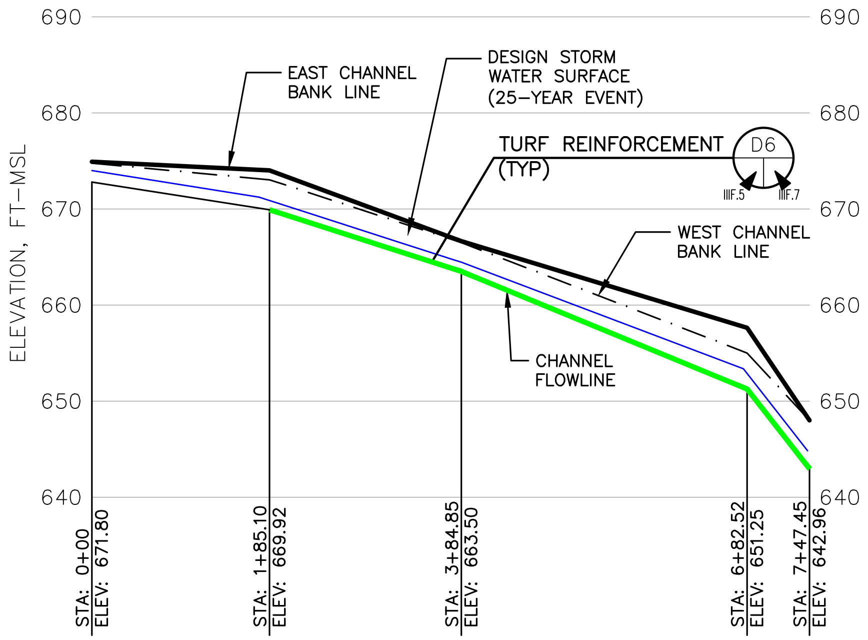
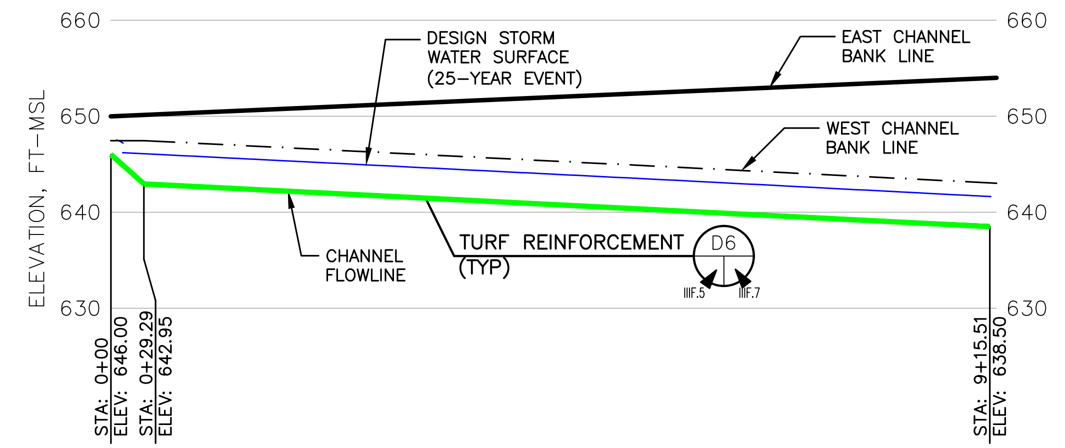


CHANNEL "1" (1 — IIIF.4 / IIIF.5)

0 100 200 SCALE IN FEET (HORIZONTAL)

0 10 20 SCALE IN FEET (VERTICAL)



CHANNEL "2" (2 — IIIF.4 / IIIF.5)

0 100 200 SCALE IN FEET (HORIZONTAL)

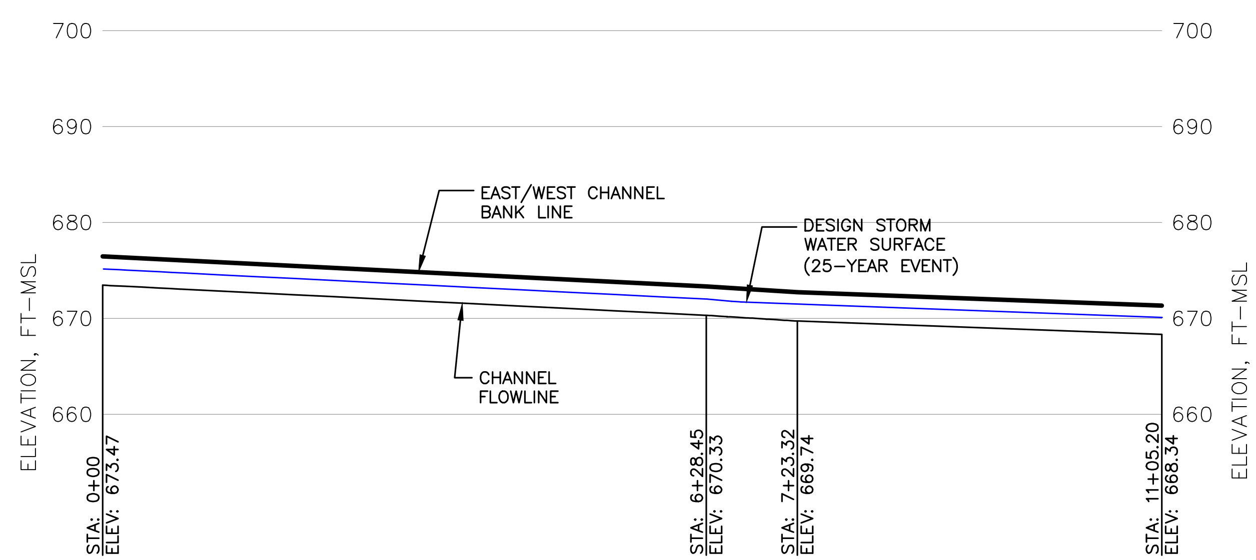
0 10 20 SCALE IN FEET (VERTICAL)

25-YEAR CHANNEL "1" INFORMATION						
CHANNEL STATION FROM	CHANNEL STATION TO	BOTTOM WIDTH (FT)	PEAK INFLOW (CFS)	SLOPE (%)	FLOW DEPTH (FT.)	VELOCITY (FT/S)
0+00	1+85.10	0	14.6	1.0	1.20	3.39
1+85.10	3+84.85	0	14.6	3.2	0.96	5.26
3+84.85	6+82.52	0	14.6	4.8	0.89	6.11
6+82.52	7+47.45	0	14.6	6.3	0.85	6.76

NOTE: NORMAL DEPTH CALCULATION DOES NOT ACCOUNT FOR BACK WATER WHICH WILL INCREASE FLOW DEPTH (SEE PROFILE) AND DECREASE VELOCITY.

25-YEAR CHANNEL "2" INFORMATION						
CHANNEL STATION FROM	CHANNEL STATION TO	BOTTOM WIDTH (FT)	PEAK INFLOW (CFS)	SLOPE (%)	FLOW DEPTH (FT.)	VELOCITY (FT/S)
0+00	0+29.29	8	294.1	12.0	1.40	17.28
0+29.29	9+15.51	8	294.1	0.5	3.11	5.46

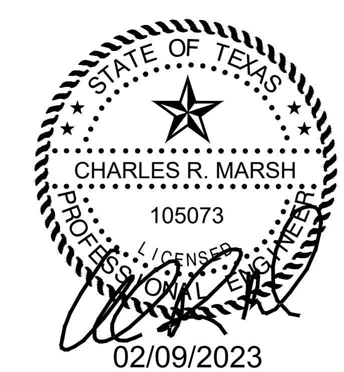
NOTE: NORMAL DEPTH CALCULATION DOES NOT ACCOUNT FOR BACK WATER WHICH WILL INCREASE FLOW DEPTH (SEE PROFILE) AND DECREASE VELOCITY.



CHANNEL "3" (3 — IIIF.4 / IIIF.5)

0 100 200 SCALE IN FEET (HORIZONTAL)

0 10 20 SCALE IN FEET (VERTICAL)



25-YEAR CHANNEL "3" INFORMATION						
CHANNEL STATION FROM	CHANNEL STATION TO	BOTTOM WIDTH (FT)	PEAK INFLOW (CFS)	SLOPE (%)	FLOW DEPTH (FT.)	VELOCITY (FT/S)
0+00	6+28.45	3	44.9	0.5	1.64	3.44
6+28.45	7+23.32	3	44.9	0.6	1.58	3.68
7+23.32	11+05.20	3	44.9	0.4	1.73	3.17

NOTE: NORMAL DEPTH CALCULATION DOES NOT ACCOUNT FOR BACK WATER WHICH WILL INCREASE FLOW DEPTH (SEE PROFILE) AND DECREASE VELOCITY.

NOTES:

1. REFER TO DRAWING IIIF.4 FOR PROFILE LOCATIONS.
2. EXISTING CONTOURS AND ELEVATIONS PROVIDED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN ON 01-08-2021.
3. HYDRAULIC CALCULATIONS INCLUDED IN APPENDIX IIIF-B.
4. GABIONS SHALL BE USED FOR VELOCITIES OF 20 FT/SEC OR HIGHER.
5. CULVERT CALCULATIONS INCLUDED IN APPENDIX IIIF-B.

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DATE: 12/2022 DRAWN BY: JDW
FILE: 0771-356-11 DESIGN BY: BPY
CAD: IIIF.5-PERIMETER CHANNEL PROFILE.DWG REVIEWED BY: CRM

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TBPE REGISTRATION NO. F-3727

PREPARED FOR
TEXAS REGIONAL LANDFILL COMPANY, LP

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MAJOR PERMIT AMENDMENT
PERIMETER CHANNEL PROFILES

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

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DRAWING IIIF.5

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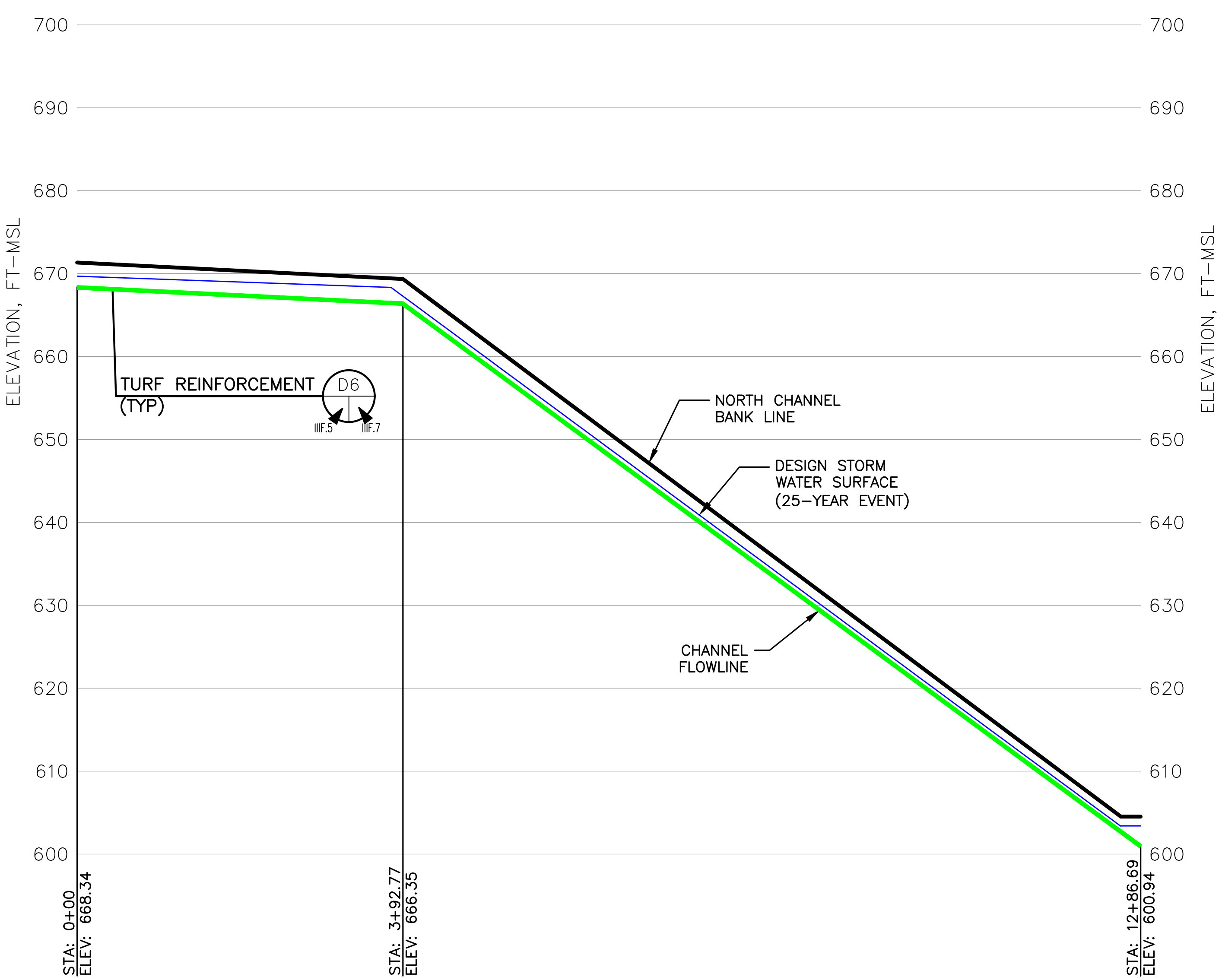
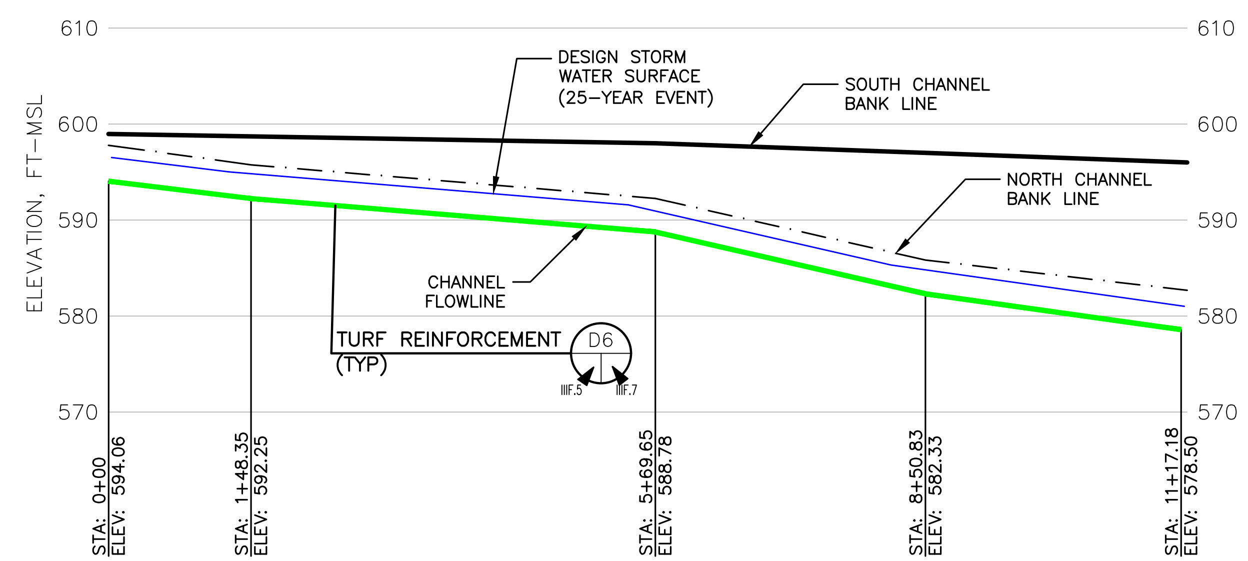


CHANNEL "4"

4 IIIF.4 IIIF.6

0 100 200 SCALE IN FEET (HORIZONTAL)

0 10 20 SCALE IN FEET (VERTICAL)



CHANNEL "5"

5 IIIF.4 IIIF.6

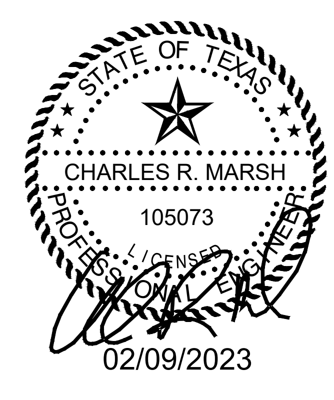
0 100 200 SCALE IN FEET (HORIZONTAL)

0 10 20 SCALE IN FEET (VERTICAL)

25–YEAR CHANNEL "4" INFORMATION

CHANNEL STATION FROM	CHANNEL STATION TO	BOTTOM WIDTH (FT)	PEAK INFLOW (CFS)	SLOPE (%)	FLOW DEPTH (FT.)	VELOCITY (FT/S)
0+00	3+92.77	8	69.6	0.4	1.60	3.40
3+92.77	12+86.69	13	69.6	7.4	0.56	8.58

NOTE: NORMAL DEPTH CALCULATION DOES NOT ACCOUNT FOR BACK WATER WHICH WILL INCREASE FLOW DEPTH (SEE PROFILE) AND DECREASE VELOCITY.



25–YEAR CHANNEL "5" INFORMATION

CHANNEL STATION FROM	CHANNEL STATION TO	BOTTOM WIDTH (FT)	PEAK INFLOW (CFS)	SLOPE (%)	FLOW DEPTH (FT.)	VELOCITY (FT/S)
0+00	1+48.35	8	300.8	1.20	2.46	7.35
1+48.35	5+69.65	8	300.8	0.80	2.56	6.01
5+69.65	8+50.83	8	300.8	2.30	2.10	9.33
8+50.83	11+17.18	8	300.8	1.40	2.45	8.02

NOTE: NORMAL DEPTH CALCULATION DOES NOT ACCOUNT FOR BACK WATER WHICH WILL INCREASE FLOW DEPTH (SEE PROFILE) AND DECREASE VELOCITY.

NOTES:

1. REFER TO DRAWING IIIF.4 FOR PROFILE LOCATIONS.
2. EXISTING CONTOURS AND ELEVATIONS PROVIDED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN ON 01–08–2021.
3. HYDRAULIC CALCULATIONS INCLUDED IN APPENDIX IIIF–B.
4. GABIONS SHALL BE USED FOR VELOCITIES OF 20 FT/SEC OR HIGHER.
5. CULVERT CALCULATIONS INCLUDED IN APPENDIX IIIF–B.

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CAD: IIIF.6–PERIMETER CHANNEL PROFILE.DWG

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DESIGN BY: BPY
REVIEWED BY: CRM

PREPARED FOR
TEXAS REGIONAL LANDFILL COMPANY, LP

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Weaver Consultants Group
TBPE REGISTRATION NO. F–3727

MAJOR PERMIT AMENDMENT
PERIMETER CHANNEL PROFILES

TURKEY CREEK LANDFILL
JOHNSON COUNTY, TEXAS

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DRAWING IIIF.6

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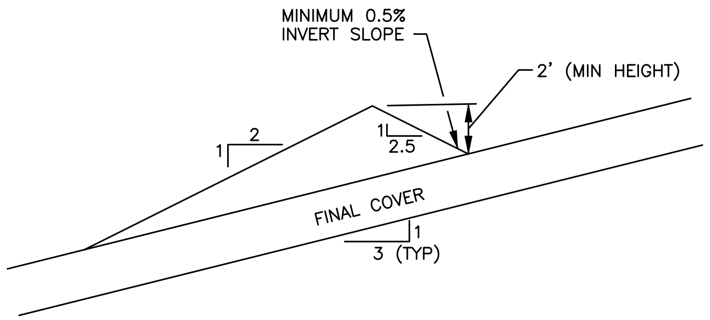
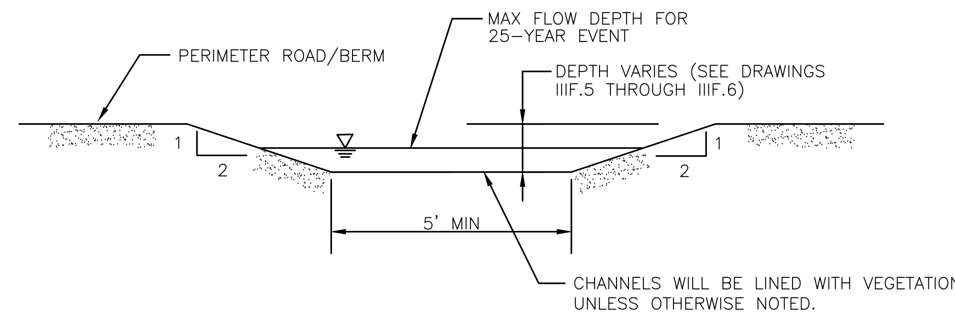


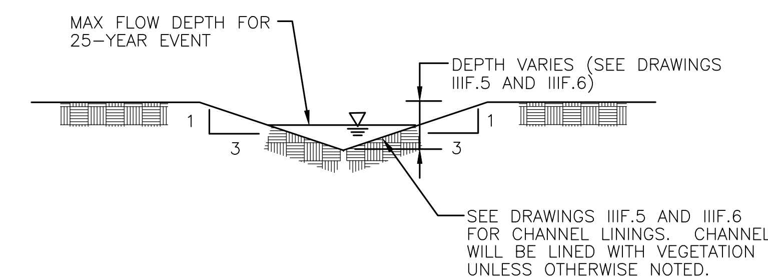
SWALE (TYP) D1 IIIF.1 IIIF.7

0 4 8
SCALE IN FEET



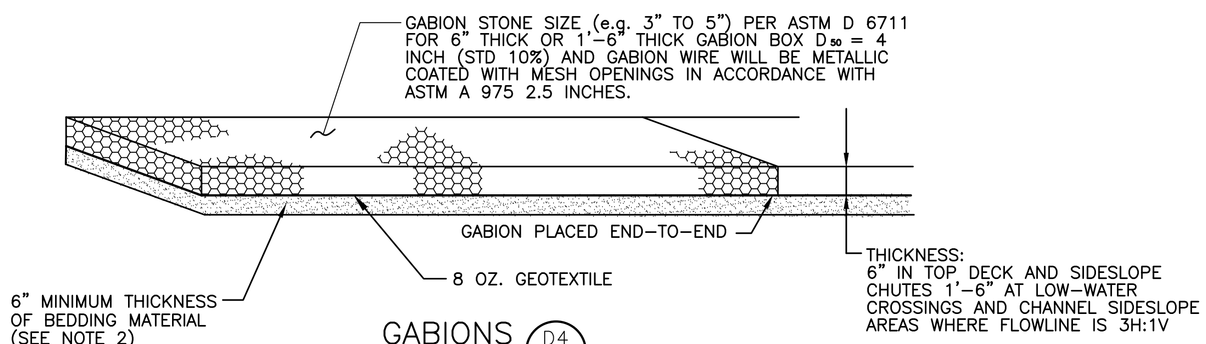
TRAPEZOIDAL CHANNEL (TYP) D2 IIIF.1,IIIF.8 IIIF.7

0 2 4
SCALE IN FEET



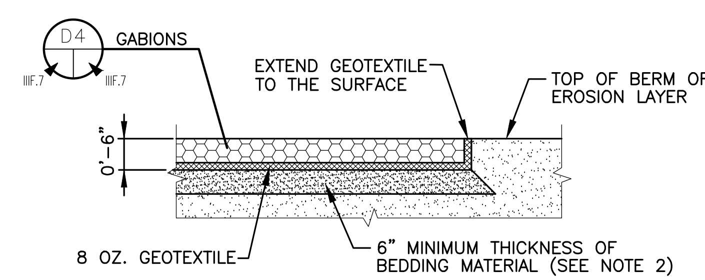
"V" CHANNEL (TYP) D3 IIIF.1 IIIF.7

0 2 4
SCALE IN FEET



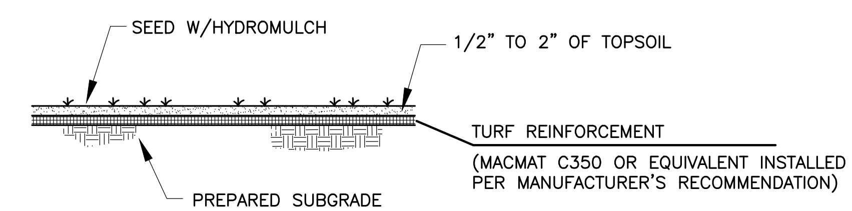
GABIONS D4 IIIF.1 IIIF.7

0 2 4
SCALE IN FEET



GABION TERMINATION D5 IIIF.7 IIIF.7

0 2 4
SCALE IN FEET



TURF REINFORCEMENT D6 IIIF.4,IIIF.5,IIIF.6 IIIF.7
NOT TO SCALE
(SEE NOTE 2)

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.

STATE OF TEXAS
CHARLES R. MARSH
105073
LICENSED
PROFESSIONAL ENGINEER
02/09/2023

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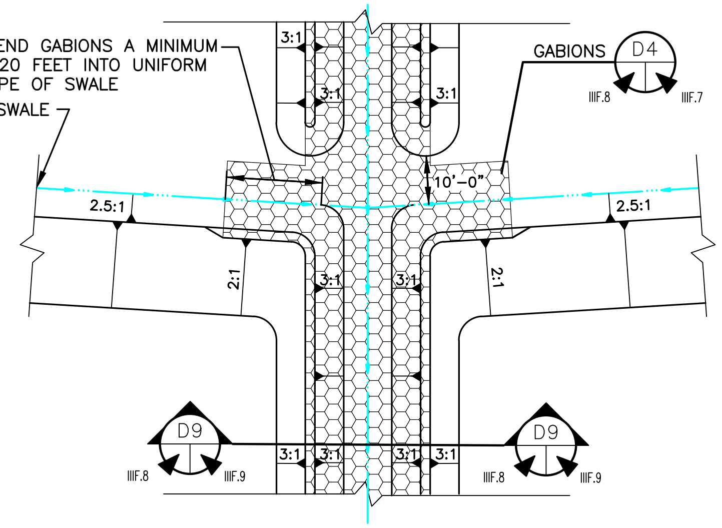
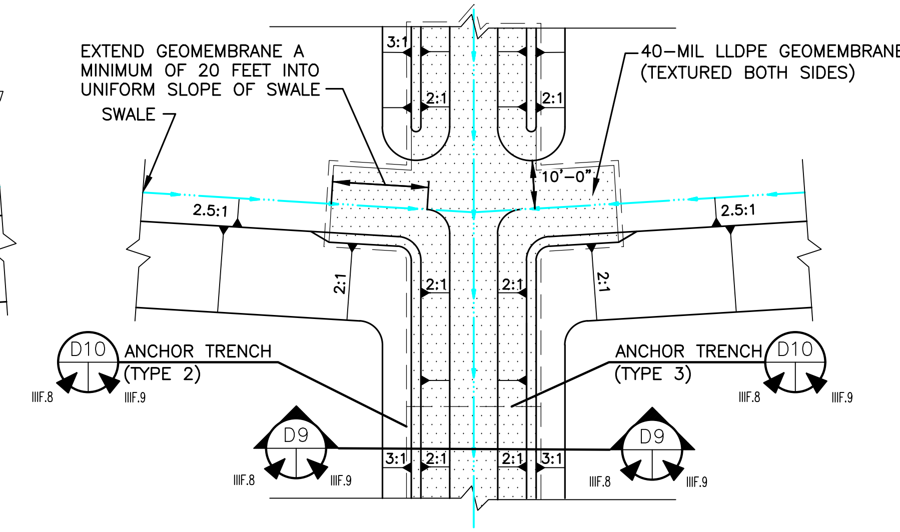


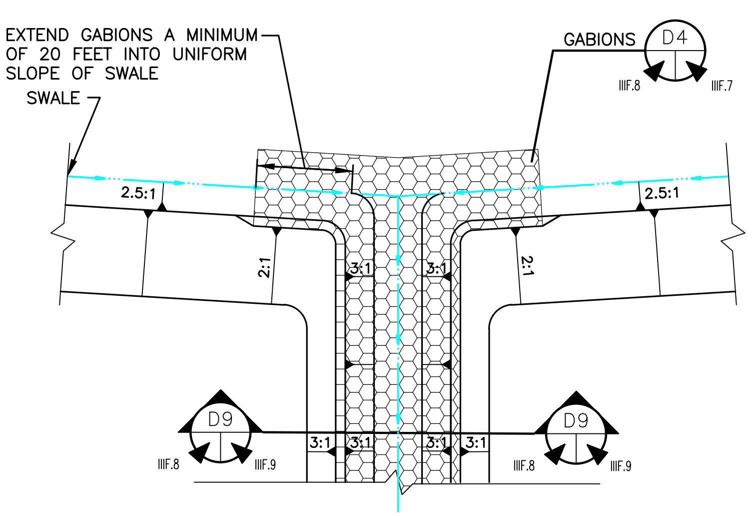
GABION OPTION



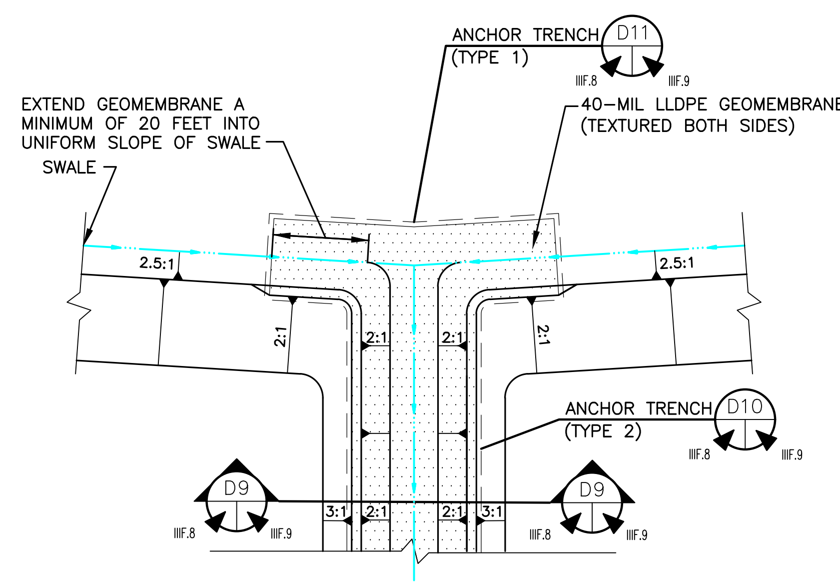
GEOMEMBRANE LINER OPTION

SWALE/CHUTE CONFLUENCE (TYP) D7 IIIF.1 IIIF.8

0 20 40
SCALE IN FEET



GABION OPTION



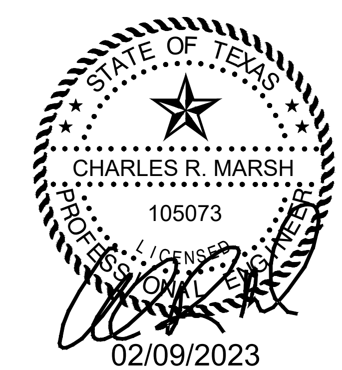
GEOMEMBRANE LINER OPTION

TOP OF CHUTE (TYP) D8 IIIF.1 IIIF.8

0 20 40
SCALE IN FEET

NOTES:

1. REFER TO FIGURE IIIF.1 DRAINAGE STRUCTURE PLAN FOR LOCATION OF DETAILS.
2. SEE APPENDIX IIIF-C FOR BOTTOM WIDTHS OF EACH INDIVIDUAL CHUTE.



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TBPE REGISTRATION NO. F-3727

MAJOR PERMIT AMENDMENT
DRAINAGE DETAILS

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

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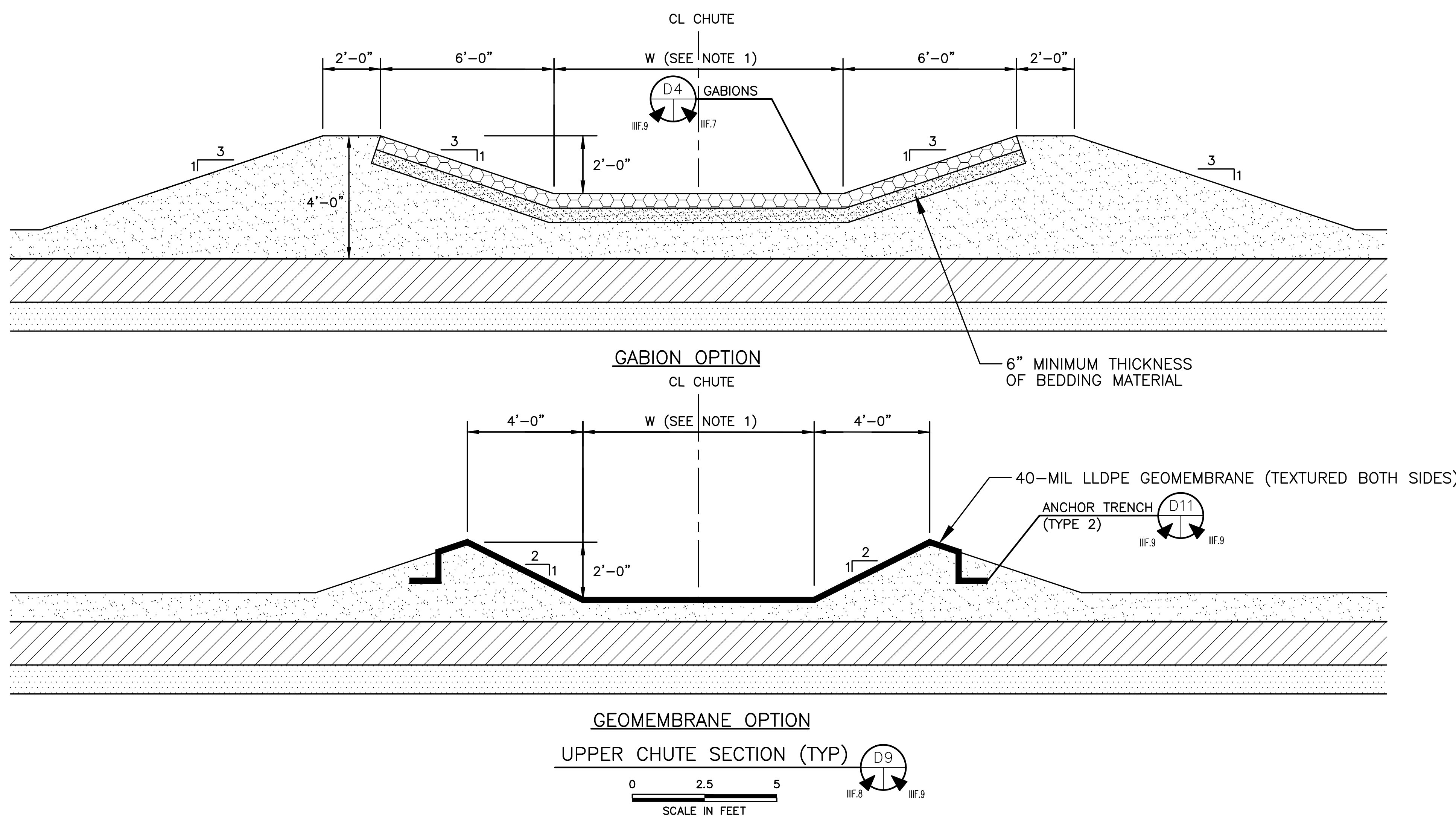
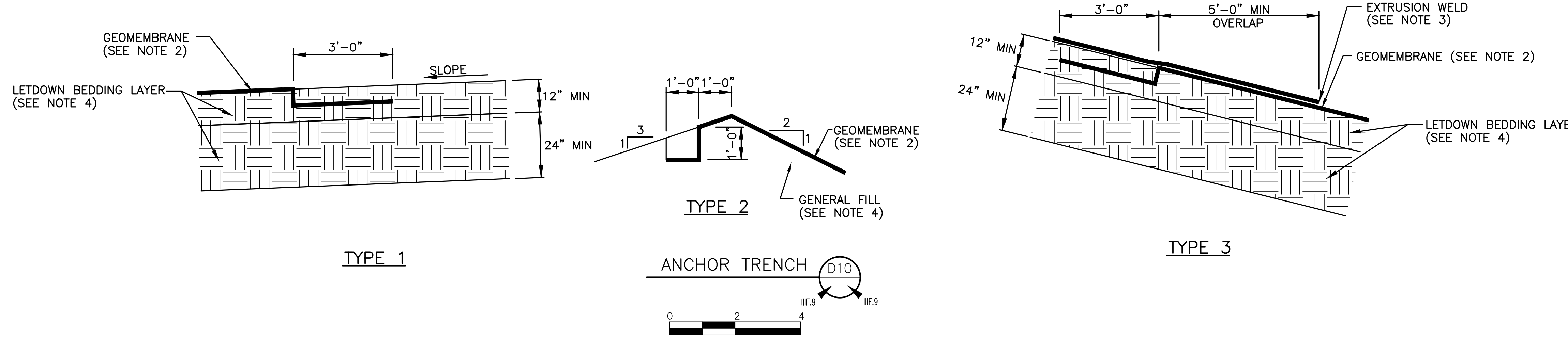
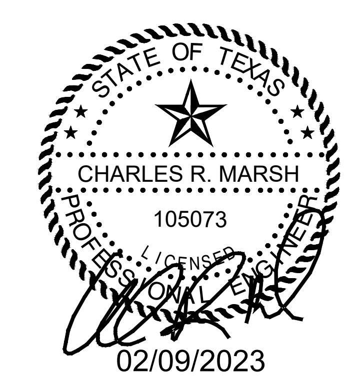


NOTES:

1. SEE APPENDIX IIIF–C FOR BOTTOM WIDTHS OF EACH INDIVIDUAL CHUTE.
2. 60 MIL HDPE GEOMEMBRANE TEXTURED BOTH SIDES SHALL BE USED FOR GEOMEMBRANE LETDOWN LINING.
3. EXTRUSION WELD UPSTREAM PANEL OVER DOWNSTREAM PANEL USING 1'–0" LONG EXTRUSION WELD WITH A SPACING OF 1'–0" BETWEEN EACH WELD.
4. SOIL PLACED UNDER GEOMEMBRANE LETDOWN AND CONCRETE DISSIPATER SHALL NOT CONTAIN TOPSOIL THAT WILL BE USED FOR VEGETATION LAYER.





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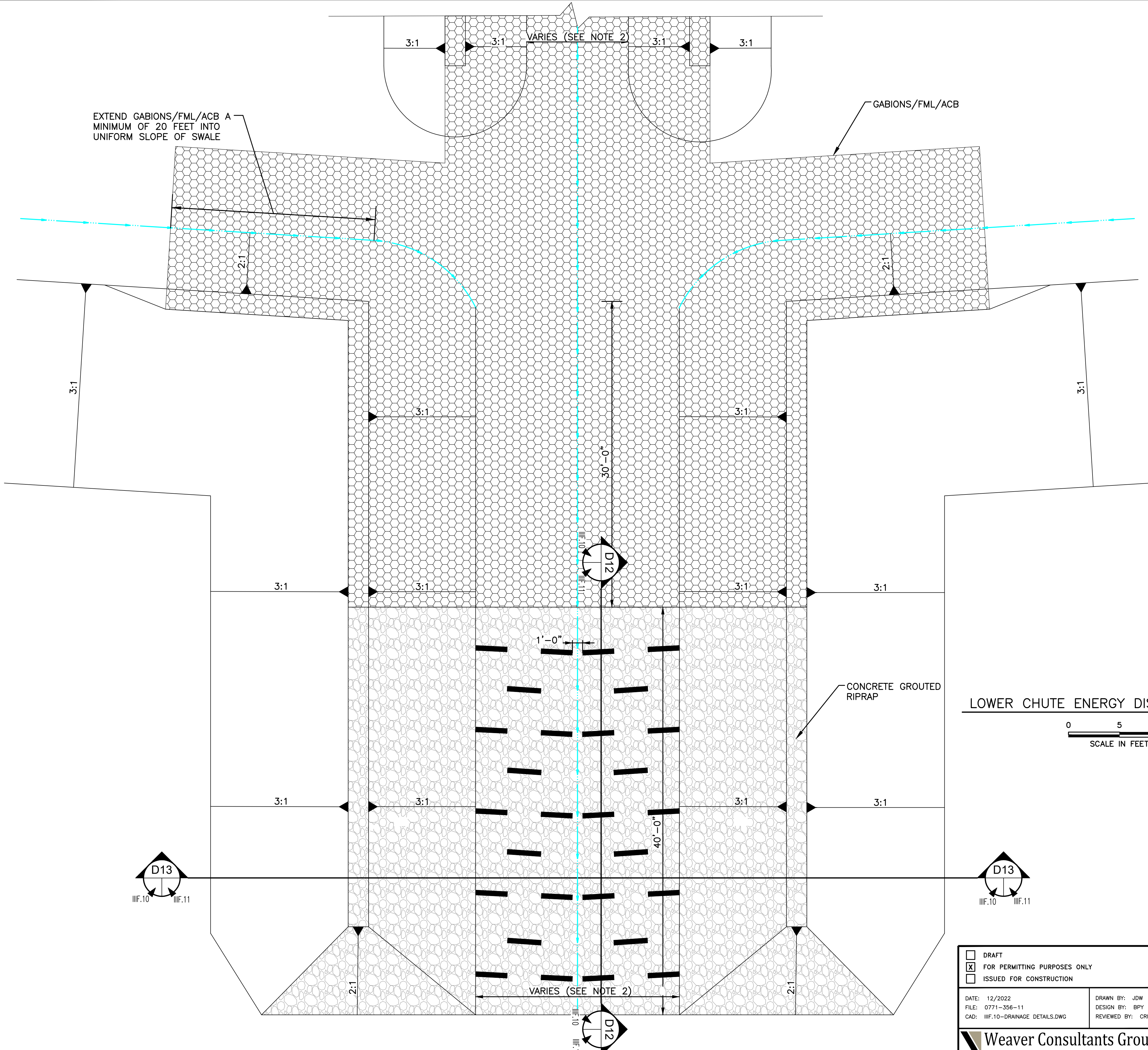
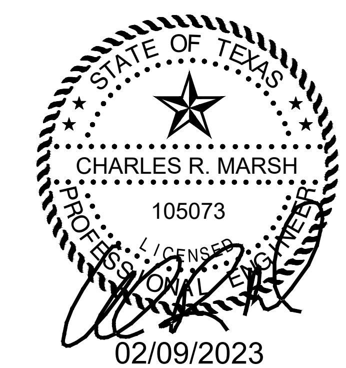


LOWER CHUTE ENERGY DISSIPATER (TYP) D11 IIIF.1 IIIF.10

0 5 10
SCALE IN FEET



NOTES:

1. REFER TO FIGURE IIIF.1 DRAINAGE STRUCTURE PLAN FOR LOCATION OF DETAILS.
2. SEE APPENDIX IIIF-C FOR BOTTOM WIDTHS OF EACH INDIVIDUAL CHUTE.

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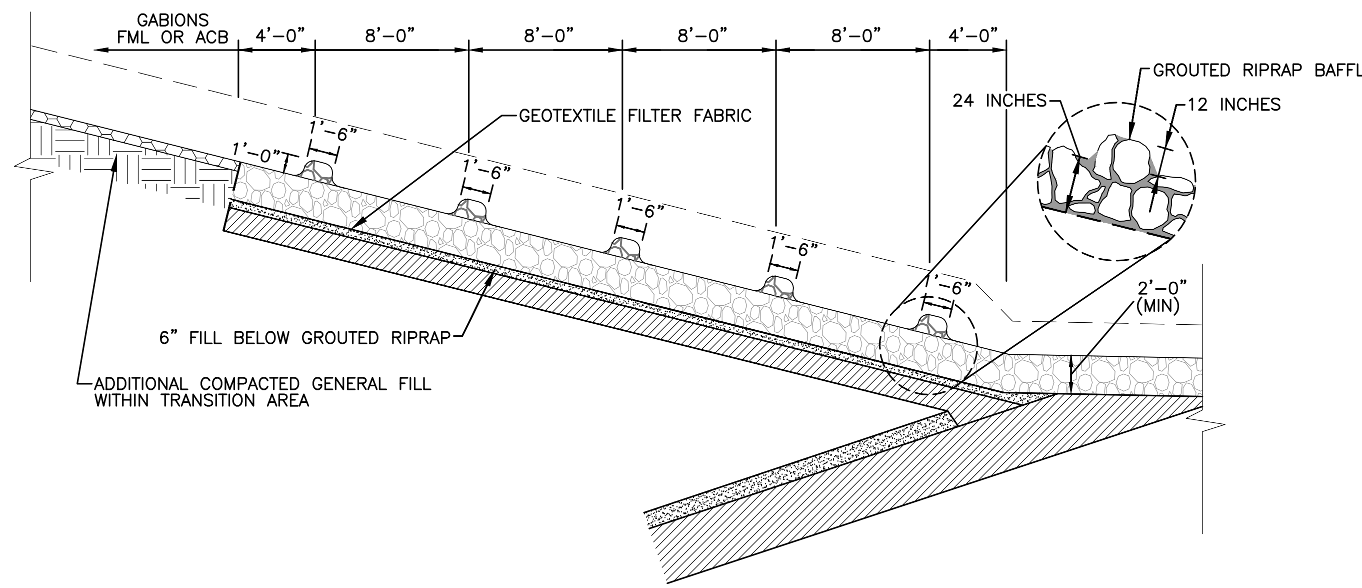
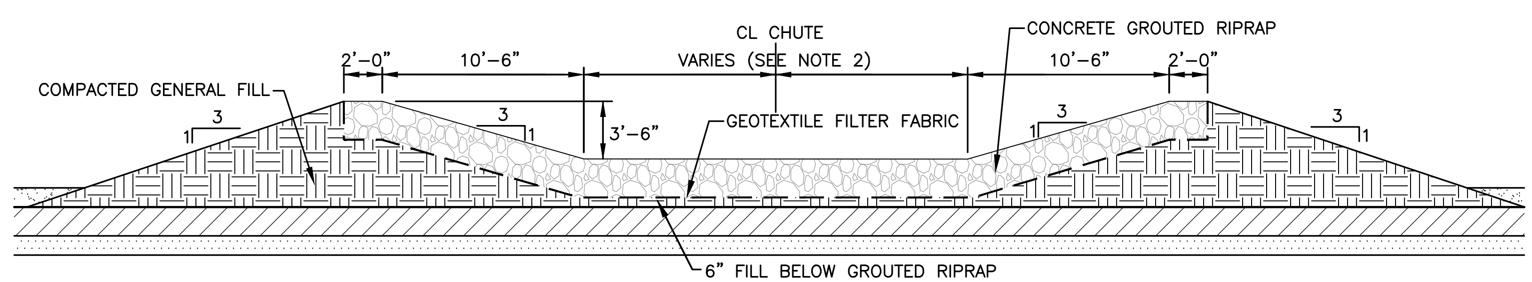


LOWER CHUTE ENERGY DISSIPATOR
SECTION (TYP)
D12
IIIF.10 IIIF.11

0 5 10

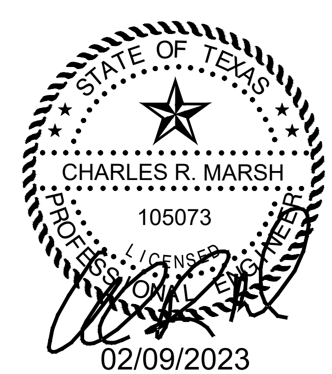


LOWER CHUTE
SECTION (TYP)
D13
IIIF.10 IIIF.11

0 5 10
SCALE IN FEET

NOTES:

1. REFER TO FIGURE IIIF.1 DRAINAGE STRUCTURE PLAN FOR LOCATION OF DETAILS.
2. SEE APPENDIX IIIF–C FOR BOTTOM WIDTHS OF EACH INDIVIDUAL CHUTE.



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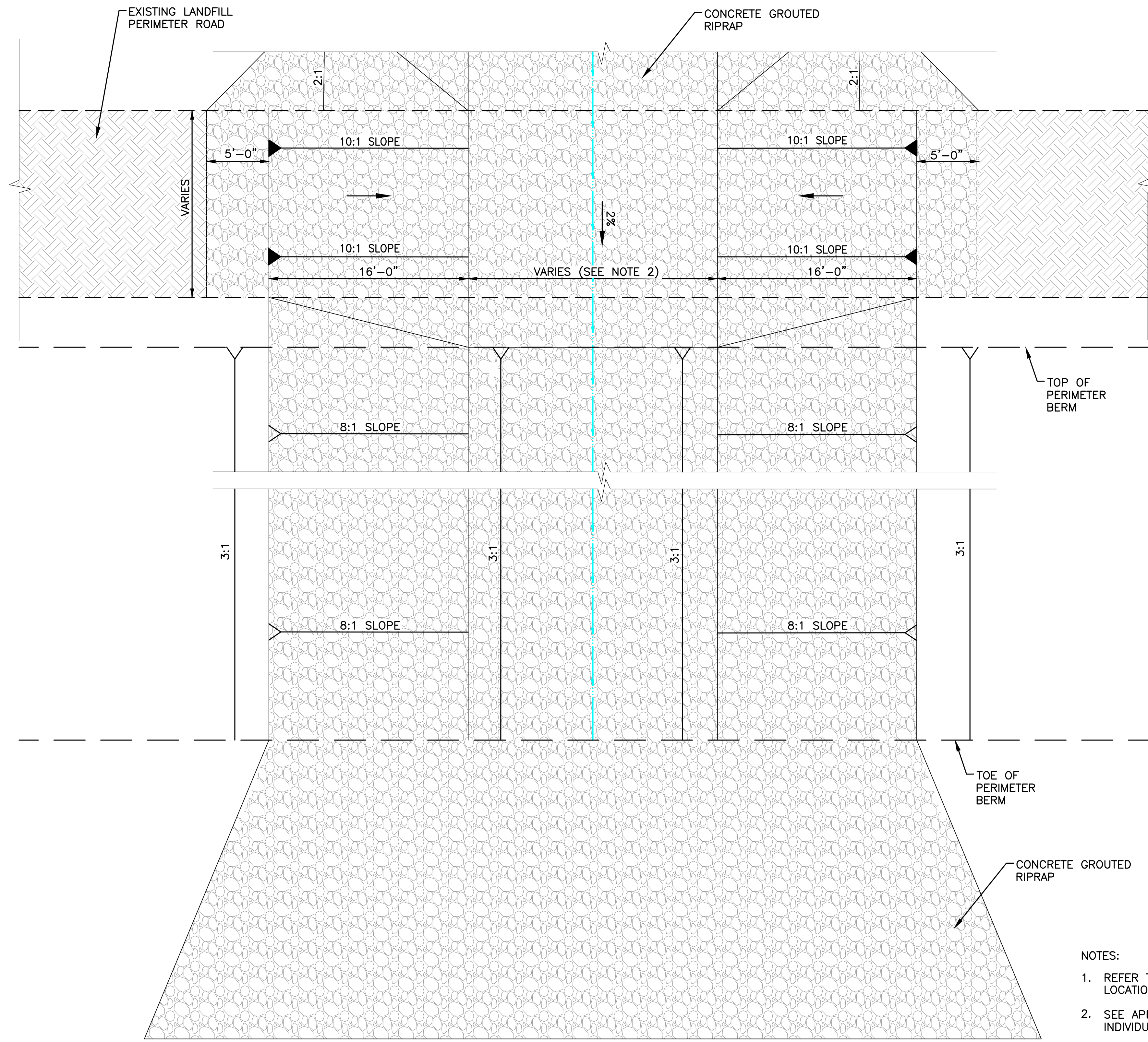
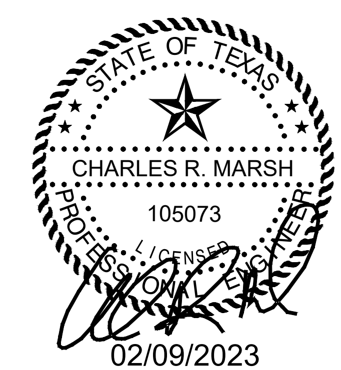


LOW WATER CROSSING (TYP)

D14 IIIF.1 IIIF.12

0 5 10
SCALE IN FEET



NOTES:

1. REFER TO FIGURE IIIF.1 DRAINAGE STRUCTURE PLAN FOR LOCATION OF DETAILS.
2. SEE APPENDIX IIIF–C FOR BOTTOM WIDTHS OF EACH INDIVIDUAL CHUTE.

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CAD: IIIF.12–DRAINAGE DETAILS.DWG

DRAWN BY: JDW
DESIGN BY: BPY
REVIEWED BY: CRM

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TEXAS REGIONAL LANDFILL COMPANY, LP

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MAJOR PERMIT AMENDMENT
DRAINAGE DETAILS

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

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DRAWING IIIF.12

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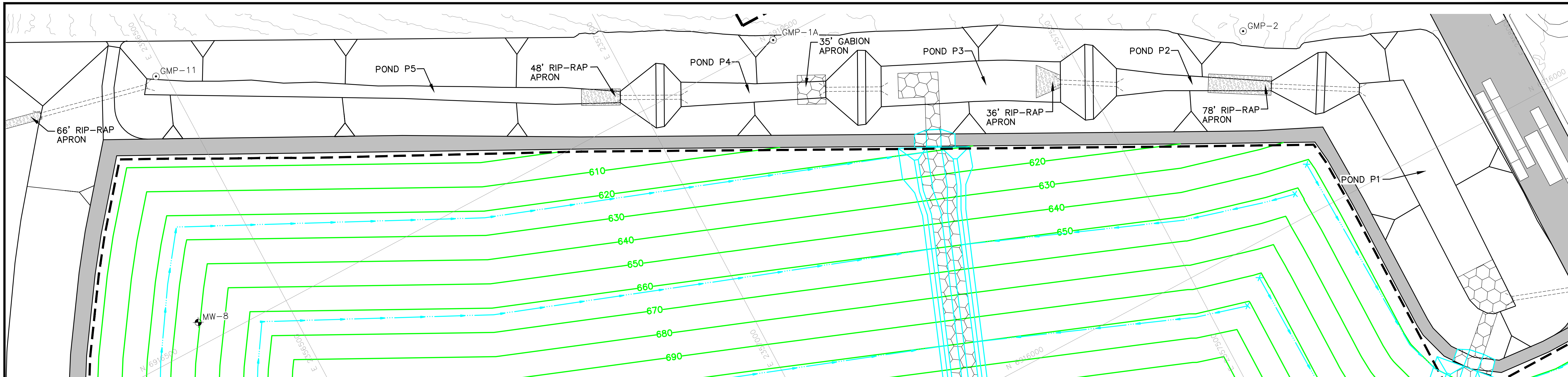

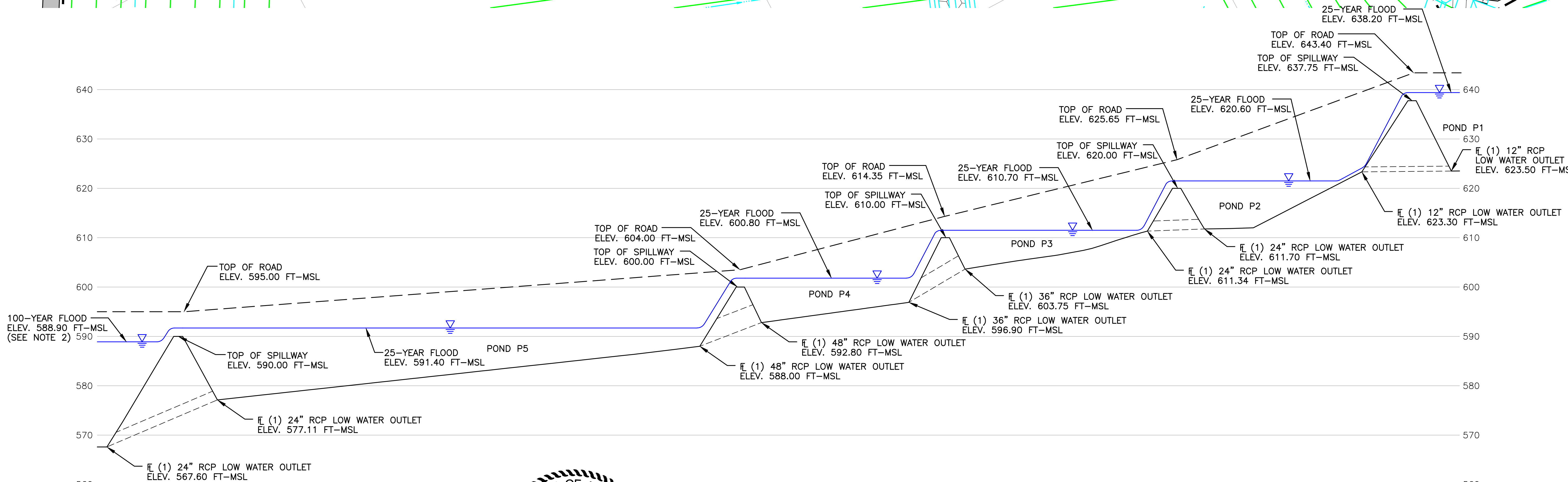

LONGITUDINAL SECTION B IIIF.14 IIIF.14

SCALE IN FEET (HORIZONTAL) 0 60 120

SCALE IN FEET (VERTICAL) 0 10 20

SCALE IN FEET 0 60 120

LEGEND

- PERMIT BOUNDARY
- LIMIT OF WASTE
- 630 EXISTING CONTOUR
- N 6915000 STATE PLANE COORDINATE
- 560 FINAL COVER CONTOUR
- DRAINAGE LETDOWN
- DRAINAGE SWALE
- GABIONS
- RIP-RAP

STATE OF TEXAS
CHARLES R. MARSH
105073
LICENSED PROFESSIONAL ENGINEER
02/09/2023

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS PROVIDED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN 02-17-2022.
2. 100-YEAR FLOOD ELEVATION BASED ON THE PROPOSED CLOMR. SEE APPENDIX IIIF-G FOR MORE INFORMATION.

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DATE: 12/2022
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MAJOR PERMIT AMENDMENT
PONDS P1 THROUGH P5 PLAN

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

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O:\0771\356\EXPANSION 2022\PART III\IIIF\IIIF-13 PONDS P1-P5.dwg, jwilson, 1:2

APPENDIX IIIF-A

POST-DEVELOPMENT CONDITION HYDROLOGIC CALCULATIONS

Includes pages IIIF-A-1 through IIIF-A-67

STATE OF TEXAS

CHARLES R. MARSH

105073

LICENSED

PROFESSIONAL ENGINEER

02/09/2023

CONTENTS

STATE OF TEXAS
CHARLES R. MARSH
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PROFESSIONAL ENGINEER
02/09/2023

HYPOTHETICAL STORM DATA

Hypothetical Storm Data

Precipitation data taken from NOAA Atlas 14 rainfall data.

Time	5 min	15 min	60 min	2 hr	3 hr	6 hr	12 hr	24 hr
25-Year Event	0.82	1.63	2.96	3.77	4.28	5.19	6.15	7.17

NOAA Atlas 14 - Precipitation-Frequency Atlas of the United States, Volume 11, Version 2.0: Texas (*U.S. Department of Commerce, National Oceanic and Atmospheric Administration, and National Weather Service, 2018*) was used to identify precipitation values for storm durations ranging from 5 minutes to 24 hours.

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. U.S. Army Corps of Engineers, Hydrologic Engineering Center, *HEC-HMS Hydrologic Modeling System 4.9,* January 2022.
2. United States Department of Agriculture, National Resource Conservation Service, Web Soil Survey for Johnson County, Texas (http://websoilsurvey.nrcs.usda.gov).
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-5).

Solution: Based on the soil survey information found in Ref. 2, hydrologic group B, C, and D soils predominate the soils within the permit boundary drainage area (see pages IIIF-A-5 through IIIF-A-8). Hydrologic group D was selected to represent the onsite soils.

The curve number for the offsite drainage areas around the site, large non-landfill drainage basins within the permit boundary, and drainage channels (O1, O2, O3, O4, S1, S2, CH1, CH2, CH3, CH4, and CH5) was calculated using the table on Page IIIF-A-11, assuming pasture land in fair conditions. The majority of the area is undeveloped and assume compare to the off-site and on-site subasins near the site.

Use:	CN = 84

The final cover system was assumed to be in place and the erosion layer will control precipitation loss. A curve number that is corrected for the surface slope of the erosion layer may be computed first using the chart on page IIIF-A-11 to select an un-adjusted curve numb Calculate the adjusted curve number using equation 34 from Ref. 3 (see page IIIF-A-10).

$$CN_{II} = 100 - (100 - CN_{II\,o}) * (L^{*\,2} / S^{*}) \wedge (CN_{II\,o}^{-0.81})$$

Use:	$CN_{II\,o}$ = 84 , L^* = (500/500) , S^* = (.05/.04)	for top dome surfaces
Use:	$CN_{II\,o}$ = 84 , L^* = (120/500) , S^* = (.33/.04)	for side slopes

Calculate:	CN = 84	for top dome surfaces
Calculate:	CN = 86	for side slopes

- Use curve number calculated for side slopes for the entire final cover area, inculding top dome areas, conservatively.

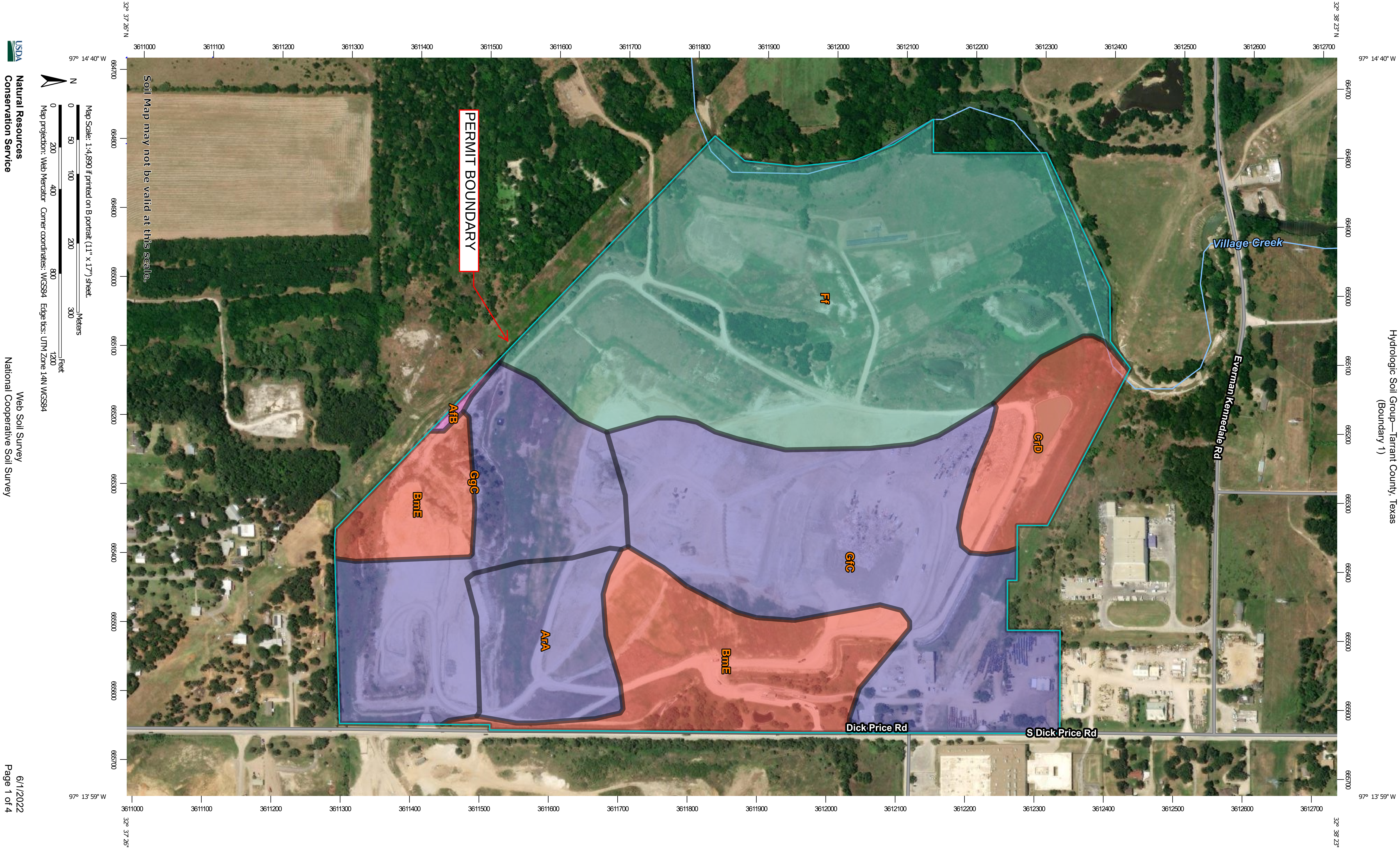
The pond areas are assumed to collect all precipitation for their areas:

Use:	CN = 99

The initial abstraction is:

Use:	I = 0.0"

- All drainage areas were modeled to assume no inital abstractions.


Hydrologic Soil Group—Tarrant County, Texas
(Boundary 1)
USDA Natural Resources Conservation Service
Web Soil Survey
National Cooperative Soil Survey
6/1/2022
Page 1 of 4
Map Scale: 1:4,890 if printed on B portrait (11" x 17") sheet.
Meters
Feet
Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 14N WGS84
Soil Map may not be valid at this scale.
PERMIT BOUNDARY
Ff
GrD
GfC
GgC
BmE
ArA
AfB
Village Creek
Everman Kennedale Rd
Dick Price Rd
S Dick Price Rd
97° 14' 40" W
97° 13' 59" W
32° 37' 26" N
32° 38' 23" N

MAP LEGEND

Area of Interest (AOI)

Area of Interest (AOI)

Soils

Soil Rating Polygons

A

A/D

B

B/D

C

C/D

D

Not rated or not available

Soil Rating Lines

A

A/D

B

B/D

C

C/D

D

Not rated or not available

Soil Rating Points

A

A/D

B

B/D

C

C/D

D

Not rated or not available

Water Features

Streams and Canals

Transportation

Rails

Interstate Highways

US Routes

Major Roads

Local Roads

Background

Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:20,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Tarrant County, Texas
Survey Area Data: Version 19, Sep 10, 2021

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Jan 18, 2020—Nov 15, 2020

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Hydrologic Soil Group

Map unit symbol	Map unit name	Rating	Acres in AOI	Percent of AOI
AfB	Arents, frequently flooded	A	0.3	0.2%
ArA	Arents, loamy	B	11.2	6.1%
BmE	Birome-Aubrey-Rayex complex, 5 to 15 percent slopes	D	26.2	14.2%
CrD	Crosstell fine sandy loam, 3 to 8 percent slopes	D	10.3	5.6%
Ff	Frio clay loam, 0 to 1 percent slopes, frequently flooded	C	69.7	37.8%
GfC	Gasil fine sandy loam, 3 to 8 percent slopes	B	42.3	22.9%
GgC	Gasil sandy clay loam, graded, 1 to 5 percent slopes	B	24.3	13.2%
Totals for Area of Interest			**184.4**	**100.0%**

Description

Hydrologic soil groups are based on estimates of runoff potential. Soils are assigned to one of four groups 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 rate of water transmission.

Group B. Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately 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 a 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 that 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 and the second is for undrained areas. Only the soils that in their natural condition are in group D are assigned to dual classes.

Rating Options

Aggregation Method: Dominant Condition

Component Percent Cutoff: None Specified

Tie-break Rule: Higher

where

CN_{II_o} = AMC-II curve number for mild slope (unadjusted for slope)

C_O = regression constant for a given level of vegetation

C_1 = regression constant for a given level of vegetation

C_2 = regression constant for a given level of vegetation

IR = infiltration correlation parameter for given soil type

The relationship between CN_{II} , the vegetative cover and default soil texture is shown graphically in Figure 8. Table 7 gives values of C_0, C_1 and C_2 for the five types of vegetative cover built into the HELP program.

4.2.3 Adjustment of Curve Number for Surface Slope

A regression equation was developed to adjust the AMC-II curve number for surface slope conditions. The regression was developed based on kinematic wave theory where

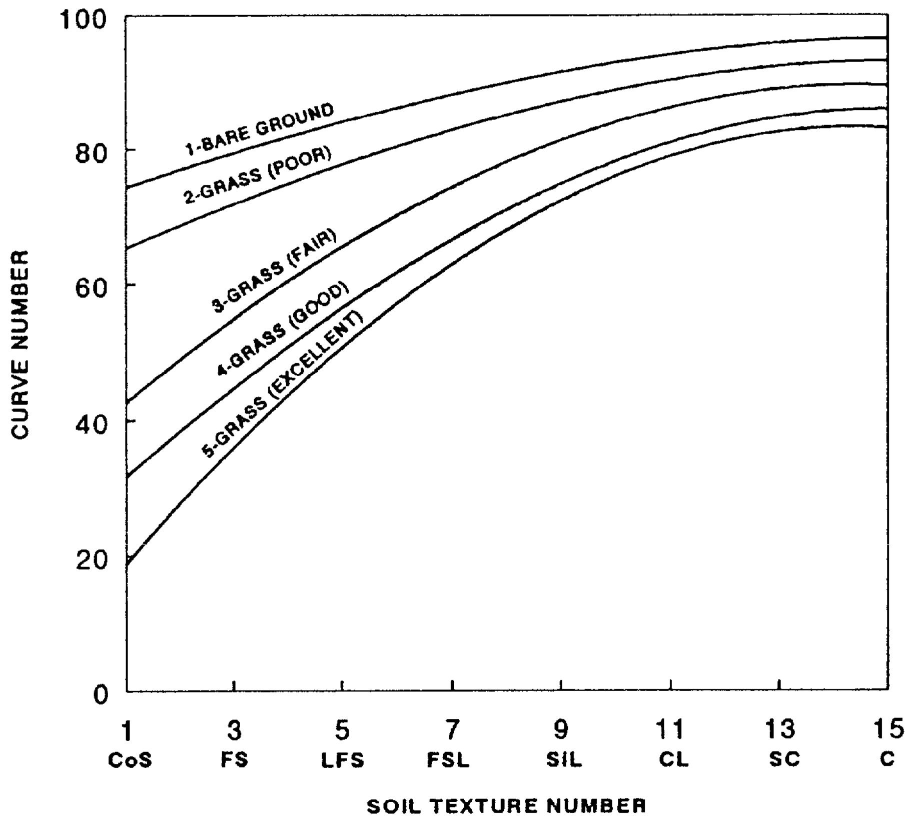


Figure 8. Relation between SCS Curve Number and Default Soil Texture Number for Various Levels of Vegetation

loam, and clayey loam as specified by saturated hydraulic conductivity, capillary drive, porosity, and maximum relative saturation, Two levels of vegetation were described--a good stand of grass (bluegrass sod) and a poor stand of grass (clipped range). Slopes of 0.04,0.10,0.20,0.35, and 0.50 ft/ft and slope lengths of 50, 100, 250, and 500 ft were used. Rainfalls of 1.1 inches, l-hour duration and 2nd quartile Huff distribution and of 3.8 inches, 6-hour duration and balanced distribution were modeled.

The resulting regression equation used for adjusting the AMC-II curve number computed for default soils and vegetation placed at mild slopes, CN_{II_o}, is:

$$CN_{II} = 100 - (100 - CN_{II_o}) \cdot \left(\frac{L^{*2}}{S^*} \right)^{CN_{II_o}^{-0.81}} \tag{34}$$

where

L^* = standardized dimensionless length, (L/500 ft)

S^* = standardized dimensionless slope, (S/0.04)

This same equation is used to adjust user-specified AMC-II curve numbers for surface slope conditions by substituting the user value for CN_{II_o} in Equation 34.

4.2.4 Adjustment of Curve Number for Frozen Soil

When the HELP program predicts frozen conditions to exist, the value of CN_{II} is increased, resulting in a higher calculated runoff. Knisel et al. (1985) found that this type of curve number adjustment in the CREAMS model resulted in improved predictions of annual runoff for several test watersheds. If the CN_{II} for unfrozen soil is less than or equal to 80, the CN_{II} for frozen soil conditions is set at 95. When the unfrozen soil CN_{II} is greater than 80, the CN_{II} is reset to be 98 on days when the program has determined the soil to be frozen. This adjustment results in an increase in CN_I and consequently a decrease in S_{mx} and S' (Equations 19, 26, and 30).

From Equations 19 and 21, it is apparent that as S' approaches zero, Q approaches P. In other words, as S' decreases, the calculated runoff becomes closer to being equal to the net rainfall which is most often, when frozen soil conditions exist, predominantly snowmelt. This will result in a decrease in infiltration under frozen soil conditions, which has been observed in numerous studies.

4.2.5 Summary of Daily Runoff Computation

The HELP model determines daily runoff by the following procedure:

TABLE 5.3 Values of SCS Curve Number for Rural Areas

Source: [McCuen, 1982]

Land Use Description	Hydrologic Soil Group 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

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HYDROGRAPH DEVELOPMENT INFORMATION

HYDROGRAPH DEVELOPMENT INFORMATION

Landfill Areas

Direct runoff methods, (i.e., kinematic wave) have been used for the majority of the landfill final cover areas. The kinematic wave method has been used to model the 5 percent topslope areas and 33 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 0.05 to 0.33 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-17, the swale spacing on 3H:1V sideslopes is 120 feet.

The percentage of the drainage area represented by these parameters is typically 100 percent.

Kinematic Wave routing for channels:

- Channel length (ft): The length of the channel section.
- Channel slope (ft/ft): Varies from 0.005 to 0.120 (0.005 for swales).
- Channel roughness coefficient: 0.03 for swales and channels.
- Channel type: A trapezoidal channel was used with varying width and 2.5:1 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. Espey "10-Minute" method has been used to estimate Snyder parameters. Snyder parameter estimations are provided on pages IIIF-A-18 through IIIF-A-23.

As discussed in Section 2 of Appendix IIIF, hydrographs for the areas outside of the permit boundary (O1, O2, O3, and O4), and larger areas inside the permit boundary (S1, S2, and S3) were developed using the Snyder unit hydrograph method. The percent imperviousness ranges from 2 percent to 25 percent, indicating the majority of each watershed is undeveloped. Pond areas are assumed to be 100 percent impervious, and areas with significant channel surface or paved surfaces were assigned higher percentages of impervious area, as shown on IIIF-A-19.

Drainage Areas

The drainage areas used for this analysis are shown on Sheets IIIF-A-25 and IIIF-A-26. The routing scheme for the post-development condition is shown in the HEC-HMS output file presented on pages IIIF-A-27 through IIIF-A-59.

DISTRIBUTED RUNOFF METHOD
KINEMATIC WAVE EXAMPLE

Drainage area "DA2" is used in this example (refer to Sheet IIIF-A-17 for location of drainage area).

Watershed Specific Parameters:

A =	32.15	acres	Watershed Area (acres)
A =	0.0502	sq-miles	Watershed Area (sq-miles)
CN=	86		SCS Curve Number (see sheet IIIF-A-4 for more information)

Kinematic Wave parameter for overland flow:

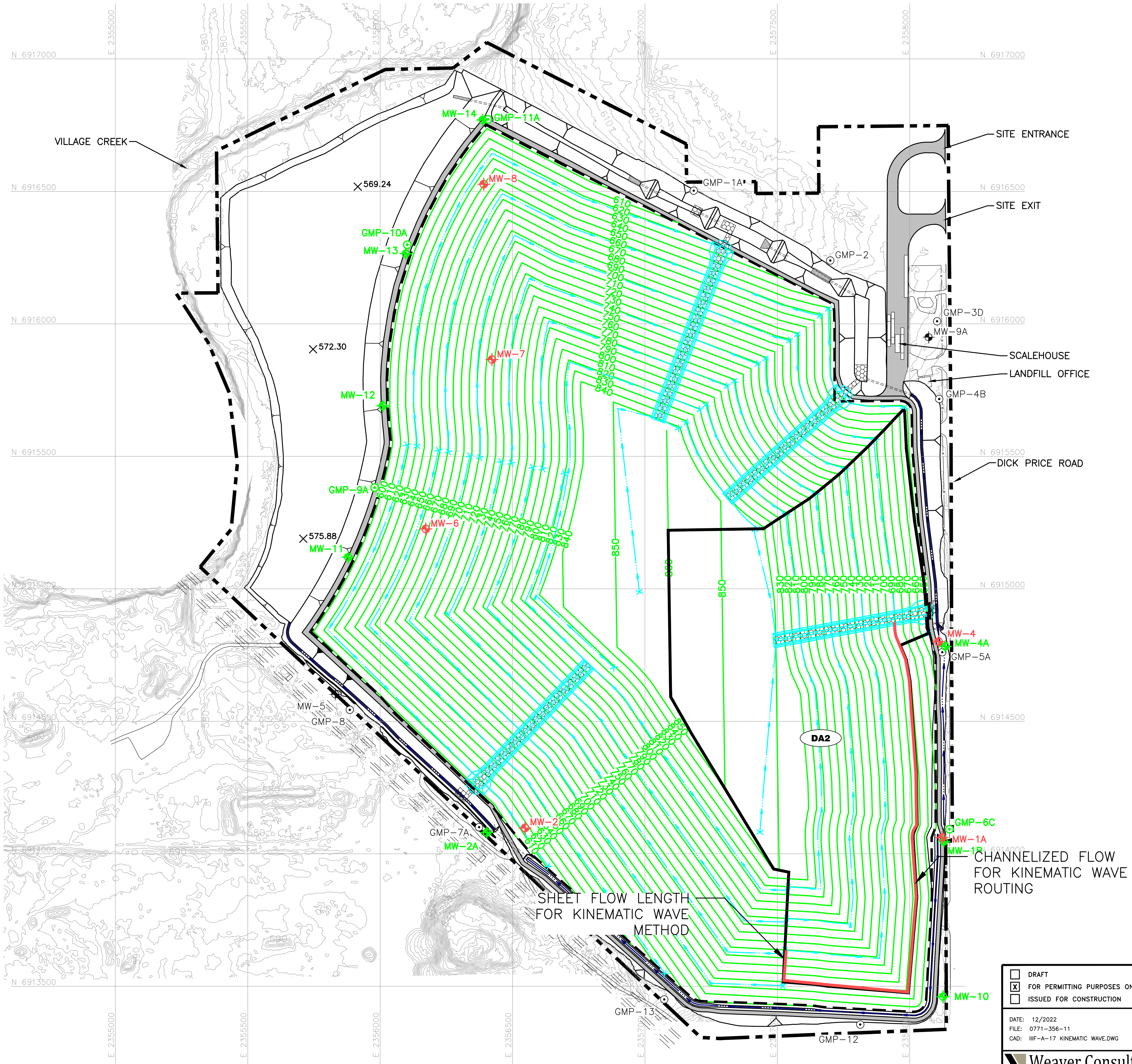
L=	120	ft	Typical overland flow (ft)
S=	0.33	ft/ft	Landfill Slope (ft/ft)
N=	0.030		Manning's Coefficient

Percentage of the drainage area represented by this element is 100 percent

Kinematic Wave routing data for the swale:

L=	1900	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.



N

0 200 400
SCALE IN FEET

LEGEND

	PERMIT BOUNDARY
	LIMIT OF WASTE
630	EXISTING CONTOUR
N 6915000	STATE PLANE COORDINATE
560	FINAL COVER CONTOUR
	DRAINAGE LETDOWN
	DRAINAGE SWALE
	PROPOSED PERIMETER CHANNEL
	GABIONS
MW-7	EXISTING/PERMITTED GROUNDWATER MONITORING WELL
MW-9A	APPROVED GROUNDWATER MONITORING WELL
MW-4	PERMITTED GROUNDWATER MONITORING WELL (TO BE ABANDONED)
GMP-4B	PERMITTED GAS MONITORING PROBE
	DRAINAGE AREA BOUNDARY
DA2	DRAINAGE AREA DESIGNATION

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS PROVIDED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN 02-17-2022.
2. REFER TO APPENDIX IIIF-SURFACE WATER DRAINAGE PLAN FOR DRAINAGE DESIGN INFORMATION.
3. MAXIMUM FINAL COVER ELEVATION 860 FT-MSL.

STATE OF TEXAS
CHARLES R. MARSH
105073
LICENSED
PROFESSIONAL ENGINEER
02/09/2023

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PREPARED FOR
TEXAS REGIONAL LANDFILL COMPANY, LP

DATE: 12/2022
FILE: 0771-356-11
CAD: IIIF-A-17 KINEMATIC WAVE.DWG

DRAWN BY: JDW
DESIGN BY: BPY
REVIEWED BY: CRM

REVISIONS

NO.	DATE	DESCRIPTION

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MAJOR PERMIT AMENDMENT
KINEMATIC WAVE PARAMETERS

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

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O:\0771\356\EXPANSION 2022\PART III\IIIF\IIIF-A\IIIF-A-17 KINEMATIC WAVE.dwg, jwilson, 1:2

ESPEY 10-MINUTE METHOD PARAMETERS

FORT WORTH C&D LANDFILL
0771-356-11-35
UNIT HYDROGRAPH DATA
PROPOSED EXPANSION CONDITION

Snyder's Hydrograph Coefficients (Espey's 10 Minute Method)

Proposed Expansion Condition

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]
O1	5.11	935	0.0310	15	0.04	0.82	16.0	13.5	0.23	0.0080	1970.2	0.69
O2	4.56	590	0.0322	15	0.04	0.82	14.3	11.8	0.20	0.0071	2239.0	0.69
O3	17.39	680	0.0838	20	0.04	0.80	10.6	8.1	0.14	0.0272	2916.5	0.62
O4	6.25	530	0.0642	20	0.04	0.80	10.7	8.2	0.14	0.0098	3007.5	0.64
S1	9.92	1,225	0.0343	25	0.04	0.79	14.3	11.8	0.20	0.0155	2166.1	0.67
S2	35.85	2,265	0.0046	2	0.04	0.87	49.8	47.3	0.79	0.0560	541.5	0.67
S3	2.03	280	0.0560	2	0.04	0.87	16.5	14.0	0.23	0.0032	1978.6	0.72

[1] Conveyance efficiency coefficient from *HEC-HMS Hydrologic Modeling System 4.9, 2000,* 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

Snyder Unit Hydrograph uses lag time (T_{lag}) and peaking coefficient accounting for flood wave and watershed storage conditions.

Drainage area "S1" in the existing permitted condition is used in this example.

Estimated Watershed specific parameters

A =	9.92	acres	watershed area
L =	1225	feet	maximun flow length with this watershed
S =	0.0343	feet/feet	watershed slope
I =	25	percent (%)	watershed imperviousness
n =	0.04		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

Φ = for 25 percent impervious cover and n = 0.04

$\Phi = 0.79$

$T_r = 3.1(1225)(0.0343^{-0.25})(25^{-0.18})(0.79^{.57})$

$T_r = 14.3$ min

Calculate T_{lag}: watershed lag time

$T_{lag} = Tr - (\Delta t/2)$ — Δt is calculation interval, and 5 minutes is used in the HEC-HMS modeling in this project

$T_{lag} = 11.8$ minutes

$T_{lag} = 0.20$ hours

$A = A/640$

$A = 0.0155$ square miles

Calculate q_p: peak discharge of unit hydrograph per unit area (cfs/sq. mi).

$q_p = 31600(A^{-0.04})(T_r^{-1.07})$

$q_p = 31600(0.0155^{-0.04})(14.3^{-1.07})$

$q_p = 2166.1$ cfs/sq. mi

Calculate Peaking coefficient C_p:

$C_p = 49.375(A^{-0.04})(T_r^{-1.07})(T_{lag})$

$C_p = 49.375(0.0155^{-0.04})(14.3^{-1.07})(0.20)$

$C_p = 0.67$

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.

$$C_p = \frac{q_p \times T_L}{640} \qquad 6.30$$

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:

$$T_r = 3.10\, L^{0.23} S^{-0.25} I^{-0.18}\, \Phi^{1.57} \qquad 6.31$$

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.

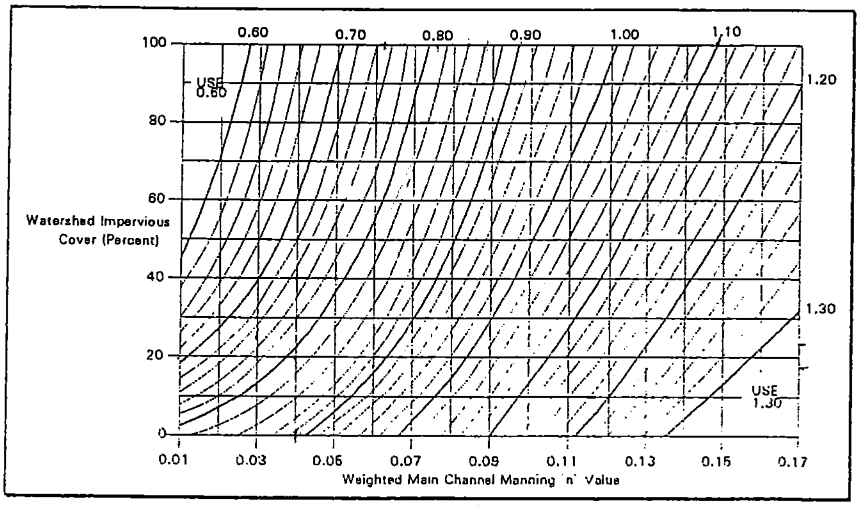

FIGURE 6.12 Determination of Conveyance Efficiency Coefficient Φ

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

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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 - Δt/2).

The relationship developed by Espey to compute the peak flow rate of the unit hydrograph is as follows:

6.32 $$Q_u = 31600 \; A^{0.96} T_r^{-1.07}$$

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 $$T_L = 1.20\left(\frac{L \times L_{ca}}{\sqrt{S}}\right)^{0.38}$$ (Mountain Areas)

6.34 $$T_L = 0.72\left(\frac{L \times L_{ca}}{\sqrt{S}}\right)^{0.38}$$ (Foothill Areas)

6.35 $$T_L = 0.38\left(\frac{L \times L_{ca}}{\sqrt{S}}\right)^{0.38}$$ (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.

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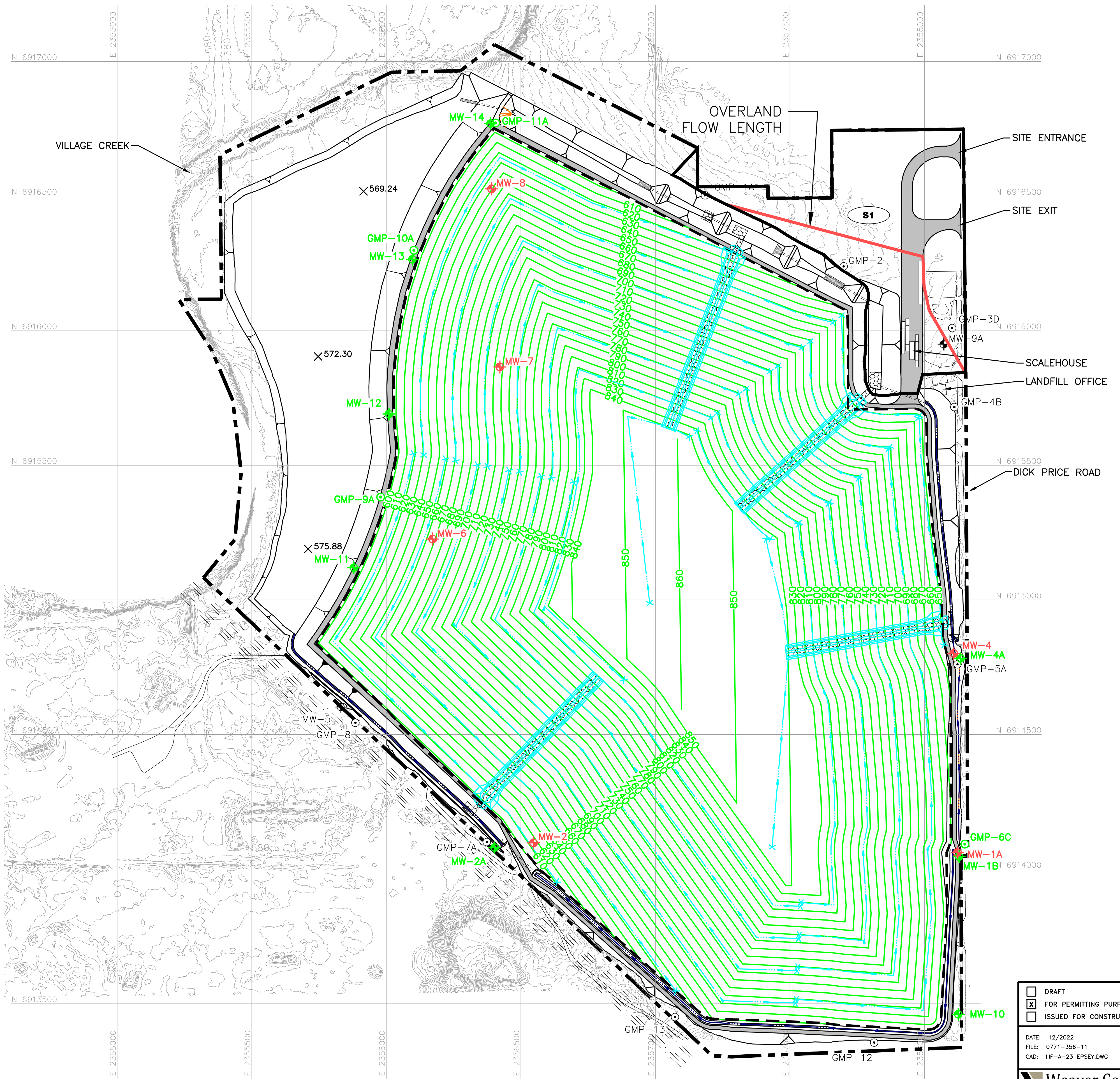

0 200 400

SCALE IN FEET

LEGEND

- PERMIT BOUNDARY
- LIMIT OF WASTE
- 630 EXISTING CONTOUR
- N 6915000 STATE PLANE COORDINATE
- 560 FINAL COVER CONTOUR
- DRAINAGE LETDOWN
- DRAINAGE SWALE
- PROPOSED PERIMETER CHANNEL
- GABIONS
- MW-7 EXISTING/PERMITTED GROUNDWATER MONITORING WELL
- MW-9A APPROVED GROUNDWATER MONITORING WELL
- MW-4 PERMITTED GROUNDWATER MONITORING WELL (TO BE ABANDONED)
- GMP-4B PERMITTED GAS MONITORING PROBE
- DRAINAGE AREA BOUNDARY
- S1 DRAINAGE AREA DESIGNATION

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS PROVIDED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN 02-17-2022.
2. REFER TO APPENDIX IIIF-SURFACE WATER DRAINAGE PLAN FOR DRAINAGE DESIGN INFORMATION.
3. MAXIMUM FINAL COVER ELEVATION 860 FT-MSL.

STATE OF TEXAS
CHARLES R. MARSH
105073
LICENSED
PROFESSIONAL ENGINEER
02/09/2023

☐ DRAFT
☒ FOR PERMITTING PURPOSES ONLY
☐ ISSUED FOR CONSTRUCTION

DATE: 12/2022
FILE: 0771-356-11
CAD: IIIF-A-23 EPSEY.DWG

DRAWN BY: JDW
DESIGN BY: BPY
REVIEWED BY: CRM

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

PREPARED FOR
TEXAS REGIONAL LANDFILL COMPANY, LP

REVISIONS

NO.	DATE	DESCRIPTION

MAJOR PERMIT AMENDMENT EPSEY "10-MINUTE METHOD PARAMETERS

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

WWW.WCGRP.COM

DRAWING IIIF-A-23

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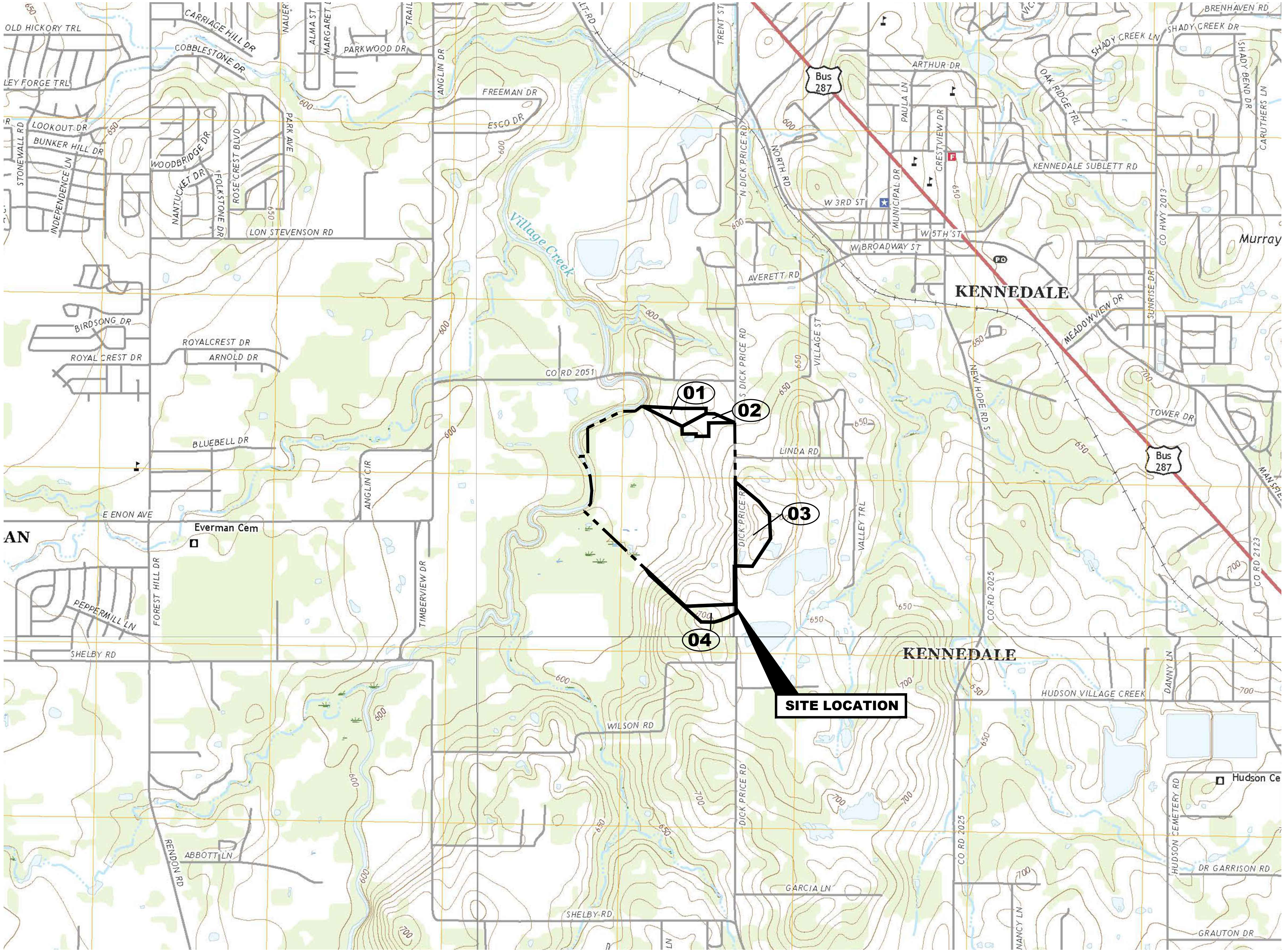
POST-DEVELOPMENT HEC-HMS ANALYSIS
DRAINAGE AREAS



N

0 1000 2000
SCALE IN FEET

LEGEND

PERMIT BOUNDARY
DRAINAGE AREA BOUNDARY
02 DRAINAGE AREA LABEL

ROAD CLASSIFICATION

Expressway — Local Connector
Secondary Hwy — Local Road
Ramp — 4WD
Interstate Route — US Route — State Route

DRAINAGE AREA NO.	AREA (ACRES)
01	5.11
02	4.56
03	17.39
04	6.25
TOTAL	33.32

NOTES:

1. ADAPTED FROM USGS 7.5 MINUTE QUADRANGLE TOPOGRAPHIC MAP (FORT WORTH, KENNEDALE, BURLESON AND MANSFIELD, TEXAS, 2019).
2. DRAINAGE AREA DELINEATION WITHIN THE PERMIT BOUNDARY IS INCLUDED ON DRAWING IIIF-E-26.

FORT WORTH, TX 2019

KENNEDALE, TX 2019

BURLESON, TX 2019

MANSFIELD, TX 2019

STATE OF TEXAS
CHARLES R. MARSH
105073
LICENSED
PROFESSIONAL ENGINEER
02/09/2023

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DATE: 12/2022
FILE: 0771-356-11
CAD: IIIF-A-25-OFFSITE DRAINAGE.DWG

DRAWN BY: JDW
DESIGN BY: BPY
REVIEWED BY: CRM

PREPARED FOR
TEXAS REGIONAL LANDFILL COMPANY, LP

REVISIONS

NO.	DATE	DESCRIPTION

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

MAJOR PERMIT AMENDMENT
OFFSITE DRAINAGE AREAS

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

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DRAWING IIIF-A-25

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0 200 400

SCALE IN FEET

LEGEND

- PERMIT BOUNDARY
- LIMIT OF WASTE
- 630 EXISTING CONTOUR
- N 6915000 STATE PLANE COORDINATE
- 560 FINAL COVER CONTOUR
- DRAINAGE LETDOWN
- DRAINAGE SWALE
- PROPOSED PERIMETER CHANNEL
- GABIONS
- MW-7 EXISTING/PERMITTED GROUNDWATER MONITORING WELL
- MW-9A APPROVED GROUNDWATER MONITORING WELL
- MW-4 PERMITTED GROUNDWATER MONITORING WELL (TO BE ABANDONED)
- GMP-4B PERMITTED GAS MONITORING PROBE
- DRAINAGE DIVIDE
- O3 DRAINAGE AREA DESIGNATION

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS PROVIDED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN 02-17-2022.
2. REFER TO APPENDIX IIIF-SURFACE WATER DRAINAGE PLAN FOR DRAINAGE DESIGN INFORMATION.
3. MAXIMUM FINAL COVER ELEVATION 860 FT-MSL.

DRAINAGE AREA NO.	AREA (ACRES)	DRAINAGE AREA NO.	AREA (ACRES)
DA1	35.42	P1	2.16
DA2	32.15	P2	3.77
DA3	8.98	P3	3.37
DA4	33.08	CH1	2.52
S1	9.92	CH2	3.31
S2	35.42	CH3	3.53
S3	2.03	CH4	4.80
O1	5.11	CH5	3.90
O2	4.56		
O3	17.39		
O4	6.25		

STATE OF TEXAS
CHARLES R. MARSH
105073
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PROFESSIONAL ENGINEER

02/09/2023

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DATE: 12/2022
FILE: 0771-356-11
CAD: IIIF-A-26 DRAINAGE AREAS.DWG

DRAWN BY: JDW
DESIGN BY: BPY
REVIEWED BY: CRM

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

PREPARED FOR
TEXAS REGIONAL LANDFILL COMPANY, LP

REVISIONS

NO.	DATE	DESCRIPTION

MAJOR PERMIT AMENDMENT POST PROJECT DRAINAGE

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

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DRAWING IIIF-A-26

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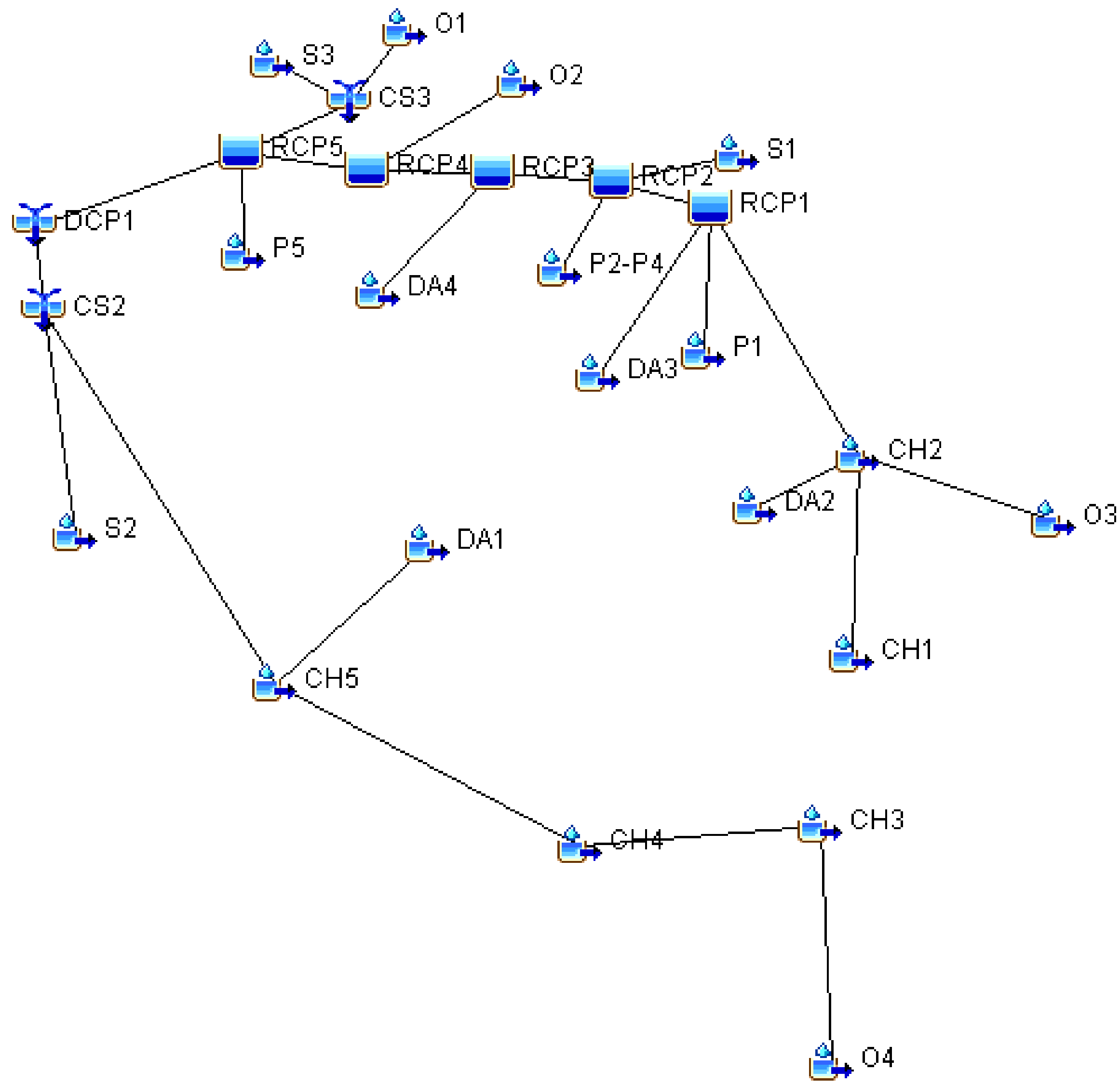
O:\0771\356\EXPANSION 2022\PART III\IIIF\IIIF-A\IIIF-A-26 DRAINAGE AREAS.dwg, jwilson, 1:2

HEC-HMS OUTPUT – POST-DEVELOPMENT
25-YEAR, 24-HOUR STORM EVENT


O1
S3
CS3
O2
RCP5
RCP4
RCP3
RCP2
S1
RCP1
DCP1
P5
DA4
P2-P4
CS2
DA3
P1
CH2
DA2
O3
S2
DA1
CH1
CH5
CH4
CH3
O4

Project: Post_Project_Condition
Simulation Run: 25-Year Storm
Simulation Start: 29 December 2020, 01:00
Simulation End: 31 December 2020, 23:00

HMS Version: 4.9
Executed: 25 September 2022, 18:00

Global Parameter Summary - Subbasin

	Area (MIē)
Element Name	Area (MIē)
Ch3	0.01
O4	0.01
Da1	0.06
Ch4	0.01
Ch5	0.01
S2	0.06
Da2	0.05
O3	0.03
Ch1	0
Ch2	0.01
Da3	0.01
P1	0
S1	0.02
P2 - P4	0.01
Da4	0.05
O2	0.01
O1	0.01
S3	0
P5	0.01

Downstream

Element Name	Downstream
Ch3	Ch4
O4	Ch3
Da1	Ch5
Ch4	Ch5
Ch5	Cs2
S2	Cs2
Da2	Ch2
O3	Ch2
Ch1	Ch2
Ch2	Rcp1
Da3	Rcp1
P1	Rcp1
S1	Rcp2
P2 - P4	Rcp2
Da4	Rcp3
O2	Rcp4
O1	Cs3
S3	Cs3
P5	Rcp5

LossRate 1

Element Name	Percent Impervious Area	Curve Number
Ch3	0	84
Da1	0	86
Ch4	0	84
Ch5	0	84
Da2	0	86
Ch1	0	84
Ch2	0	84
Da3	0	86
Da4	0	86

Transform: Kinematic Wave

Element Name	Transform
Ch3	Kinematic Wave
Da1	Kinematic Wave
Ch4	Kinematic Wave
Ch5	Kinematic Wave
Da2	Kinematic Wave
Ch1	Kinematic Wave
Ch2	Kinematic Wave
Da3	Kinematic Wave
Da4	Kinematic Wave

Transform: Snyder

Element Name	Snyder Method	Snyder Tp	Snyder Cp
O4	Standard	0.14	0.64
S2	Standard	0.79	0.67
O3	Standard	0.14	0.62
S1	Standard	0.2	0.67
O2	Standard	0.2	0.69
O1	Standard	0.23	0.69
S3	Standard	0.23	0.72

Transform: Scs

Element Name	Lag	Unitgraph Type
P1	0.1	Standard
P2 - P4	0.1	Standard
P5	0.1	Standard

Global Results Summary

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (IN)
Ch3	0.02	44.94	29Dec2020, 13:10	5.17
O4	0.01	30.55	29Dec2020, 13:15	5.3
Da1	0.06	212.14	29Dec2020, 13:10	4.95
Ch4	0.02	69.58	29Dec2020, 13:10	5.04
Ch5	0.08	300.85	29Dec2020, 13:05	5
S2	0.06	74.49	29Dec2020, 13:50	5.3
Da2	0.05	188.8	29Dec2020, 13:10	4.62
O3	0.03	83.69	29Dec2020, 13:15	5.3
Ch1	0	14.59	29Dec2020, 13:10	5.1
Ch2	0.09	294.11	29Dec2020, 13:10	4.89
Da3	0.01	53.57	29Dec2020, 13:10	5.23

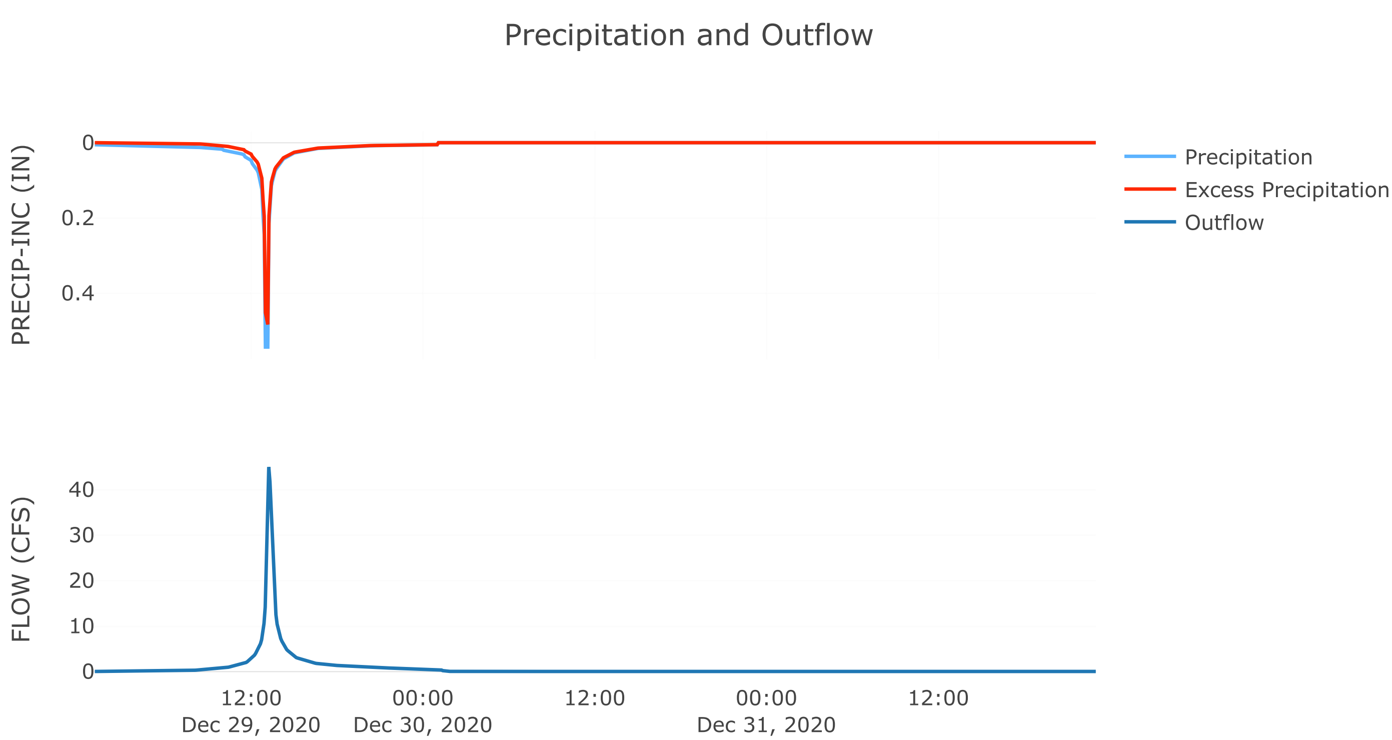
P1	0	14.23	29Dec2020, 13:10	7.05
Rcp1	0.1	119.41	29Dec2020, 13:30	5.01
S1	0.02	43.57	29Dec2020, 13:15	5.3
P2 - P4	0.01	24.7	29Dec2020, 13:10	7.05
Rcp2	0.13	152.44	29Dec2020, 13:30	5.15
Da4	0.05	197.15	29Dec2020, 13:10	5.11
Rcp3	0.18	283.49	29Dec2020, 13:10	5.14
O2	0.01	20.32	29Dec2020, 13:15	5.3
Rcp4	0.18	289.6	29Dec2020, 13:15	5.15
O1	0.01	21.48	29Dec2020, 13:20	5.3
S3	0	8.8	29Dec2020, 13:15	5.3
Cs3	0.01	30.28	29Dec2020, 13:20	5.3
P5	0.01	22.19	29Dec2020, 13:10	7.05
Rcp5	0.2	266.33	29Dec2020, 13:20	5.22
Cs2	0.14	322.84	29Dec2020, 13:05	5.12
Dcp1	0.34	424.54	29Dec2020, 13:20	5.18

Subbasin: CH3

Area (MIē) : 0.01
Downstream : Ch4
Transform : Kinematic Wave

LossRate 1: Scs	
Percent Impervious Area	0
Curve Number	84

Results: CH3	
Peak Discharge (CFS)	44.94
Time of Peak Discharge	29Dec2020, 13:10
Volume (IN)	5.17
Precipitation Volume (AC - FT)	5.85
Loss Volume (AC - FT)	1.52
Excess Volume (AC - FT)	4.33
Direct Runoff Volume (AC - FT)	4.21
Baseflow Volume (AC - FT)	0



Subbasin: O4

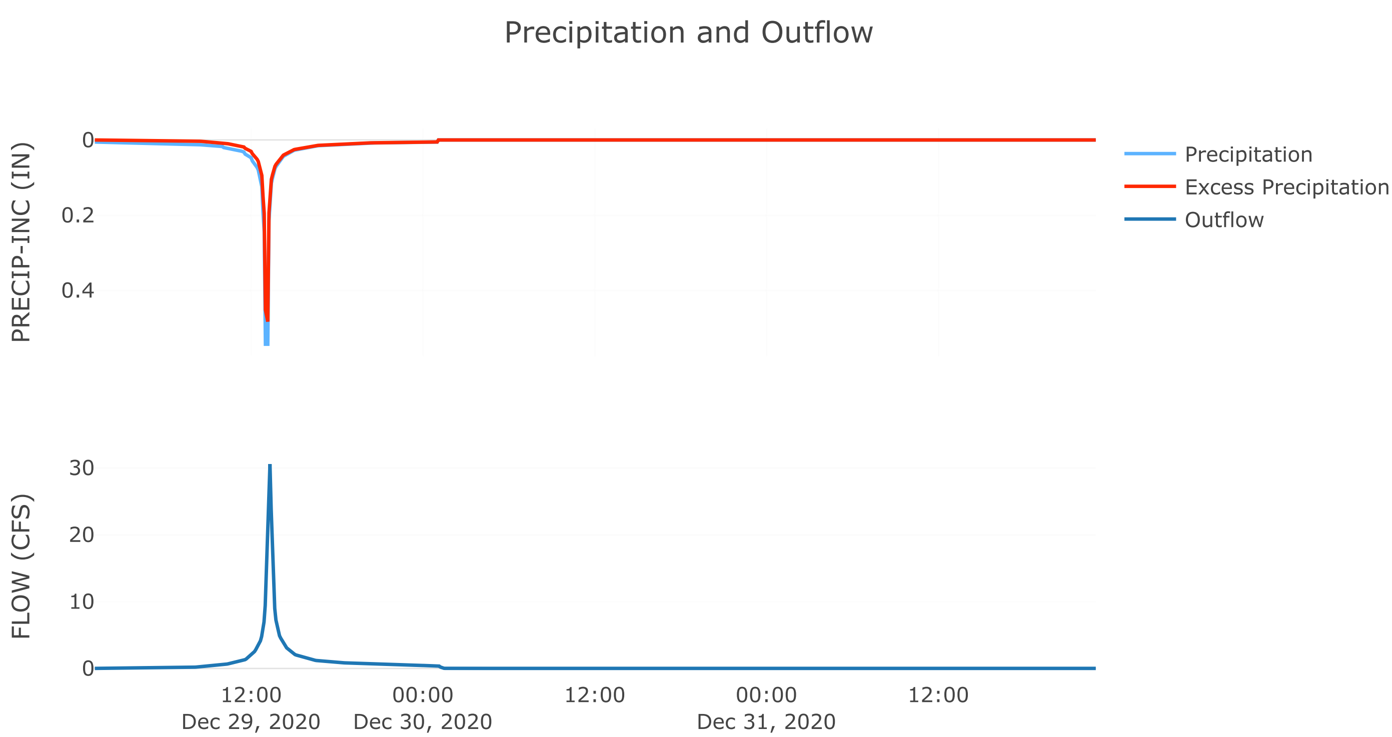
Area (MIē) : 0.01
Downstream : Ch3

Transform: Snyder

Snyder Method	Standard
Snyder Tp	0.14
Snyder Cp	0.64

Results: O4

Peak Discharge (CFS)	30.55
Time of Peak Discharge	29Dec2020, 13:15
Volume (IN)	5.3
Precipitation Volume (AC - FT)	3.75
Loss Volume (AC - FT)	0.98
Excess Volume (AC - FT)	2.77
Direct Runoff Volume (AC - FT)	2.77
Baseflow Volume (AC - FT)	0



Subbasin: DA1

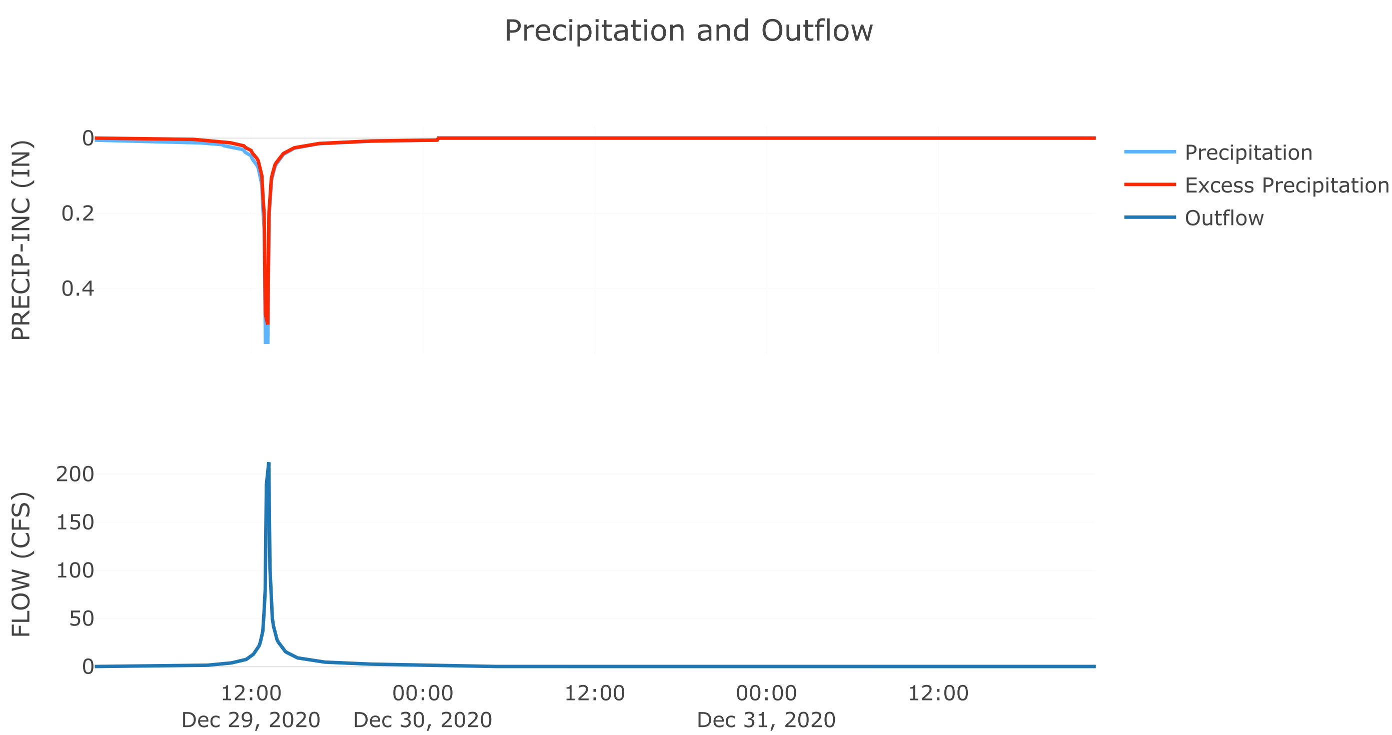
Area (MIē) : 0.06
Downstream : Ch5
Transform : Kinematic Wave

LossRate 1: Scs

Percent Impervious Area	0
Curve Number	86

Results: DA1

Peak Discharge (CFS)	212.14
Time of Peak Discharge	29Dec2020, 13:10
Volume (IN)	4.95
Precipitation Volume (AC - FT)	21.15
Loss Volume (AC - FT)	4.84
Excess Volume (AC - FT)	16.31
Direct Runoff Volume (AC - FT)	14.6
Baseflow Volume (AC - FT)	0



Subbasin: CH4

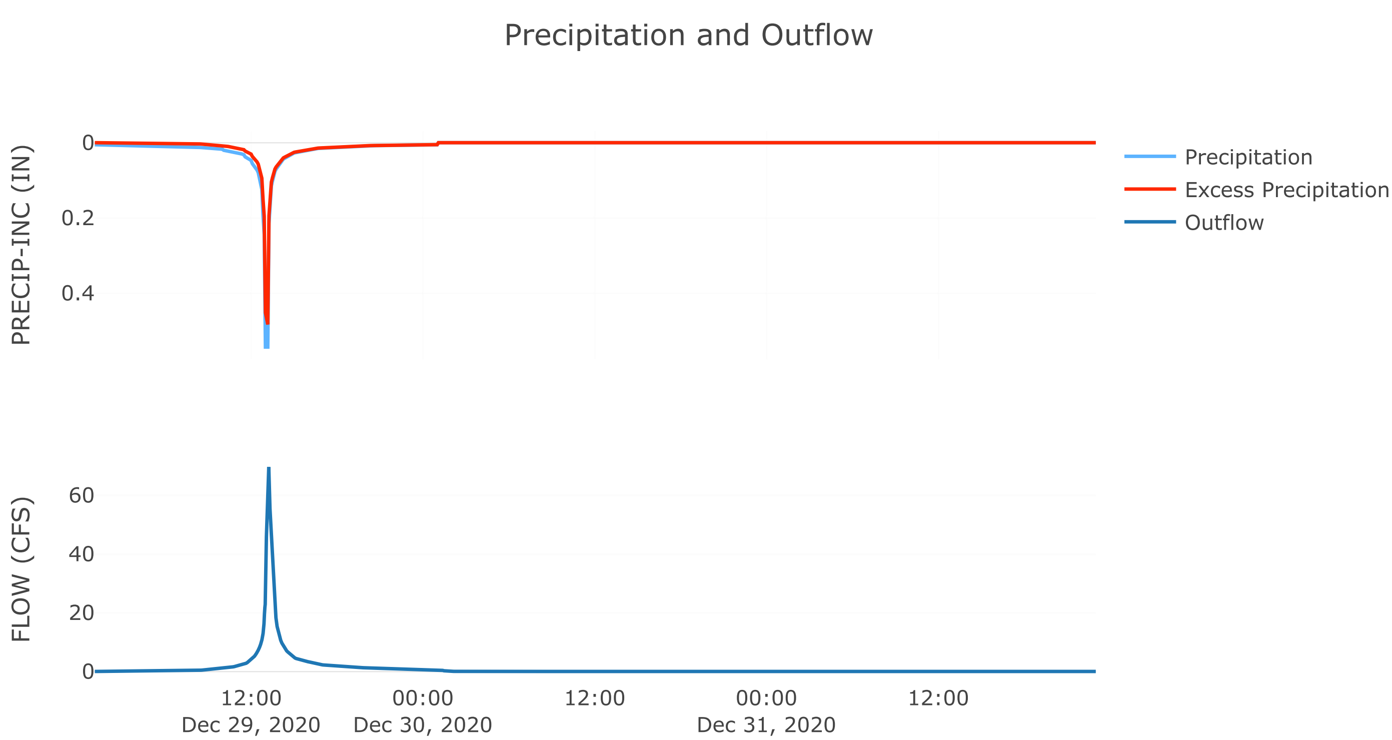
Area (MIē) : 0.01
Downstream : Ch5
Transform : Kinematic Wave

LossRate 1: Scs

Percent Impervious Area	0
Curve Number	84

Results: CH4

Peak Discharge (CFS)	69.58
Time of Peak Discharge	29Dec2020, 13:10
Volume (IN)	5.04
Precipitation Volume (AC - FT)	8.72
Loss Volume (AC - FT)	2.27
Excess Volume (AC - FT)	6.45
Direct Runoff Volume (AC - FT)	6.13
Baseflow Volume (AC - FT)	0



Subbasin: CH5

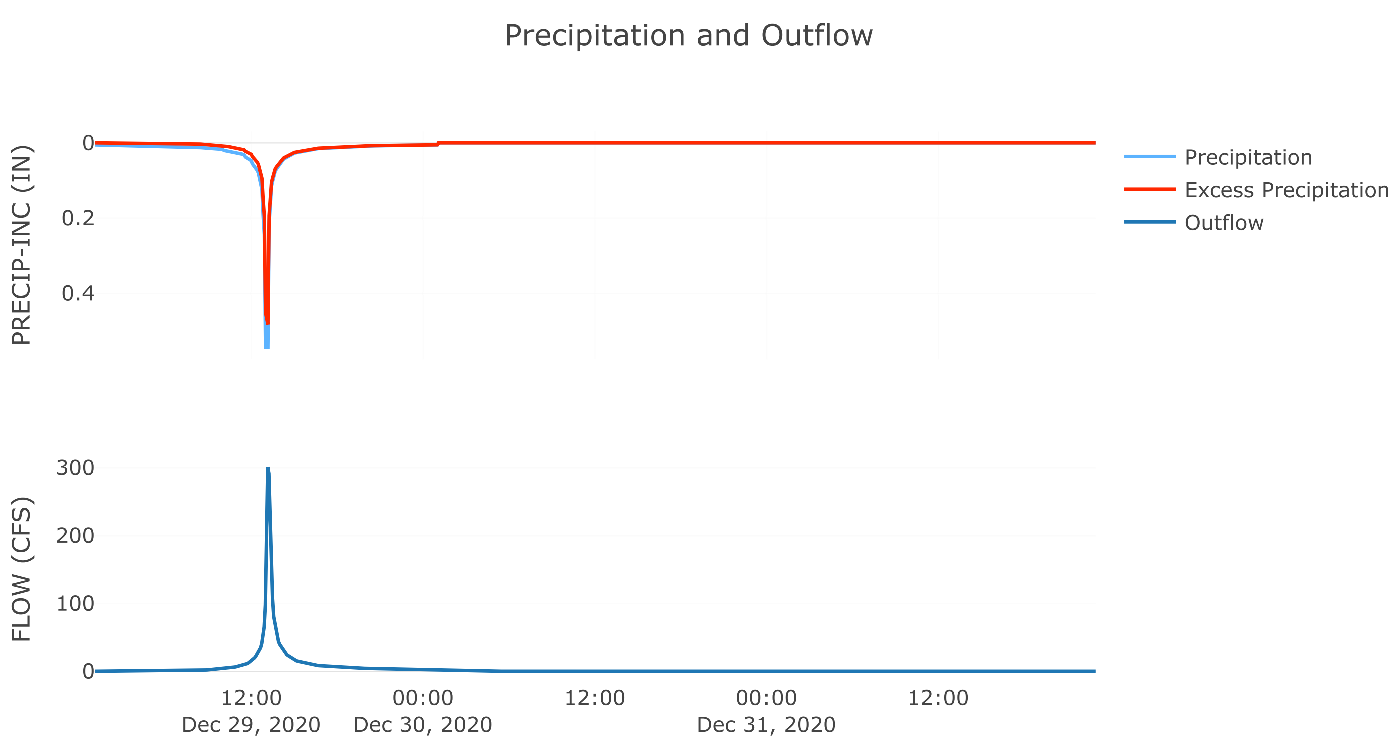
Area (MIē) : 0.01
Downstream : Cs2
Transform : Kinematic Wave

LossRate 1: Scs

Percent Impervious Area	0
Curve Number	84

Results: CH5

Peak Discharge (CFS)	300.85
Time of Peak Discharge	29Dec2020, 13:05
Volume (IN)	5
Precipitation Volume (AC - FT)	32.2
Loss Volume (AC - FT)	8.39
Excess Volume (AC - FT)	23.81
Direct Runoff Volume (AC - FT)	22.44
Baseflow Volume (AC - FT)	0

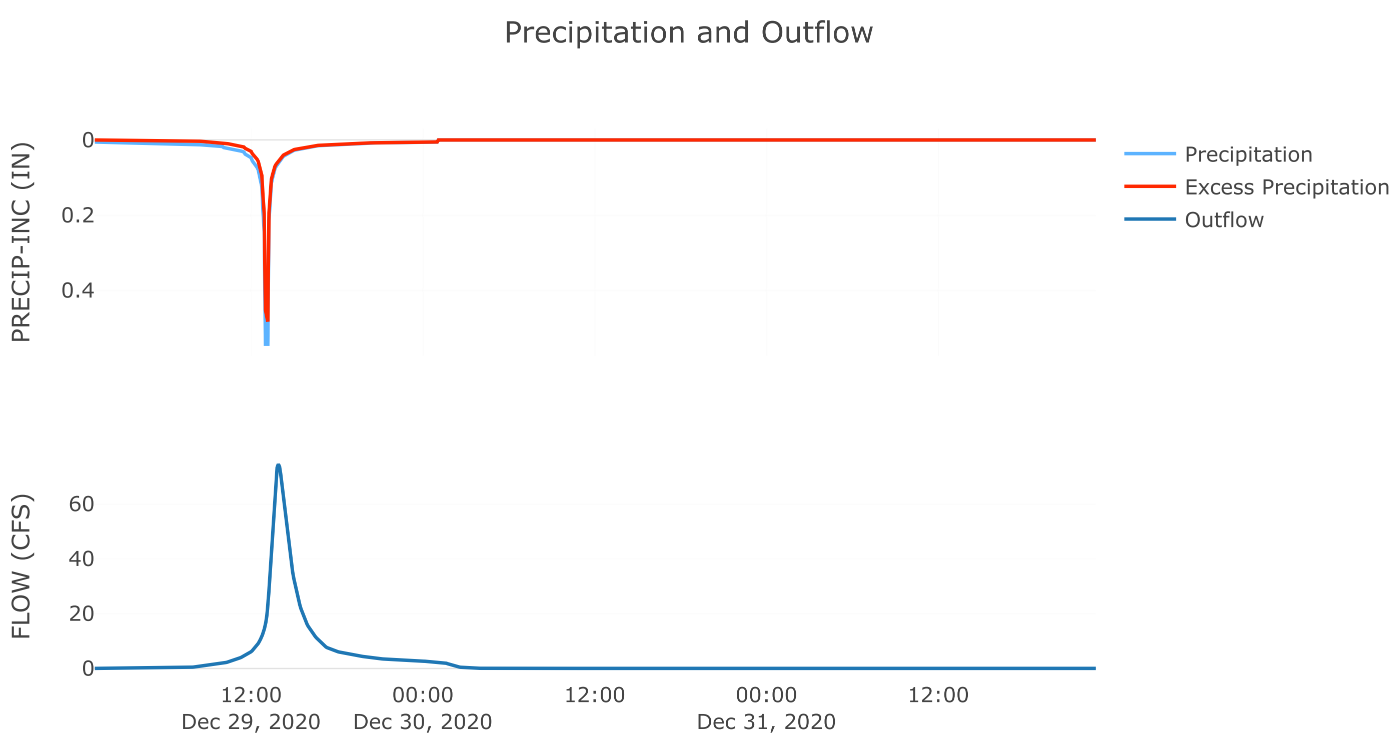


Subbasin: S2

Area (MIē) : 0.06
Downstream : Cs2

Transform: Snyder	
Snyder Method	Standard
Snyder Tp	0.79
Snyder Cp	0.67

Results: S2	
Peak Discharge (CFS)	74.49
Time of Peak Discharge	29Dec2020, 13:50
Volume (IN)	5.3
Precipitation Volume (AC - FT)	21.15
Loss Volume (AC - FT)	5.51
Excess Volume (AC - FT)	15.64
Direct Runoff Volume (AC - FT)	15.64
Baseflow Volume (AC - FT)	0

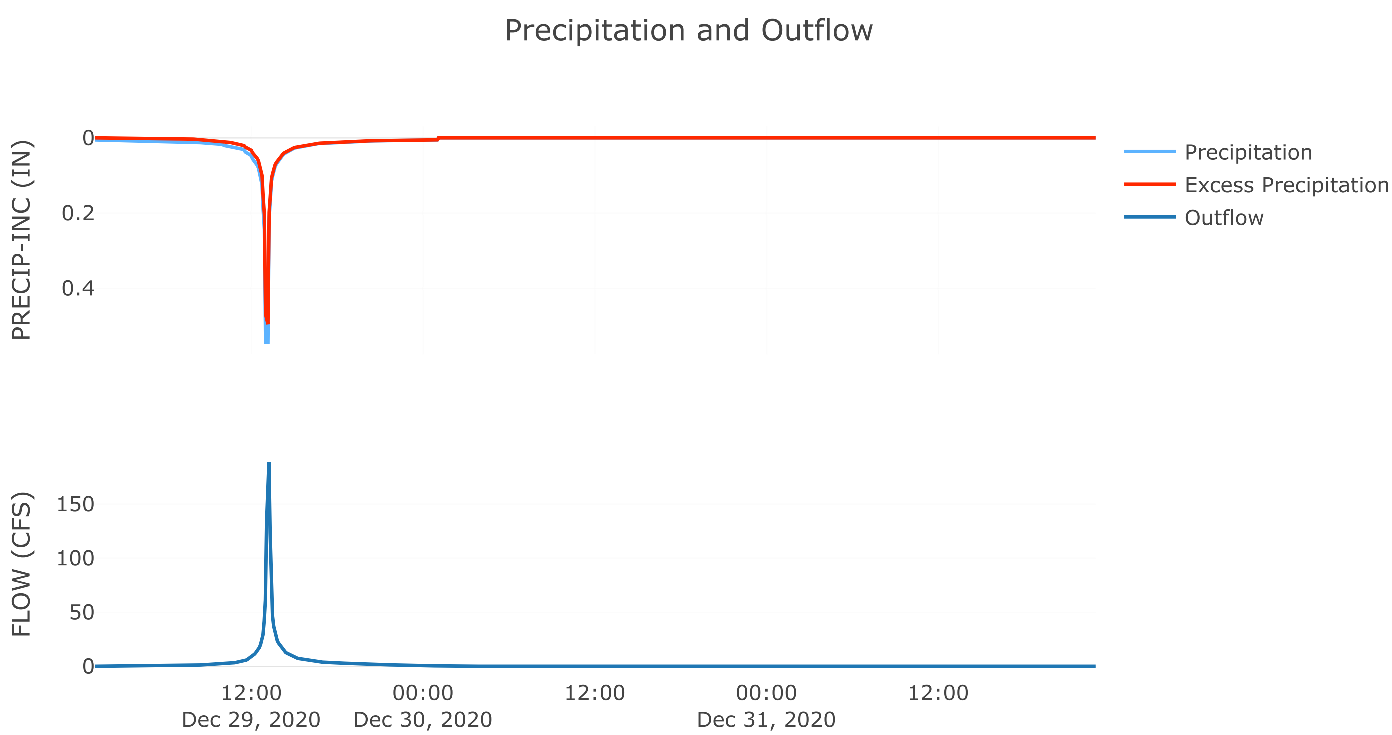


Subbasin: DA2

Area (MIē) : 0.05
Downstream : Ch2
Transform : Kinematic Wave

LossRate 1: Scs	
Percent Impervious Area	0
Curve Number	86

Results: DA2	
Peak Discharge (CFS)	188.8
Time of Peak Discharge	29Dec2020, 13:10
Volume (IN)	4.62
Precipitation Volume (AC - FT)	19.2
Loss Volume (AC - FT)	4.39
Excess Volume (AC - FT)	14.8
Direct Runoff Volume (AC - FT)	12.37
Baseflow Volume (AC - FT)	0



Subbasin: O3

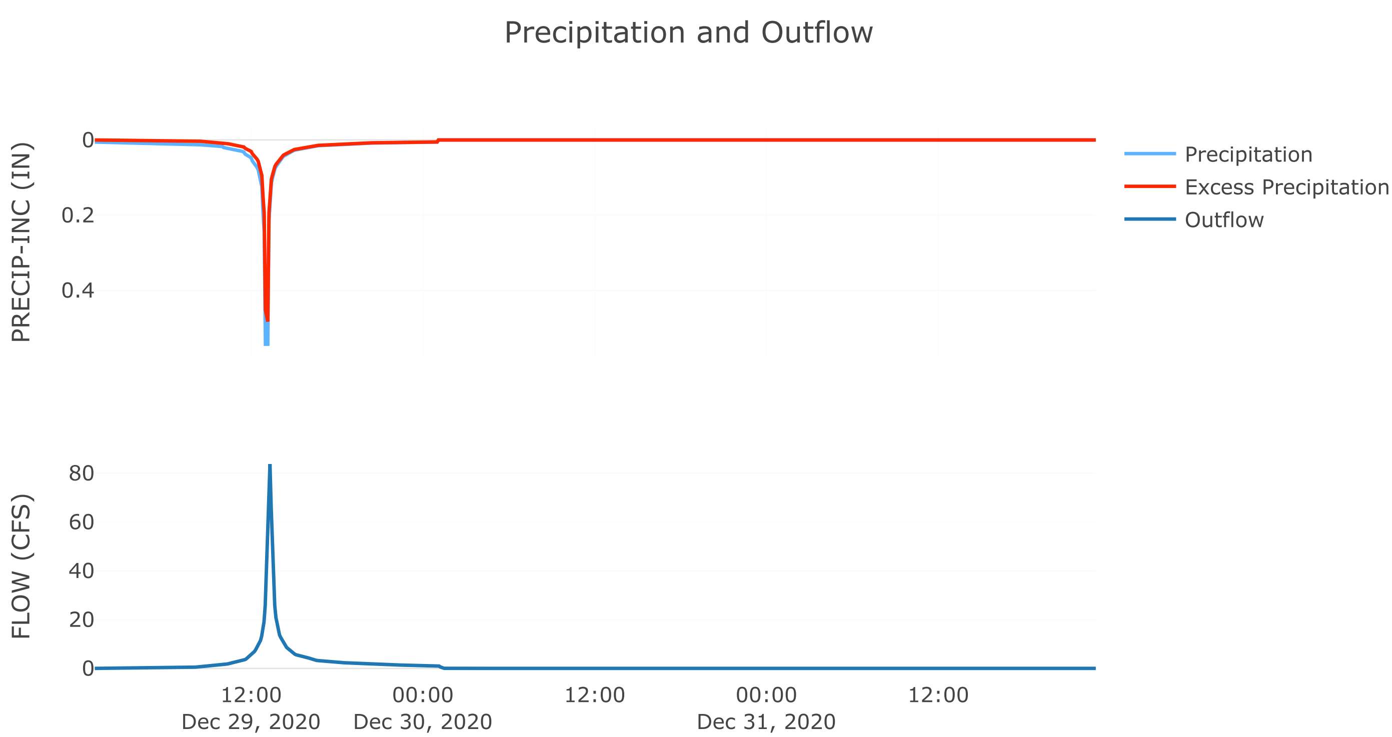
Area (MIē) : 0.03
Downstream : Ch2

Transform: Snyder

Snyder Method	Standard
Snyder Tp	0.14
Snyder Cp	0.62

Results: O3

Peak Discharge (CFS)	83.69
Time of Peak Discharge	29Dec2020, 13:15
Volume (IN)	5.3
Precipitation Volume (AC - FT)	10.4
Loss Volume (AC - FT)	2.71
Excess Volume (AC - FT)	7.69
Direct Runoff Volume (AC - FT)	7.69
Baseflow Volume (AC - FT)	0



Subbasin: CH1

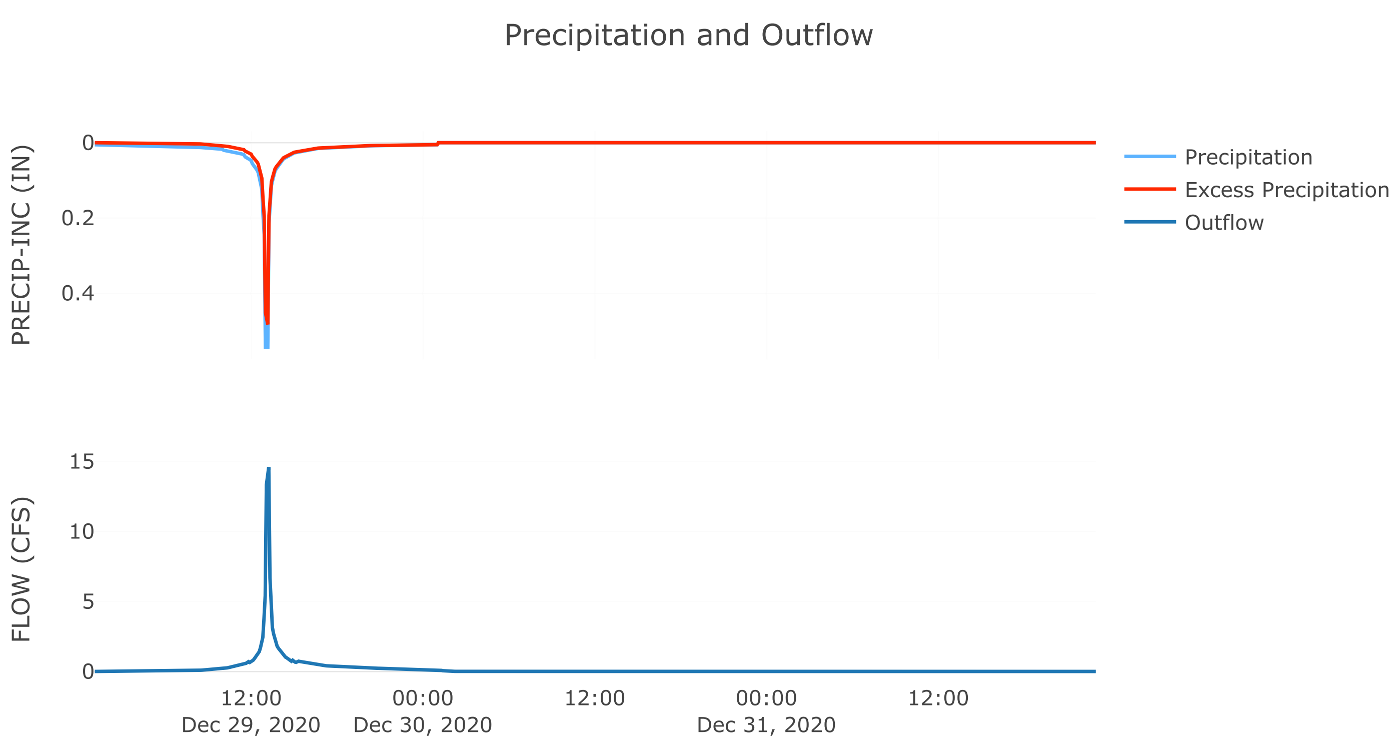
Area (MI²) : 0
Downstream : Ch2
Transform : Kinematic Wave

LossRate 1: Scs

Percent Impervious Area	0
Curve Number	84

Results: CH1

Peak Discharge (CFS)	14.59
Time of Peak Discharge	29Dec2020, 13:10
Volume (IN)	5.1
Precipitation Volume (AC - FT)	1.49
Loss Volume (AC - FT)	0.39
Excess Volume (AC - FT)	1.1
Direct Runoff Volume (AC - FT)	1.06
Baseflow Volume (AC - FT)	0



Subbasin: CH2

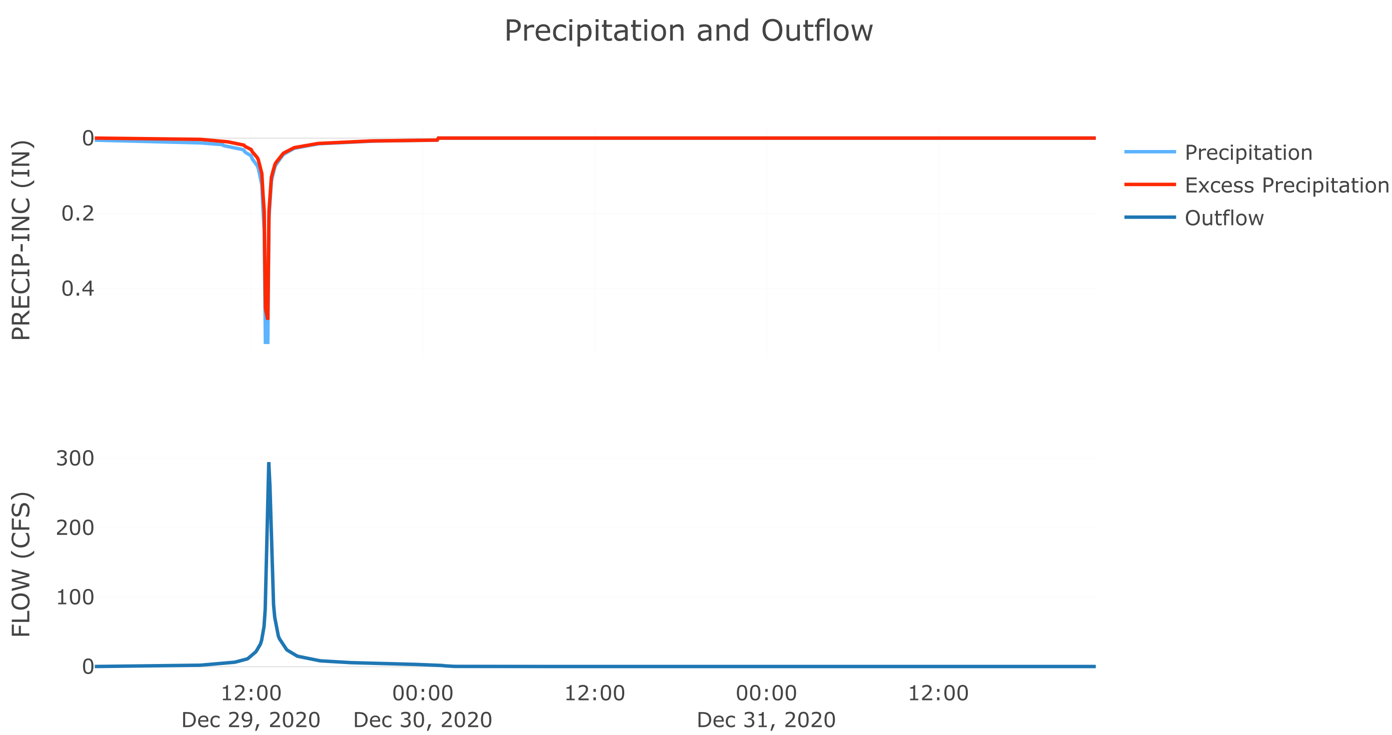
Area (MIē) : 0.01
Downstream : Rcp1
Transform : Kinematic Wave

LossRate 1: Scs

Percent Impervious Area	0
Curve Number	84

Results: CH2

Peak Discharge (CFS)	294.11
Time of Peak Discharge	29Dec2020, 13:10
Volume (IN)	4.89
Precipitation Volume (AC - FT)	33.08
Loss Volume (AC - FT)	8.62
Excess Volume (AC - FT)	24.46
Direct Runoff Volume (AC - FT)	22.56
Baseflow Volume (AC - FT)	0



Subbasin: DA3

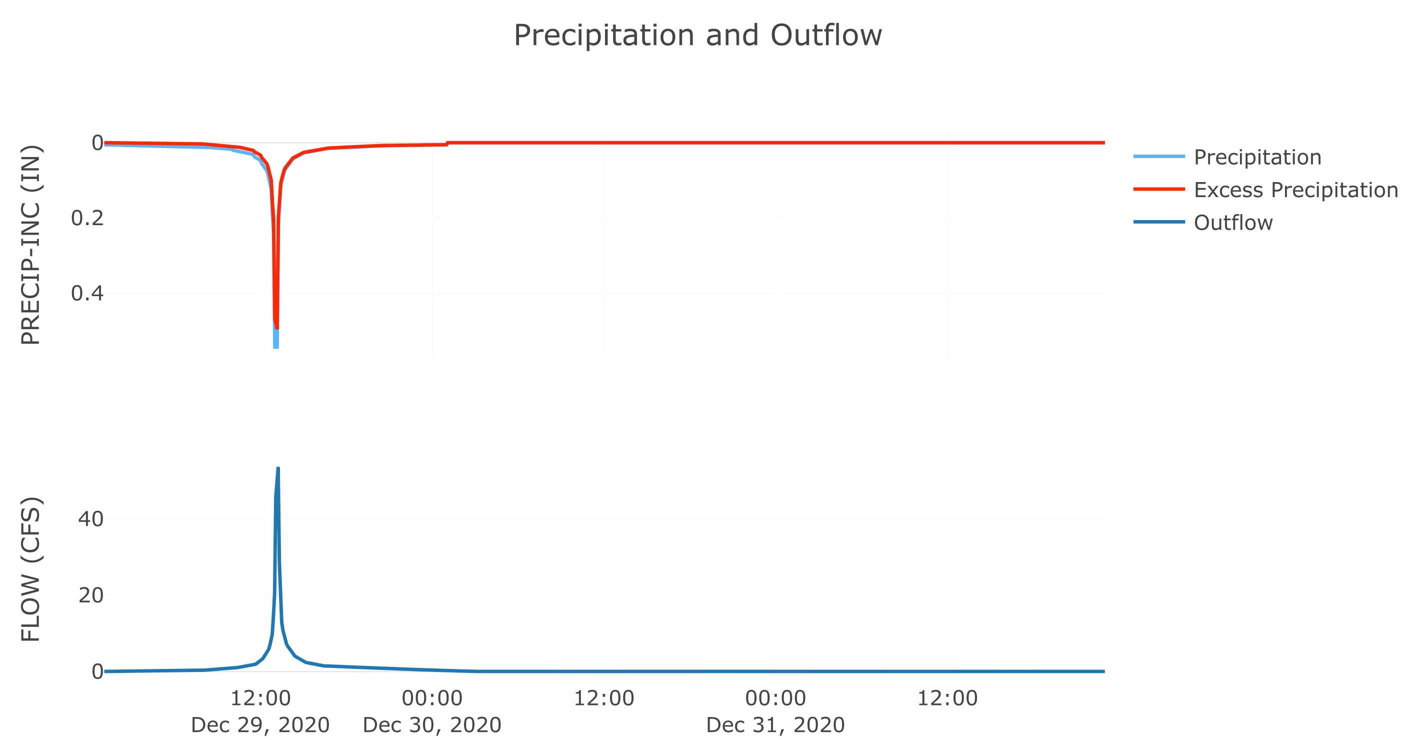
Area (MIē) : 0.01
Downstream : Rcp1
Transform : Kinematic Wave

LossRate 1: Scs

Percent Impervious Area	0
Curve Number	86

Results: DA3

Peak Discharge (CFS)	53.57
Time of Peak Discharge	29Dec2020, 13:10
Volume (IN)	5.23
Precipitation Volume (AC - FT)	5.35
Loss Volume (AC - FT)	1.23
Excess Volume (AC - FT)	4.13
Direct Runoff Volume (AC - FT)	3.9
Baseflow Volume (AC - FT)	0



Subbasin: P1

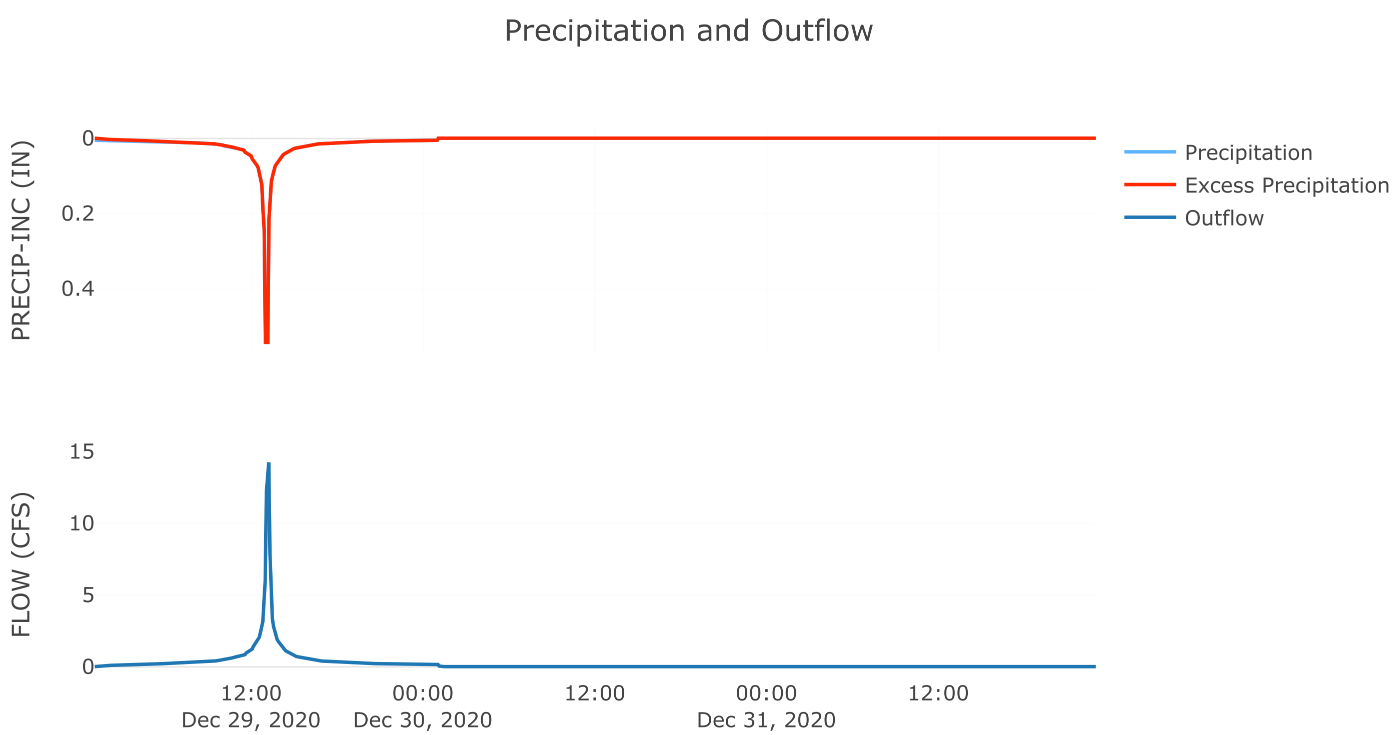
Area (MI²) : 0
Downstream : Rcp1

Transform: Scs

Lag	0.1
Unitgraph Type	Standard

Results: P1

Peak Discharge (CFS)	14.23
Time of Peak Discharge	29Dec2020, 13:10
Volume (IN)	7.05
Precipitation Volume (AC - FT)	1.3
Loss Volume (AC - FT)	0.02
Excess Volume (AC - FT)	1.28
Direct Runoff Volume (AC - FT)	1.28
Baseflow Volume (AC - FT)	0

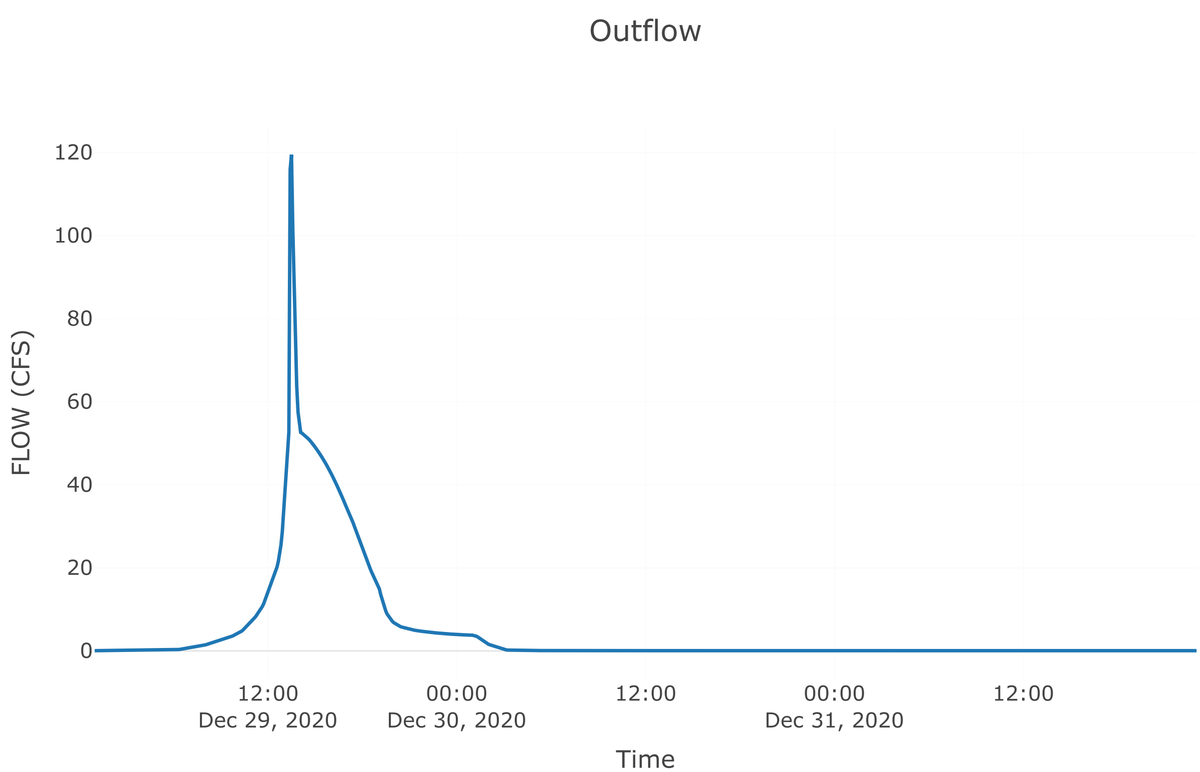


Reservoir: RCP1

Downstream : Rcp2

Results: RCP1

Peak Discharge (CFS)	119.41
Time of Peak Discharge	29Dec2020, 13:30
Volume (IN)	5.01
Peak Inflow (CFS)	361.9
Time of Peak Inflow	29Dec2020, 13:10
Inflow Volume (AC - FT)	27.74
Maximum Storage (AC - FT)	10.64
Peak Elevation (FT)	638.23
Discharge Volume (AC - FT)	27.77



Subbasin: S1

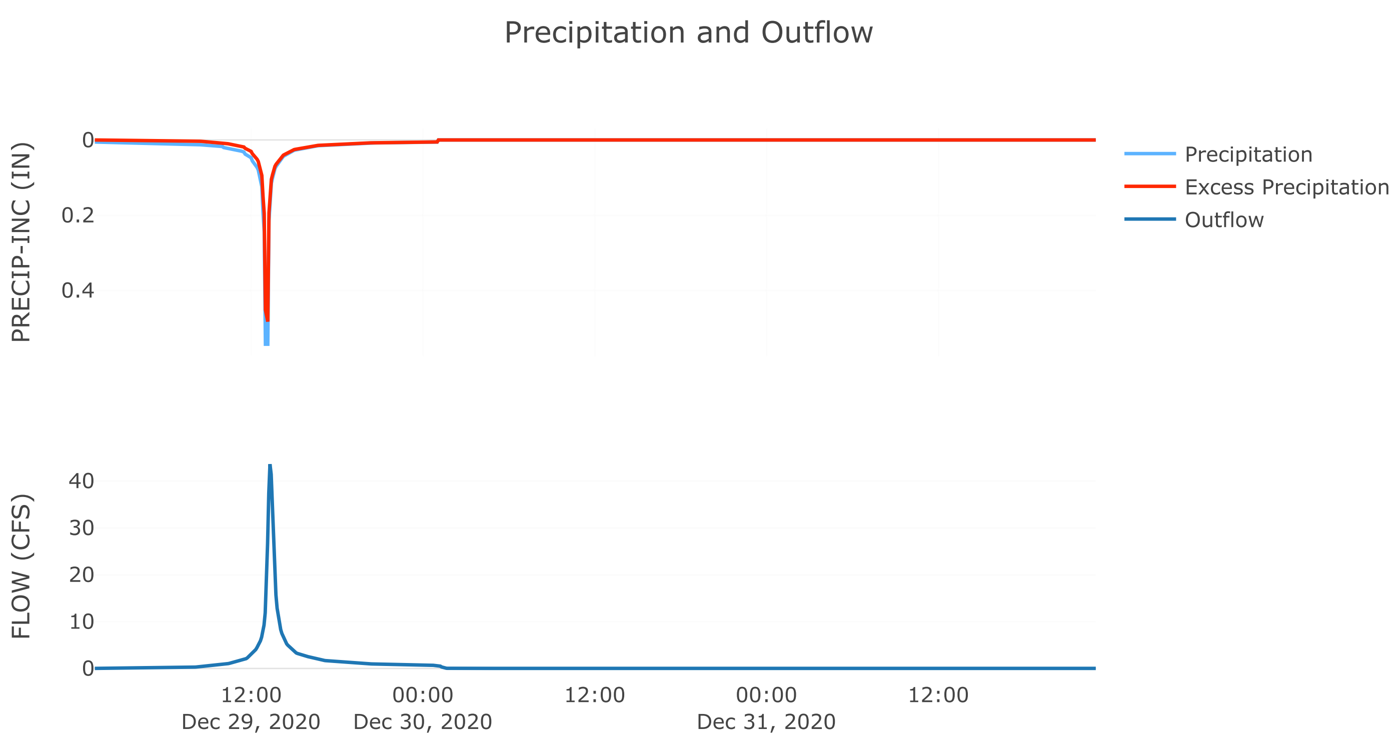
Area (MIē) : 0.02
Downstream : Rcp2

Transform: Snyder

Snyder Method	Standard
Snyder Tp	0.2
Snyder Cp	0.67

Results: S1

Peak Discharge (CFS)	43.57
Time of Peak Discharge	29Dec2020, 13:15
Volume (IN)	5.3
Precipitation Volume (AC - FT)	5.93
Loss Volume (AC - FT)	1.54
Excess Volume (AC - FT)	4.38
Direct Runoff Volume (AC - FT)	4.38
Baseflow Volume (AC - FT)	0



Subbasin: P2-P4

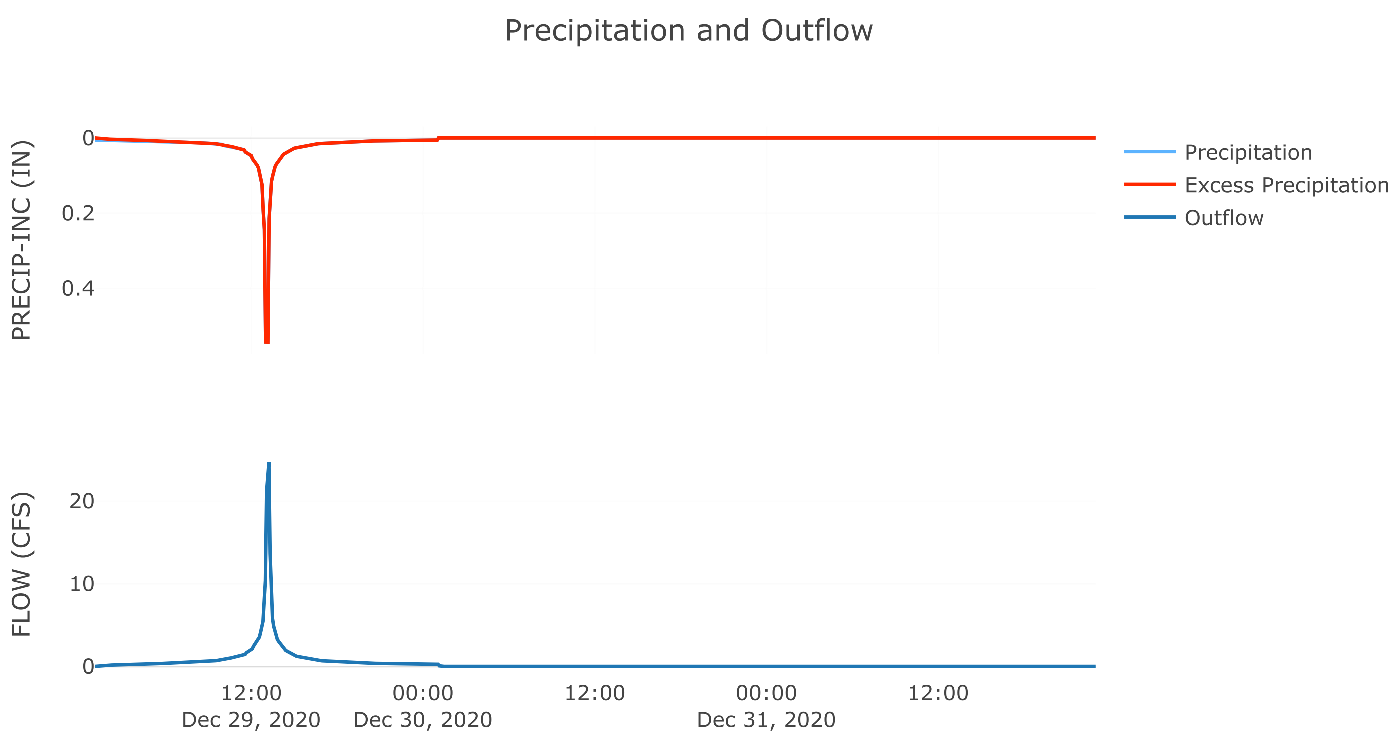
Area (MIē) : 0.01
Downstream : Rcp2

Transform: Scs

Lag	0.1
Unitgraph Type	Standard

Results: P2-P4

Peak Discharge (CFS)	24.7
Time of Peak Discharge	29Dec2020, 13:10
Volume (IN)	7.05
Precipitation Volume (AC - FT)	2.26
Loss Volume (AC - FT)	0.04
Excess Volume (AC - FT)	2.22
Direct Runoff Volume (AC - FT)	2.22
Baseflow Volume (AC - FT)	0

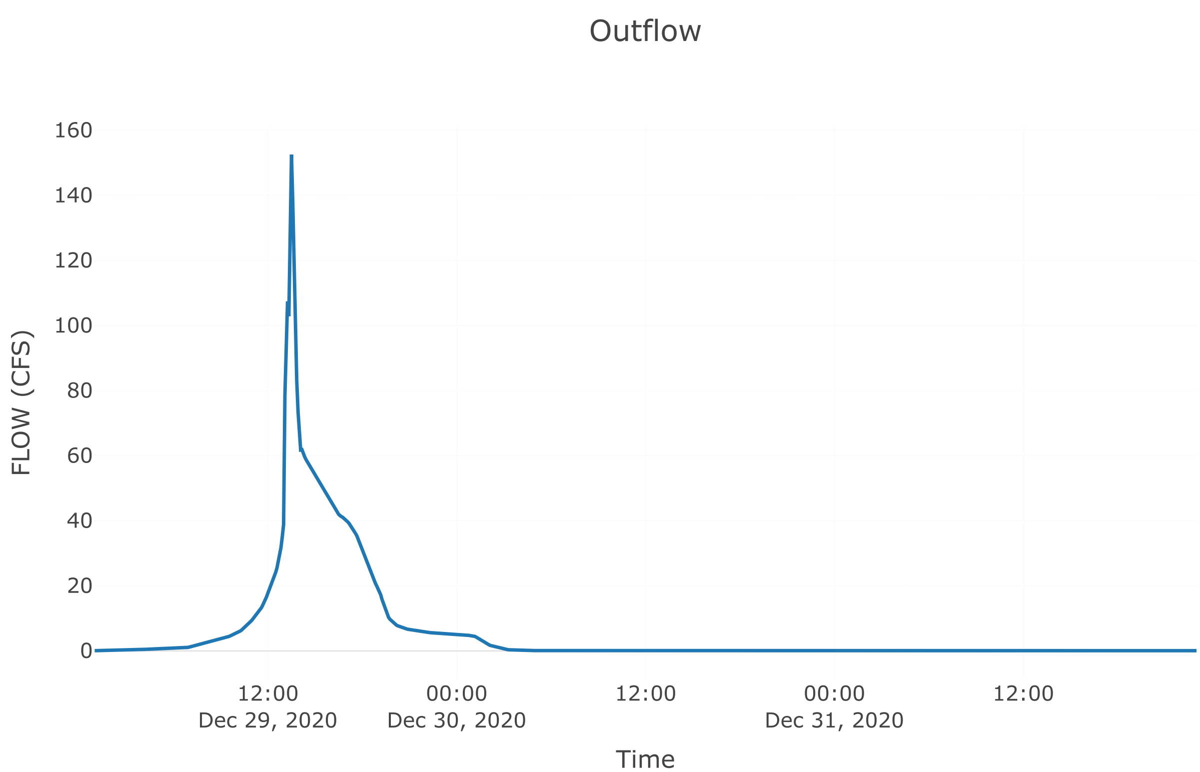


Reservoir: RCP2

Downstream : Rcp3

Results: RCP2

Peak Discharge (CFS)	152.44
Time of Peak Discharge	29Dec2020, 13:30
Volume (IN)	5.15
Peak Inflow (CFS)	155.15
Time of Peak Inflow	29Dec2020, 13:25
Inflow Volume (AC - FT)	34.37
Maximum Storage (AC - FT)	0.94
Peak Elevation (FT)	620.6
Discharge Volume (AC - FT)	34.42



Subbasin: DA4

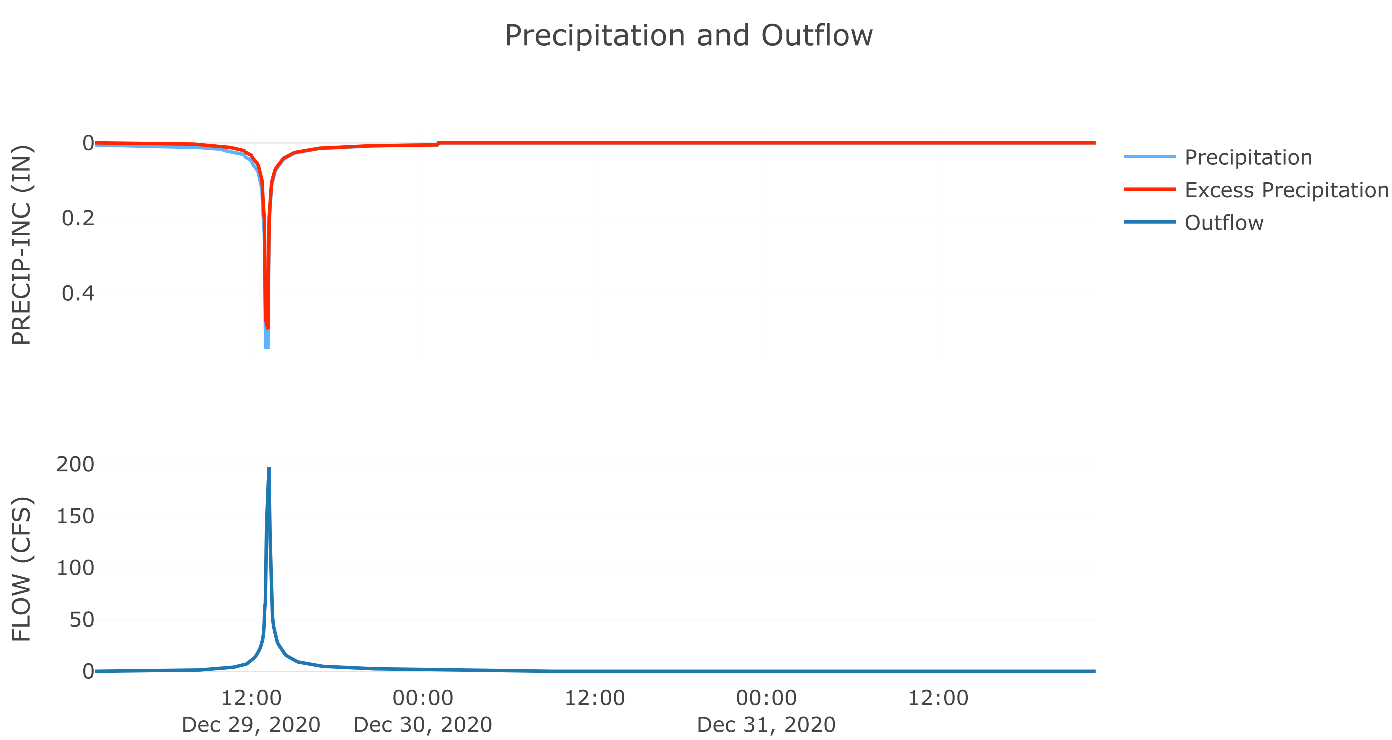
Area (MIē) : 0.05
Downstream : Rcp3
Transform : Kinematic Wave

LossRate 1: Scs

Percent Impervious Area	0
Curve Number	86

Results: DA4

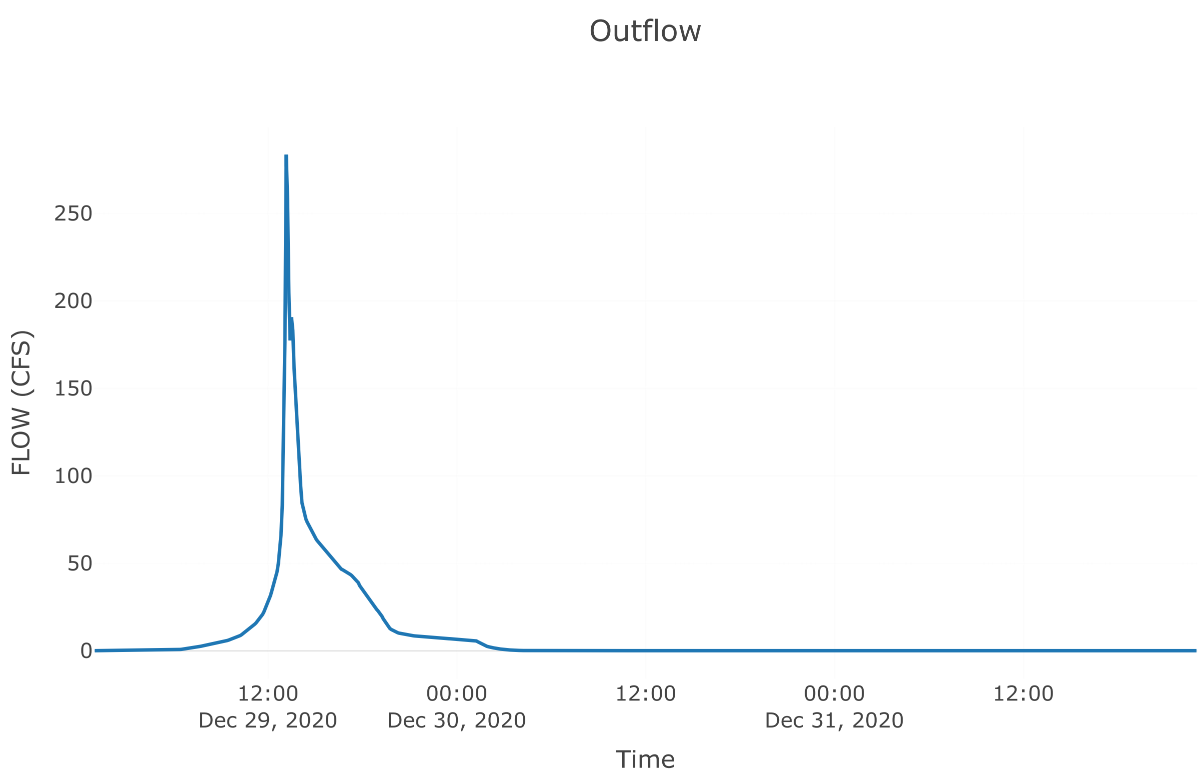
Peak Discharge (CFS)	197.15
Time of Peak Discharge	29Dec2020, 13:10
Volume (IN)	5.11
Precipitation Volume (AC - FT)	19.77
Loss Volume (AC - FT)	4.52
Excess Volume (AC - FT)	15.25
Direct Runoff Volume (AC - FT)	14.1
Baseflow Volume (AC - FT)	0



Reservoir: RCP3

Downstream : Rcp4

Results: RCP3	
Peak Discharge (CFS)	283.49
Time of Peak Discharge	29Dec2020, 13:10
Volume (IN)	5.14
Peak Inflow (CFS)	295.95
Time of Peak Inflow	29Dec2020, 13:10
Inflow Volume (AC - FT)	48.52
Maximum Storage (AC - FT)	1.39
Peak Elevation (FT)	610.65
Discharge Volume (AC - FT)	48.5



Subbasin: O2

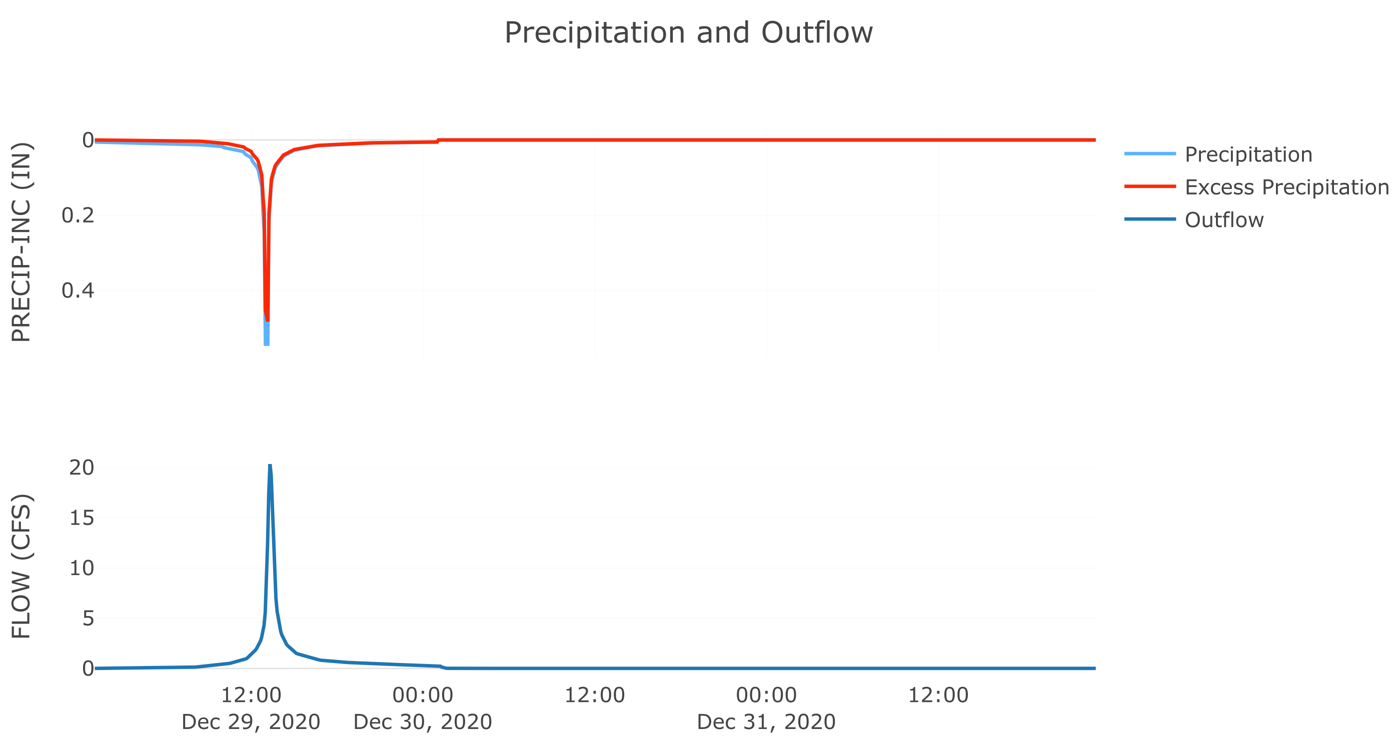
Area (MIē) : 0.01
Downstream : Rcp4

Transform: Snyder

Snyder Method	Standard
Snyder Tp	0.2
Snyder Cp	0.69

Results: O2

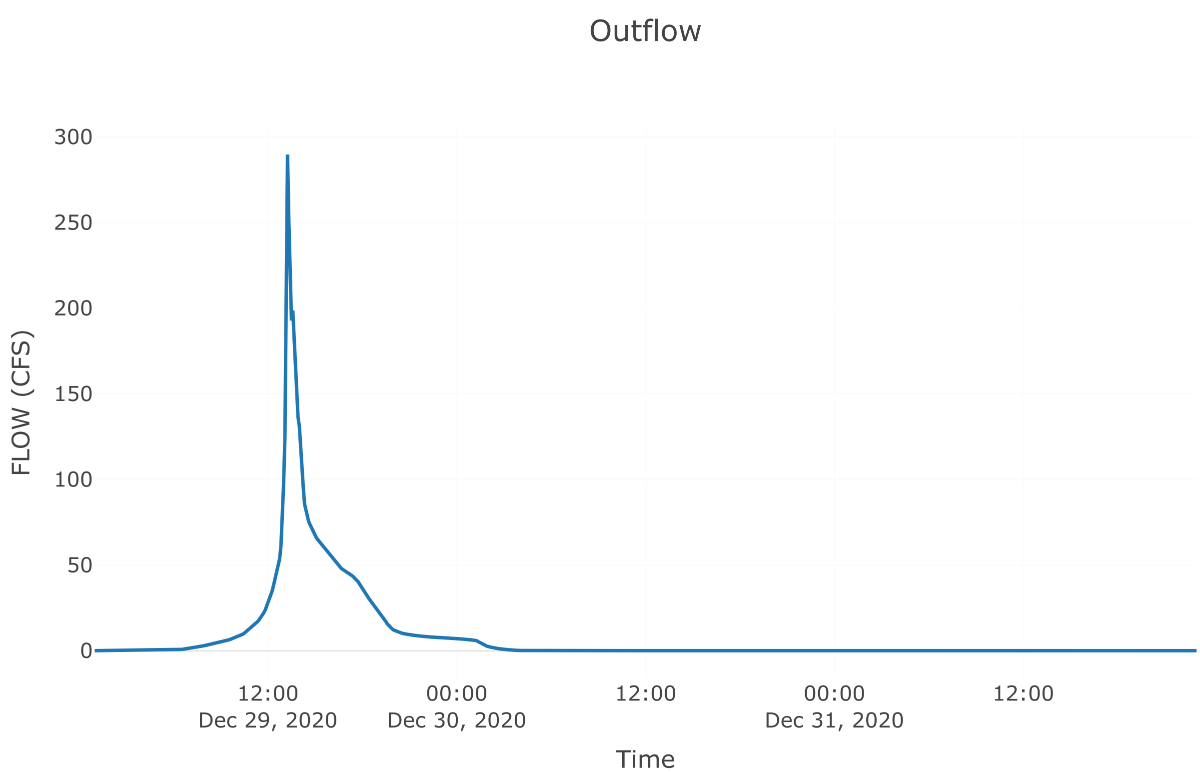
Peak Discharge (CFS)	20.32
Time of Peak Discharge	29Dec2020, 13:15
Volume (IN)	5.3
Precipitation Volume (AC - FT)	2.72
Loss Volume (AC - FT)	0.71
Excess Volume (AC - FT)	2.01
Direct Runoff Volume (AC - FT)	2.01
Baseflow Volume (AC - FT)	0



Reservoir: RCP4

Downstream : Rcp5

Results: RCP4	
Peak Discharge (CFS)	289.6
Time of Peak Discharge	29Dec2020, 13:15
Volume (IN)	5.15
Peak Inflow (CFS)	300.93
Time of Peak Inflow	29Dec2020, 13:10
Inflow Volume (AC - FT)	50.51
Maximum Storage (AC - FT)	1.2
Peak Elevation (FT)	600.8
Discharge Volume (AC - FT)	50.6

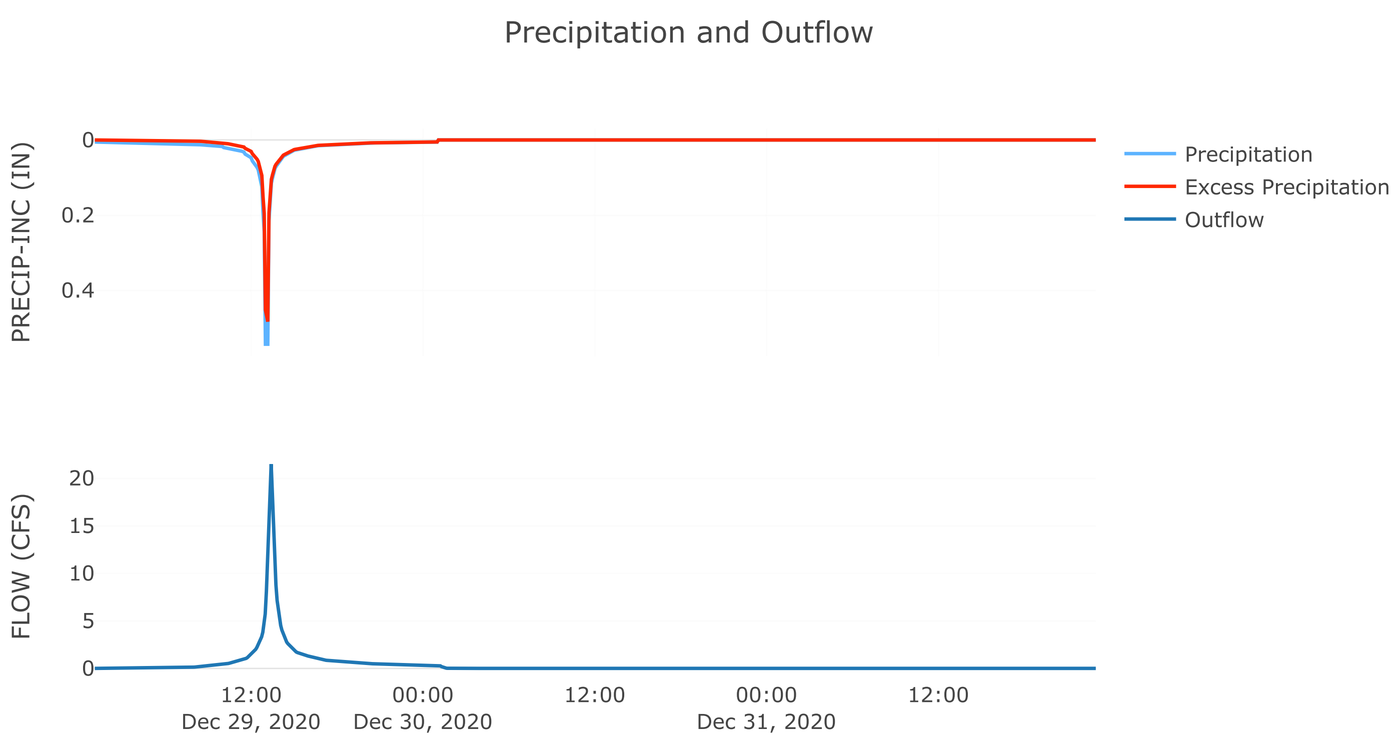


Subbasin: O1

Area (MI²) : 0.01
Downstream : Cs3

Transform: Snyder	
Snyder Method	Standard
Snyder Tp	0.23
Snyder Cp	0.69

Results: O1	
Peak Discharge (CFS)	21.48
Time of Peak Discharge	29Dec2020, 13:20
Volume (IN)	5.3
Precipitation Volume (AC - FT)	3.06
Loss Volume (AC - FT)	0.8
Excess Volume (AC - FT)	2.26
Direct Runoff Volume (AC - FT)	2.26
Baseflow Volume (AC - FT)	0



Subbasin: S3

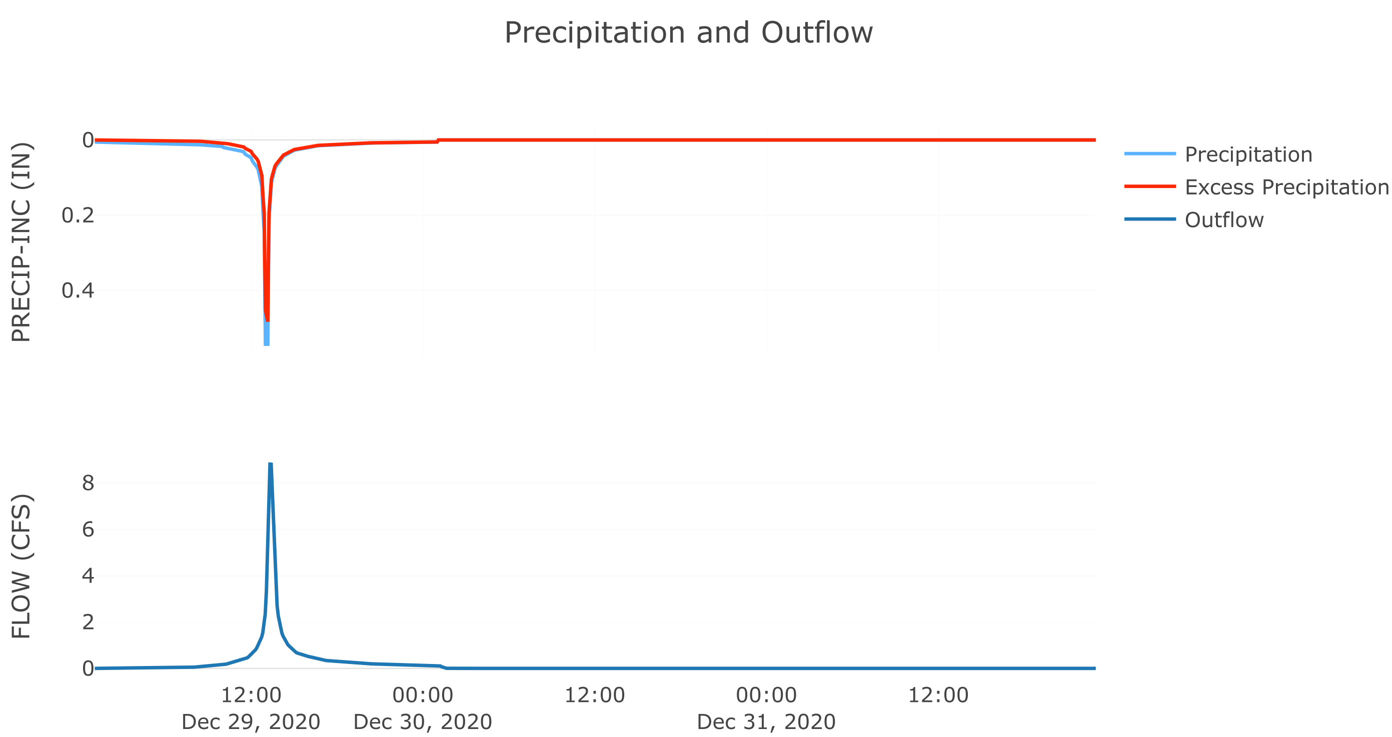
Area (MIē) : 0
Downstream : Cs3

Transform: Snyder

Snyder Method	Standard
Snyder Tp	0.23
Snyder Cp	0.72

Results: S3

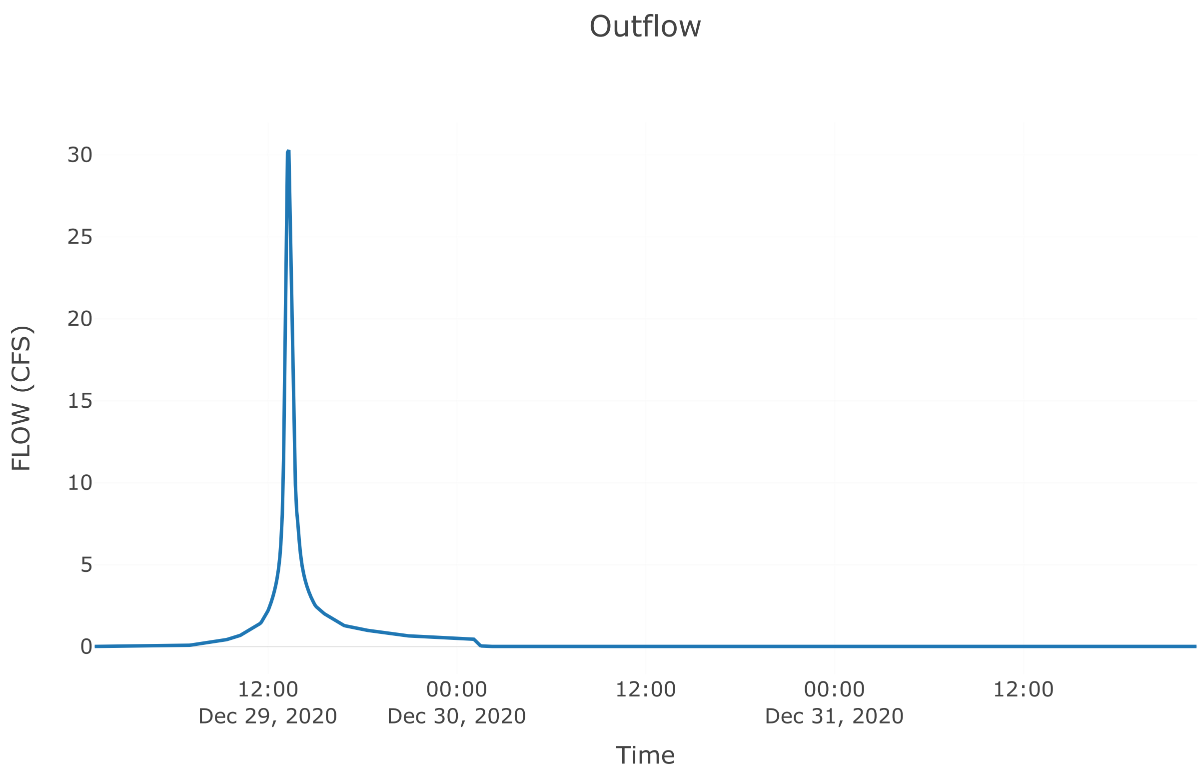
Peak Discharge (CFS)	8.8
Time of Peak Discharge	29Dec2020, 13:15
Volume (IN)	5.3
Precipitation Volume (AC - FT)	1.22
Loss Volume (AC - FT)	0.32
Excess Volume (AC - FT)	0.9
Direct Runoff Volume (AC - FT)	0.9
Baseflow Volume (AC - FT)	0



Junction: CS3

Downstream : Rcp5

Results: CS3	
Peak Discharge (CFS)	30.28
Time of Peak Discharge	29Dec2020, 13:20
Volume (IN)	5.3

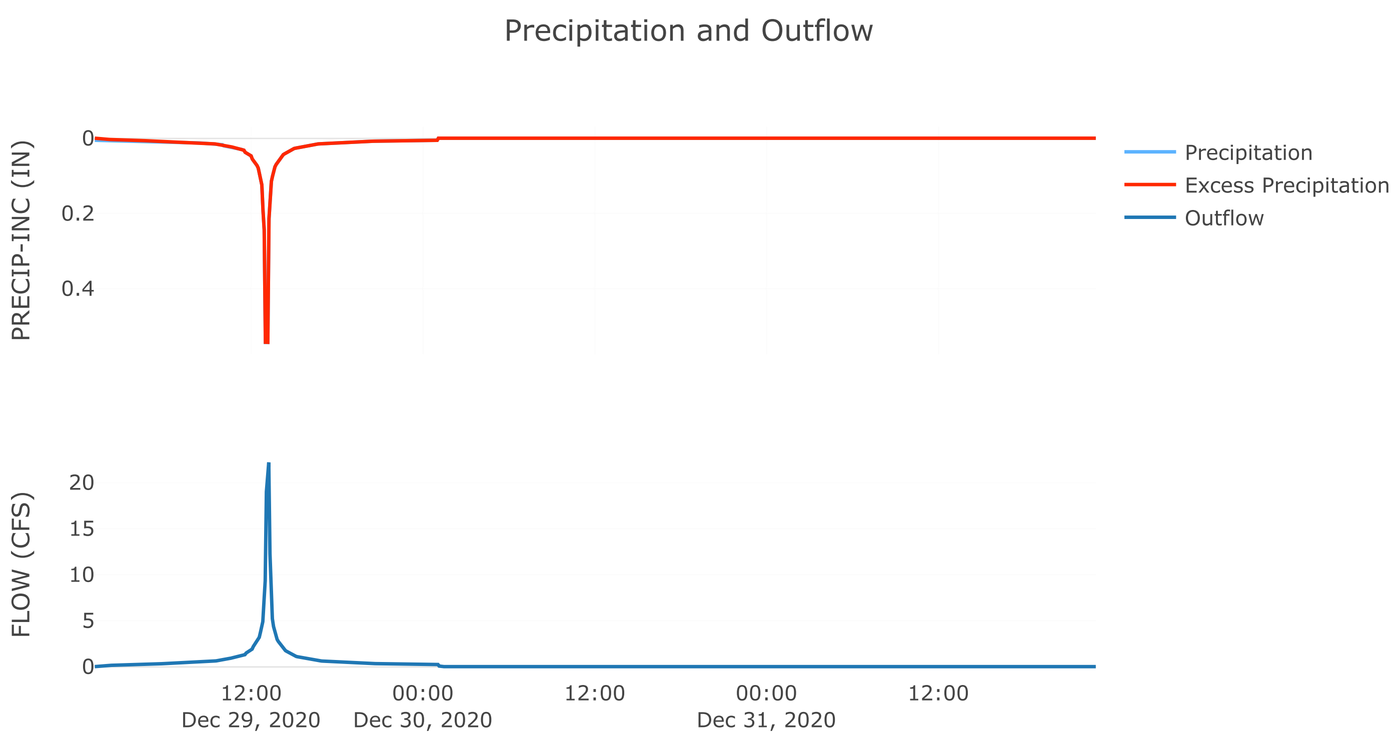


Subbasin: P5

Area (MIē) : 0.01
Downstream : Rcp5

Transform: Scs	
Lag	0.1
Unitgraph Type	Standard

Results: P5	
Peak Discharge (CFS)	22.19
Time of Peak Discharge	29Dec2020, 13:10
Volume (IN)	7.05
Precipitation Volume (AC - FT)	2.03
Loss Volume (AC - FT)	0.03
Excess Volume (AC - FT)	1.99
Direct Runoff Volume (AC - FT)	1.99
Baseflow Volume (AC - FT)	0

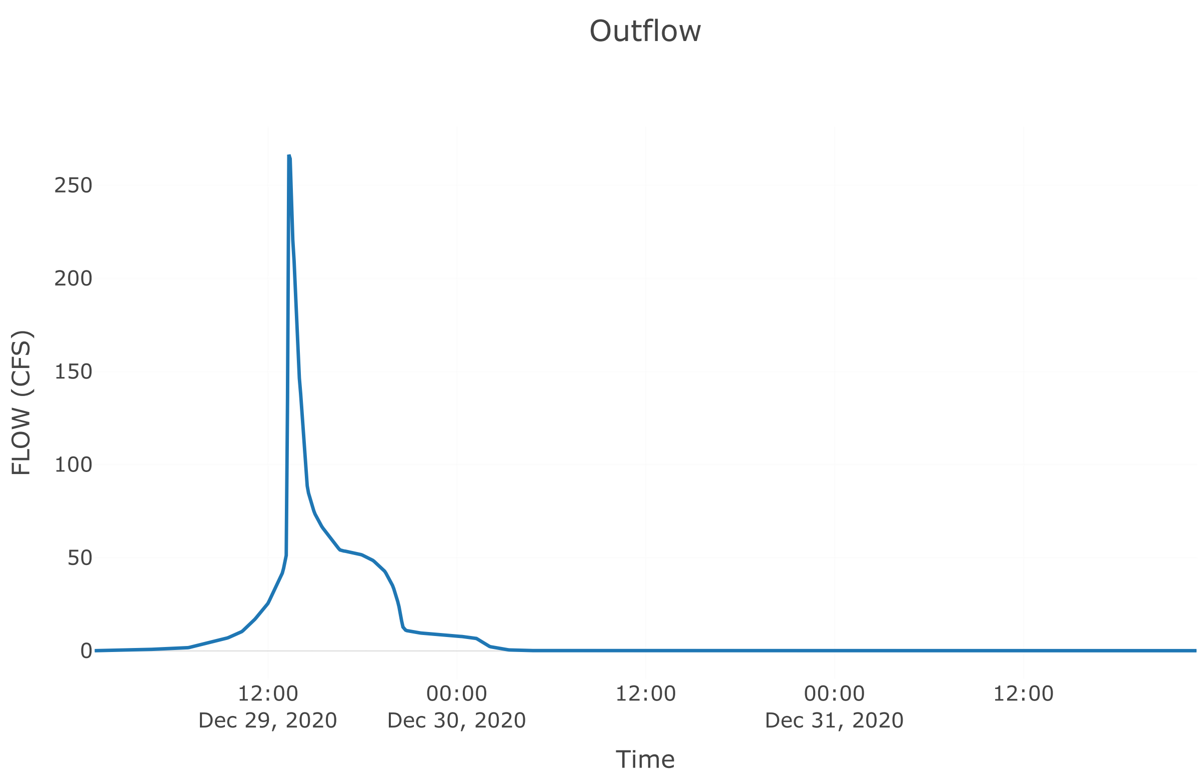


Reservoir: RCP5

Downstream : Dcp1

Results: RCP5

Peak Discharge (CFS)	266.33
Time of Peak Discharge	29Dec2020, 13:20
Volume (IN)	5.22
Peak Inflow (CFS)	332
Time of Peak Inflow	29Dec2020, 13:15
Inflow Volume (AC - FT)	55.76
Maximum Storage (AC - FT)	6.31
Peak Elevation (FT)	591.37
Discharge Volume (AC - FT)	55.85

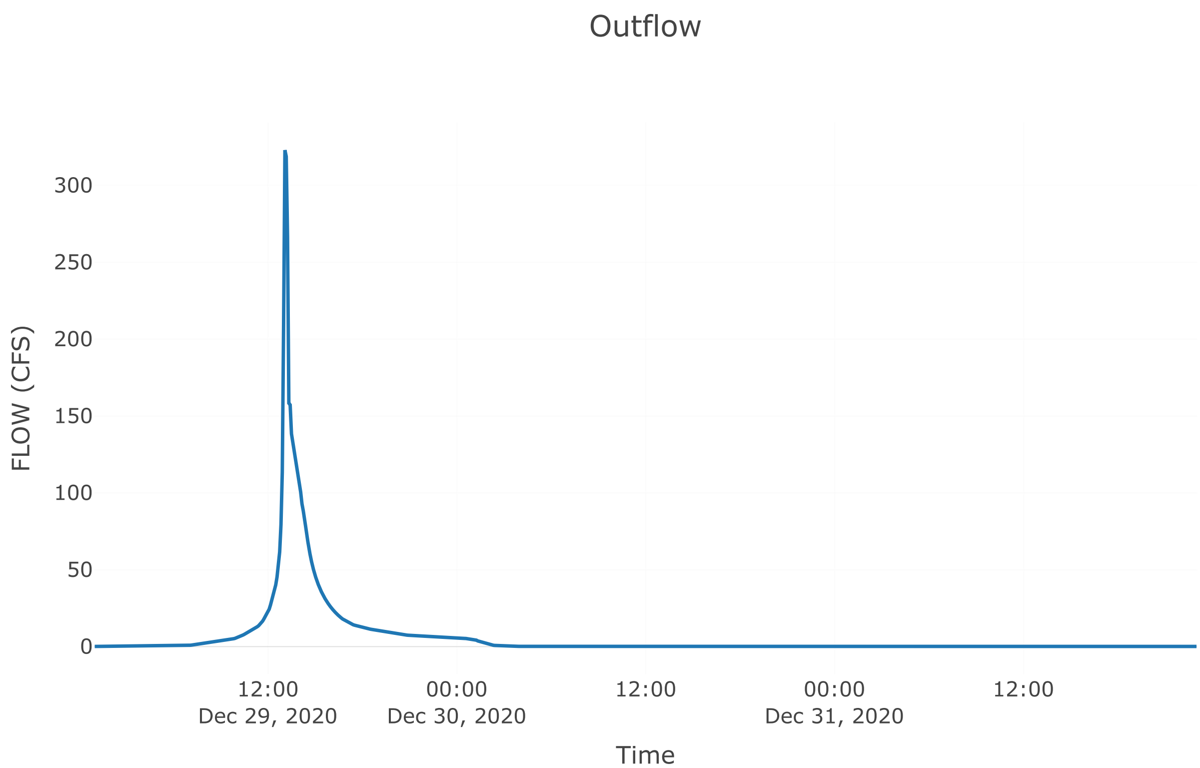


Junction: CS2

Downstream : Dcp1

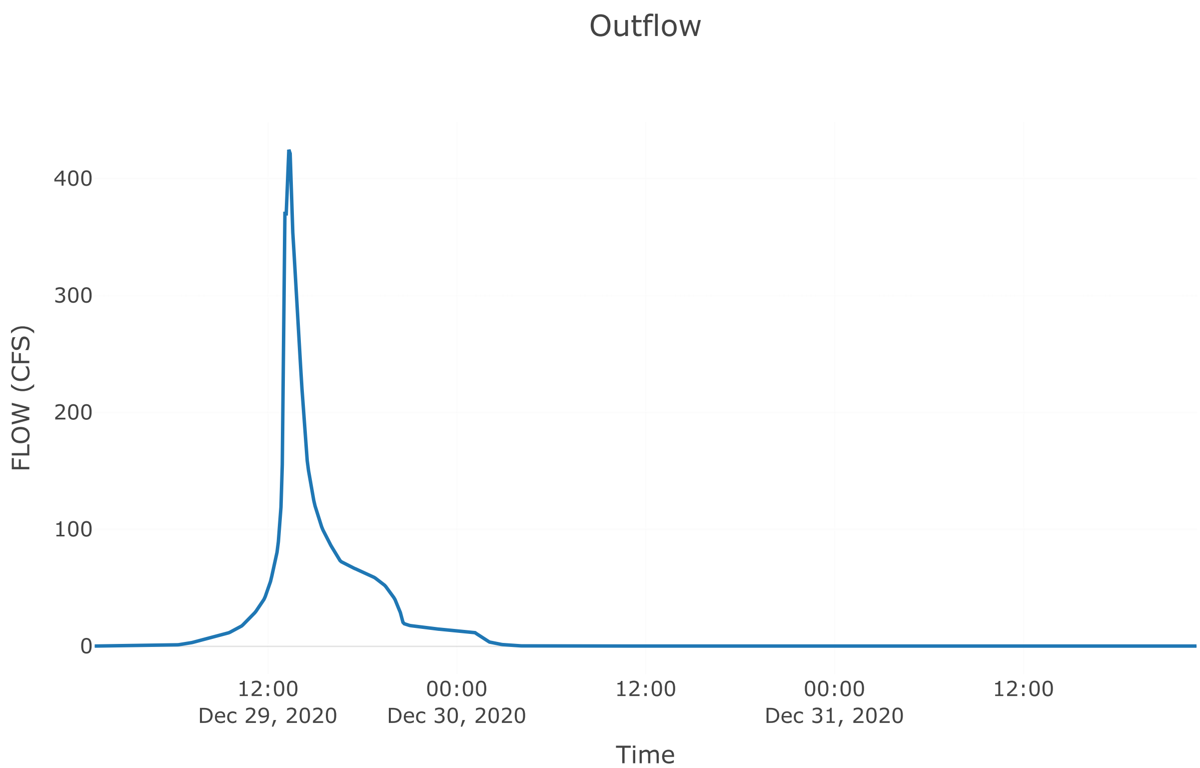
Results: CS2

Peak Discharge (CFS)	322.84
Time of Peak Discharge	29Dec2020, 13:05
Volume (IN)	5.12



Junction: DCP1

Results: DCP1	
Peak Discharge (CFS)	424.54
Time of Peak Discharge	29Dec2020, 13:20
Volume (IN)	5.18



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 on pages IIIF-A-27 through IIIF-A-59.
- Post-development condition volume information is summarized on page IIIF-A-62.

Required: Determine the volume generated by the site and offsite areas using the excess rainfall calculated in the HEC-HMS analysis of the post-development site conditions.

Method: 1. Use the excessive rainfall data generated by the HEC-HMS analysis (see IIIF-A-27 through IIIF-A-59) to determine the volume produced by the site for the post-development conditions.

1. Post-development Condition

1. a. Total Flow to Village Creek of Fort Worth C&D northwest of permit boundary (DCP1)

Area No.	Area (sq mi)	Total Excess Rainfall (in)	Area (ac)	Volume (ac-ft)
DA1	**0.0553**	**5.53**	**35.42**	**16.3**
DA2	**0.0502**	**5.53**	**32.15**	**14.8**
DA3	**0.0140**	**5.53**	**8.98**	**4.1**
DA4	**0.0517**	**5.53**	**33.08**	**15.2**
S1	**0.0155**	**5.30**	**9.92**	**4.4**
S2	**0.0553**	**5.30**	**35.42**	**15.6**
S3	**0.0032**	**5.30**	**2.03**	**0.9**
CH1	**0.0039**	**5.17**	**2.52**	**1.1**
CH2	**0.0052**	**5.17**	**3.31**	**1.4**
CH3	**0.0055**	**5.17**	**3.53**	**1.5**
CH4	**0.0075**	**5.17**	**4.80**	**2.1**
CH5	**0.0061**	**5.17**	**3.90**	**1.7**
P1	**0.0034**	**7.05**	**2.16**	**1.3**
P2-P4	**0.0059**	**7.05**	**3.77**	**2.2**
P5	**0.0053**	**7.05**	**3.37**	**2.0**
O1	**0.0080**	**5.30**	**5.11**	**2.3**
O2	**0.0071**	**5.30**	**4.56**	**2.0**
O3	**0.0272**	**5.30**	**17.39**	**7.7**
O4	**0.0098**	**5.30**	**6.25**	**2.8**

Total Volume of flow discharging from the Permit Boundary to Village Creek (refer to Figure 4.4 in the Drainage Report for the location) = 99.4 ac-ft

VELOCITY CALCULATIONS

FORT WORTH C&D LANDFILL
0771-356-11-35
VELOCITY CALCULATIONS
PROPOSED EXPANSION CONDITION

Required: Determine the flow velocities entering and exiting the permit boundary using HYDROCALC HYDRAULICS (Version 2.0, 1996-2010) for the flows calculated for the 25-year and 25- year storm event in the HEC-HMS analysis.

Method:

1. Use the flow data generated by the HEC-HMS analysis to determine velocity of runoff entering the landfill permit boundary.
2. Use the flow data generated by the HEC-HMS analysis to determine velocity of runoff exiting the landfill permit boundary.

1. Flow Velocity entering the landfill permit boundary

O1

- Flows were obtained from the HEC-HMS files included in this Appendix and are summarized below.

Q_{25} = 21.5 cfs

Storm Year	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)
25	21.5	0.0310	0.04	20.00	20.00	25.00	0.28	2.51

Note: Calculations were performed using the HYDROCALC HYDRAULICS for Windows program developed by Dodson and Associates (Version 2.01, 1996-2010)

O2

- Flows were obtained from the HEC-HMS files included in this Appendix and are summarized below.

Q_{25} = 20.3 cfs

Storm Year	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)
25	20.3	0.0322	0.04	13.00	26.00	25.00	0.27	2.50

Note: Calculations were performed using the HYDROCALC HYDRAULICS for Windows program developed by Dodson and Associates (Version 2.01, 1996-2010)

O3

- Flows were obtained from the HEC-HMS files included in this Appendix and are summarized below.

Q_{25} = 83.7 cfs

Storm Year	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)
25	83.7	0.0838	0.04	2.50	2.50	15.00	0.66	7.60

Note: Calculations were performed using the HYDROCALC HYDRAULICS for Windows program developed by Dodson and Associates (Version 2.01, 1996-2010)

O4

- Flows were obtained from the HEC-HMS files included in this Appendix and are summarized below.

Q_{25} = 30.5 cfs

Storm Year	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)
25	30.5	0.0642	0.04	4.00	4.00	2.00	0.89	6.16

Note: Calculations were performed using the HYDROCALC HYDRAULICS for Windows program developed by Dodson and Associates (Version 2.01, 1996-2010)

FORT WORTH C&D LANDFILL
0771-356-11-35
VELOCITY CALCULATIONS
PROPOSED EXPANSION CONDITION

2.
Flow Velocity exiting the landfill permit boundary

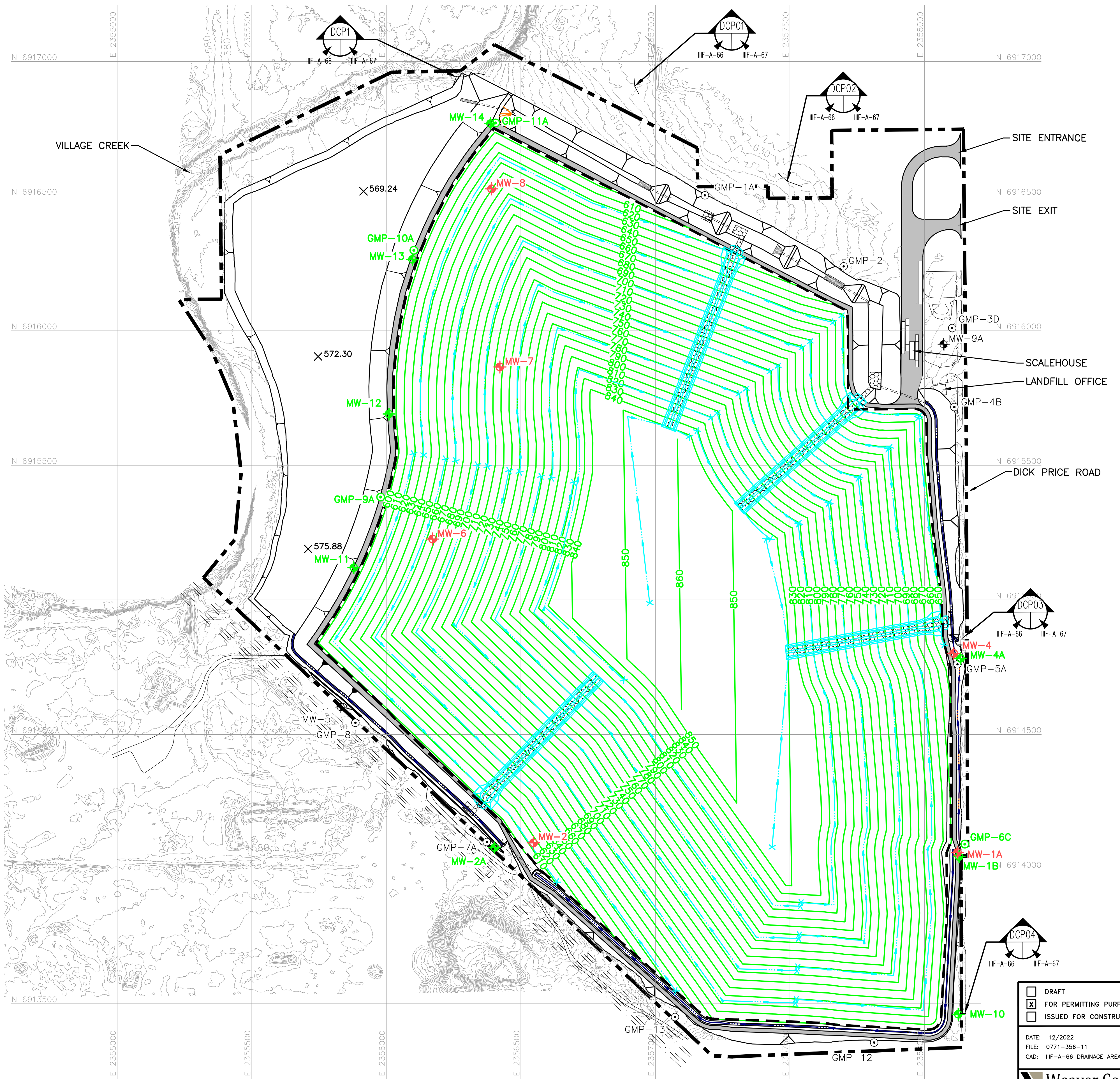
DCP1

- Flows were obtained from the HEC-HMS files included in this Appendix and are summarized below.

Q_{25} = 424.5 cfs

Storm Year	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)
25	424.5	0.0046	0.04	2.73	2.30	19.00	3.35	4.61

Note: Calculations were performed using the HYDROCALC HYDRAULICS for Windows program developed by Dodson and Associates (Version 2.01, 1996-2010).



0 200 400
SCALE IN FEET

LEGEND

	PERMIT BOUNDARY
	LIMIT OF WASTE
630	EXISTING CONTOUR
N 6915000	STATE PLANE COORDINATE
560	FINAL COVER CONTOUR
	DRAINAGE LETDOWN
	DRAINAGE SWALE
	PROPOSED PERIMETER CHANNEL
	GABIONS
MW-7	EXISTING/PERMITTED GROUNDWATER MONITORING WELL
MW-9A	APPROVED GROUNDWATER MONITORING WELL
MW-4	PERMITTED GROUNDWATER MONITORING WELL (TO BE ABANDONED)
GMP-4B	PERMITTED GAS MONITORING PROBE

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS PROVIDED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN 02-17-2022.
2. REFER TO APPENDIX IIIF-SURFACE WATER DRAINAGE PLAN FOR DRAINAGE DESIGN INFORMATION.
3. MAXIMUM FINAL COVER ELEVATION 860 FT-MSL.

STATE OF TEXAS
CHARLES R. MARSH
105073
LICENSED
PROFESSIONAL ENGINEER
02/09/2023

☐ DRAFT
☒ FOR PERMITTING PURPOSES ONLY
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DATE: 12/2022
FILE: 0771-356-11
CAD: IIIF-A-66 DRAINAGE AREAS.DWG

DRAWN BY: JDW
DESIGN BY: BPY
REVIEWED BY: CRM

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

PREPARED FOR
TEXAS REGIONAL LANDFILL COMPANY, LP

REVISIONS

NO.	DATE	DESCRIPTION

MAJOR PERMIT AMENDMENT
POST PROJECT DRAINAGE

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

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DRAWING IIIF-A-66

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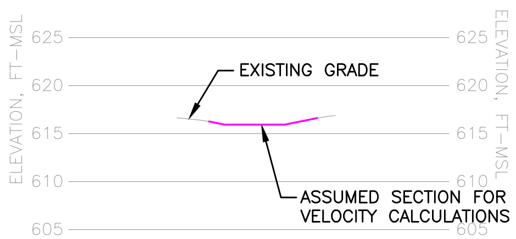
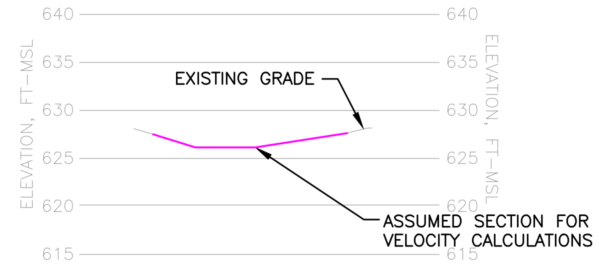


DISCHARGE POINT 01

DCP01 IIIF-A-66 IIIF-A-66

0 40 80
SCALE IN FEET (HORIZONTAL)

0 10 20
SCALE IN FEET (VERTICAL)

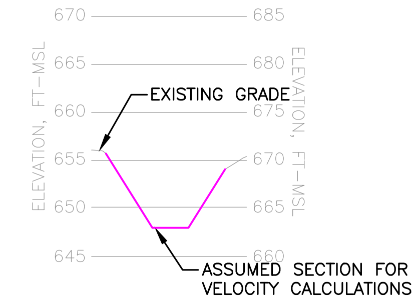


DISCHARGE POINT 02

DCP02 IIIF-A-66 IIIF-A-67

0 40 80
SCALE IN FEET (HORIZONTAL)

0 10 20
SCALE IN FEET (VERTICAL)

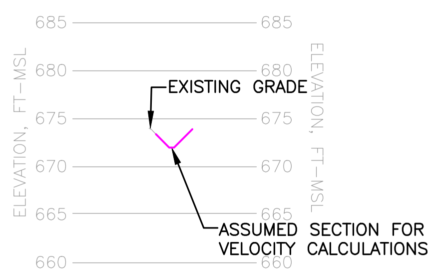


DISCHARGE POINT 03

DCP03 IIIF-A-66 IIIF-A-67

0 40 80
SCALE IN FEET (HORIZONTAL)

0 10 20
SCALE IN FEET (VERTICAL)

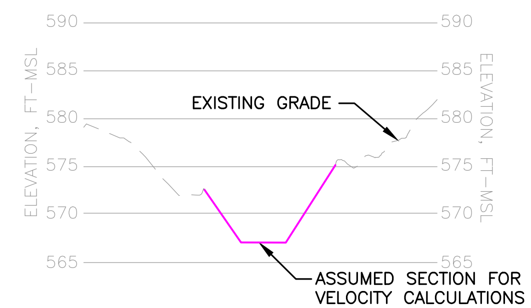


DISCHARGE POINT 04

DCP04 IIIF-A-66 IIIF-A-67

0 40 80
SCALE IN FEET (HORIZONTAL)

0 10 20
SCALE IN FEET (VERTICAL)



DISCHARGE POINT 1

DCP1 IIIF-A-66 IIIF-A-67

0 40 80
SCALE IN FEET (HORIZONTAL)

0 10 20
SCALE IN FEET (VERTICAL)

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02/09/2023

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DATE: 12/2022
FILE: 0771-356-11
CAD: IIIF-A-67-DISCHARGE POINT SEC.DWG

DRAWN BY: RAA
DESIGN BY: BPY
REVIEWED BY: CRM

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

PREPARED FOR
TEXAS REGIONAL LANDFILL COMPANY, LP

REVISIONS

NO.	DATE	DESCRIPTION

MAJOR PERMIT AMENDMENT
UPDATED PERMITTED DISCHARGE POINT VELOCITY CALCULATIONS

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

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DRAWING IIIF-A-67

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APPENDIX IIIF-B

PERIMETER CHANNEL, DETENTION POND, AND CULVERT DESIGN

Includes pages IIIF-B-1 through IIIF-B-20

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CONTENTS

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02/09/2023

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.
- Perimeter channel design system information is summarized on Drawing IIIF.4 in Appendix IIIF.
- Channel profiles are presented on Drawings IIIF.5 through IIIF.6 in Appendix IIIF.
- Hydraulic calculations are summarized on pages IIIF-B-2.
- Channel Erosion Control Design information is included on page IIIF-B-5.

FORT WORTH C&D LANDFILL
0771-356-11-35
PROPOSED PERIMETER CHANNEL DESIGN

Channel	Station		Flow Rate (cfs)	Bottom Slope (ft/ft)	Bottom Width (ft)	Side Slope (ft/ft) Right	Left	Mannings $n^{2,3}$	Normal Depth (ft)	Flow Vel. (fps)	Froude No.	Vel. Head (ft)	Energy Head (ft)	Flow Area (sq.ft.)	Top width of Flow (ft)	Freeboard [4] (ft)
CH1	0 + 0.00	1 + 85.10	14.6	0.010	0	3	3	0.03	1.20	3.39	0.773	0.18	1.38	4.30	7.19	1.76
	1 + 85.10	3 + 84.85	14.6	0.032	0	3	3	0.03	0.96	5.26	1.338	0.43	1.39	2.77	5.77	2.16
	3 + 84.85	6 + 82.52	14.6	0.048	0	3	3	0.03	0.89	6.11	1.611	0.58	1.47	2.39	5.36	2.11
	6 + 82.52	7 + 47.45	14.6	0.063	0	3	3	0.03	0.85	6.76	1.829	0.71	1.56	2.16	5.09	4.66
CH2	0 + 0.00	0 + 29.29	294.1	0.120	8	3	3	0.03	1.40	17.28	2.989	4.64	6.04	17.02	16.38	3.09
	0 + 29.29	9 + 15.51	294.1	0.005	8	3	3	0.03	3.11	5.46	0.677	0.46	3.57	53.86	26.65	1.38
CH3	0 + 0.00	6 + 28.45	44.9	0.005	3	3	3	0.03	1.64	3.44	0.603	0.18	1.83	13.04	12.87	1.36
	6 + 28.45	7 + 23.32	44.9	0.006	3	3	3	0.03	1.58	3.68	0.656	0.21	1.79	12.19	12.46	1.42
	7 + 23.32	11 + 5.2	44.9	0.004	3	3	3	0.03	1.73	3.17	0.543	0.16	1.89	14.17	13.38	1.27
CH4	0 + 0.00	3 + 92.77	69.6	0.004	8	3	3	0.03	1.60	3.40	0.555	0.18	1.78	20.48	17.60	1.40
	3 + 92.77	12 + 86.69	69.6	0.074	13	2.5	2.5	0.03	0.56	8.58	2.111	1.14	1.71	8.12	15.82	1.24
CH5	0 + 0.00	1 + 48.35	300.8	0.012	8	4	3	0.03	2.46	7.35	1.018	0.84	3.30	40.91	25.24	2.48
	1 + 48.35	5 + 69.65	300.8	0.008	8	6	3	0.03	2.56	6.01	0.835	0.56	3.12	50.06	31.06	3.19
	5 + 69.65	8 + 50.83	300.8	0.023	8	4	3	0.03	2.10	9.33	1.381	1.35	3.45	32.23	22.70	7.12
	8 + 50.83	11 + 17.18	300.8	0.014	8	3	3	0.03	2.45	8.02	1.099	1.00	3.44	37.52	22.68	12.22

Note:
1. Calculations were performed using the HYDROCALC Computer Program developed by Dodson and Associates (Version 2.0, 1996-2010).
2. n = 0.03 (Manning Coefficient) is used for grass-lined and turf mat-lined channels.
3. n = 0.04 (Manning Coefficient) is used for gabion-lined channels.
4. Freeboard is considered the difference between the water surface elevation and the top of the channel bank.

Example Calculation: Calculate the 25-year normal depth for Channel 3 between stations 6+28.45 and 7+23.32.

List of Symbols:

Q_d = peak flow rate for channel, cfs - obtained from HEC-HMS Analysis (Appendix IIIF-A)
R = hydraulic radius, ft
n = Manning's roughness coefficient
S = channel slope, ft/ft
b = bottom width of channel, ft
z = z-ratio (ratio of run to rise for channel sideslope)
A_f = flow area, sf
g = gravitational acceleration = 32.2 ft/s^2
T = top width of flow, ft
d = normal depth of channel, ft

The program uses an iterative process to calculate the normal depth of the channel to satisfy Manning's Equation

$$Q = \frac{1.486}{n} A R^{0.67} S^{0.5}$$

Design Inputs:

Q_d =	44.9	cfs
S =	0.006	ft/ft
b =	3	ft
z =	3	(H) : 1 (V)
n =	0.03	

Step 1 - Based on the geometry of the channel cross-section, solve for R and A_f

$$R = \frac{bd + zd^2}{b + 2d(z^2 + 1)^{0.5}}$$

$$A_f = bd + zd^2$$

assume: d = 1.58 ft

R = 0.940 ft

A_f = 12.19 sf

solve for Q: Q = 44.9

Step 2 - solve for velocity, T, Froude number, velocity head, and energy head

Q = VA => V = Q/A

V = 3.68 ft/s

T = b + 2(z x d)

T = 12.46 ft

$$F_r = \frac{V}{(gA/T)^{0.5}}$$

F_r = 0.656

$$\text{Velocity Head} = \frac{V^2}{2g}$$

Velocity Head = 0.21 ft

Energy Head = water elevation + velocity head

Energy Head = 1.79 ft

CHANNEL EROSION CONTROL DESIGN

Channel erosion controls have been designed for flow velocities resulted from the 25-year frequency flow rates. As shown on pages IIIF-B-2, velocities in the perimeter channels range from 3.17 ft/s to 17.28 ft/s. The channel lining needed to protect against erosive velocities is shown on Drawings IIIF.5 and IIIF.6 in Appendix IIIF. All channels and drainage features will be inspected and maintained in accordance with the Site Operating Plan.

The following was used to select the type of channel lining material.

- Vegetation – used in all areas where velocities are less than 5 ft/s for channels.
- Turf reinforcement matting – used in channels for velocities between 5 ft/s and 20 ft/s. Please refer to page IIIF-B-6 for more information.
- 2-foot-thick Gabions – used at chute discharges in channels, anywhere that flow velocities could exceed 20 ft/s, and detention ponds outlets (see Appendix IIIF-C – Final Cover Erosion Control Structure Design).

Channel lining details are presented on Drawings IIIF.7 in Appendix IIIF.

North American Green
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Evansville, IN 47725
800-772-2040
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www.nagreen.com

Material and Performance Specification Sheet

A tensar Company

C350 Turf Reinforcement Mat

The composite turf reinforcement mat (C-TRM) shall be a machine-produced mat of 100% coconut fiber matrix incorporated into a permanent three-dimensional turf reinforcement matting. The matrix shall be evenly distributed across the entire width of the matting and stitch bonded between a super heavy duty UV stabilized nettings with 0.50 x 0.50 inch (1.27 x 1.27 cm) openings, an ultra heavy UV stabilized, dramatically corrugated (crimped) intermediate netting with 0.5 x 0.5 inch (1.27 x 1.27 cm) openings, and covered by an super heavy duty UV stabilized nettings with 0.50 x 0.50 inch (1.27 x 1.27 cm) openings. The middle corrugated netting shall form prominent closely spaced ridges across the entire width of the mat. The three nettings shall be stitched together on 1.50 inch (3.81cm) centers with UV stabilized polypropylene thread to form a permanent three-dimensional turf reinforcement matting.

The C350 shall meet requirements established by the Erosion Control Technology Council (ECTC) Specification and the US Department of Transportation, Federal Highway Administration's (FHWA) *Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects, FP-03 Section 713.18 as a Type 5A, B, and C Permanent Turf Reinforcement Mat.*

Installation staple patterns shall be clearly marked on the turf reinforcement matting with environmentally safe paint. All mats shall be manufactured with a colored thread stitched along both outer edges (approximately 2-5 inches [5-12.5 cm] from the edge) as an overlap guide for adjacent mats.

Material Content		
Matrix	100% Coconut fibers	0.50 lbs/yd² (0.27 kg/m²)
Nettings	Top and Bottom, UV stabilized Polypropylene	8 lb/1000 ft² (3.91 kg/100 m²)
	Middle, corrugated UV stabilized Polypropylene	24 lb/1000 ft² (11.7 kg/100 m²)
Thread	Polypropylene, UV stabilized	

C350 is available in the following roll sizes:

Width	6.5 ft (2.0 m)
Length	55.5 ft (16.9 m)
Weight ± 10%	37 lbs (16.8 kg)
Area	40.0 yd² (33.4 m²)

Index Value Properties:

Property	Test Method	Typical	Net Only
Thickness	ASTM D6525	0.67 in (17.0 mm)	0.51 in
Resiliency	ASTM 6524	90%	---
Density	ASTM D792	0.53 oz/in³	---
Mass/Unit Area	ASTM 6566	12.57 oz/yd² (426 g/m²)	---
Porosity	ECTC Guidelines	99%	---
Stiffness	ASTM D1388	3.83 oz-in	---
Light Penetration	ECTC Guidelines	9.0%	---
UV Stability	ASTM D4355/ 1000 hr	86%	86%
Tensile Strength MD	ASTM D6818	625 lbs/ft (9.12 kN/m)	698 lbs/ft
Elongation MD	ASTM D6818	22%	30%
Tensile Strength TD	ASTM D6818	768 lbs/ft (11.21 kN/m)	710 lbs/ft
Elongation TD	ASTM D6818	15%	20%

Bench Scale Testing* (NTPEP):

Test Method	Parameters	Results
ECTC Method 2 Rainfall	50 mm (2 in)/hr for 30 min	SLR** = 18.32
	100mm (4 in)/hr for 30 min	SLR** = 19.65
	150 mm (6 in)/hr for 30 min	SLR** = 20.48
ECTC Method 3 Shear Resistance	**Shear at 0.50 inch soil loss**	**7.5 lbs/ft²**
ECTC Method 4 Germination	Top Soil, Fescue, 21 day incubation	243% improvement of biomass

* Bench Scale tests should not be used for design purposes

** Soil Loss Ratio = Soil loss with Bare Soil/Soil Loss with RECP (soil loss is based on regression analysis)

Performance Design Values:

Maximum Permissible Shear Stress		
	Short Duration	Long Duration
Phase 1 Unvegetated	3.2 lbs/ft² (153 Pa)	3.0 lbs/ft² (144 Pa)
Phase 2 Partially Veg.	10.0 lbs/ft² (480 Pa)	10.0 lbs/ft² (480 Pa)
Phase 3 Fully Veg.	12.0 lbs/ft² (576 Pa)	10.0 lbs/ft² (480 Pa)
Velocity Unveg	10.5 ft/s (3.2 m/s)	
Velocity Veg.	20 ft/s (6.0 m/s)	

Slope Design Data: C Factors			
	Slope Gradients (S)		
Slope Length (L)	≤ 3:1	3:1 – 2:1	≥ 2:1
≤ 20 ft (6 m)	0.0005	0.015	0.043
20-50 ft	0.018	0.031	0.050
≥ 50 ft (15.2 m)	0.035	0.047	0.057

Roughness Coefficients- Unveg.	
Flow Depth	Manning's n
≤ 0.50 ft (0.15 m)	0.041
0.50 – 2.0 ft	0.040 – 0.013
≥ 2.0 ft (0.60 m)	0.012

Product Participant of: QDOR PERFORMANCE VERIFIED

Updated 3/09

DETENTION POND DESIGN

Detention ponds have 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-8.

Downstream sides of the low-water outlets will be designed with either rock riprap or gabions as shown on pages IIIF-B-9 and IIIF-B-10.

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	P2	P3	P4	P5
Bottom ELEV, ft[1]	623.50	611.70	603.75	592.80	577.00
Spillway ELEV, ft	637.75	620.00	610.00	600.00	590.00
Spillway Length, ft	75	91	91	77	50
Top of Road/Berm, ft	643.40	625.65	614.35	604.00	595.00
Discharge Pipe Downstream Invert ELEV, ft	623.30	611.34	596.90	588.00	567.60
Peak Inflow Q_{25}, cfs	361.9	155.2	296.0	300.9	332.0
Peak Outflow Q_{25}, cfs	119.4	152.4	283.5	289.6	266.3
Peak Stage in Pond Q_{25}, ft	638.20	620.60	610.70	600.80	591.40
Est. Flow (Q_{25}) over Spillway, cfs	59.8	111.7	140.7	145.5	218.7

Note:
1) Details of the pond outlet structures are presented on Drawing IIIF.13.
2) The flow over the spillway is estimated using the formula $Q = CLH^{3/2}$ where C = 2.64, L is the length of the spillway in feet, and H is the head on the spillway in feet. The flow over the spillway conservatively assumes no flow through the low water outlet.

Required: Determine the minimum length and median diameter of riprap required at the detention pond outlet structures and creek culverts to control erosion in the detention pond outlet channels.

Reference:

1. Haan, Barfield, and Hayes, *Design Hydrology and Sedimentology for Small Catchments* , 1994.
2. U.S. Army Corps of Engineers, Hydrologic Engineering Center, *HEC-HMS Hydrologic Modeling System 4.9,* January 2022.
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 rates at the detention pond outlet structures and culverts. The flow at the outlet structures and culverts can be divided into two categories:

1. Flow over the Spillway/Road

Erosion protection calculations for the drainage structures will be based on flow through low water outlets/culverts only.

Flow Structure Spillway Topslope	25-Year Flow Rate (cfs)	25-Year Velocity (ft/s)	25-Year Flow Depth (ft)	25-Year Foude Number	25-Year Velocity Head (ft)	25-Year Energy Head (ft)	25-Year Flow Area (sq. ft.)	25-Year Top Width (ft)
P1	59.8	1.58	0.50	0.399	0.04	0.53	37.98	77.97
P2	111.7	1.87	0.64	0.417	0.05	0.70	59.62	94.85
P3	140.7	2.05	0.74	0.426	0.07	0.80	68.60	95.42
P4	145.5	2.21	0.83	0.434	0.08	0.91	65.92	81.98
P5	218.7	2.61	1.28	0.453	0.11	1.39	83.66	80.72

Flow Structure Spillway Sideslope	25-Year Flow Rate (cfs)	25-Year Velocity (ft/s)	25-Year Flow Depth (ft)	25-Year Foude Number	25-Year Velocity Head (ft)	25-Year Energy Head (ft)	25-Year Flow Area (sq. ft.)	25-Year Top Width (ft)
P1	59.8	6.83	0.12	3.538	0.72	0.84	8.76	75.70
P2	111.7	8.07	0.15	3.664	1.01	1.16	13.85	91.91
P3	140.7	8.83	0.17	3.747	1.21	1.39	15.90	92.04
P4	145.5	9.56	0.20	3.822	1.42	1.62	15.21	78.18
P5	218.7	13.25	0.32	4.142	2.73	3.05	16.51	51.94

2. Flow through the Low Water Outlet

The flow rate through the low water outlet (LWO) is summarized below.

Flow Structure	Pond Bottom Elev (ft-msl)	LWO Invert Elev. Upstream (ft-msl)	LWO Invert Elev. Downstream (ft-msl)	LWO Dimensions (in)	25-Year Flow Rate[2] (cfs)	25-Year Outlet Velocity[1] (ft/s)
RCP NORTH	-	638.50	624.70	2 X 48 (DIA)	147.05	29.51
P1	623.50	623.50	623.30	24 (DIA)	59.60	18.97
P2	611.70	611.70	611.34	24 (DIA)	40.70	12.96
P3	603.75	603.75	596.90	2 X 36 (DIA)	71.40	27.04
P4	592.80	592.80	588.00	48 (DIA)	144.10	27.25
P5	577.00	577.11	567.60	24 (DIA)	47.60	15.15
RCP SOUTH	-	600.94	594.06	4 X 24 (DIA)	17.40	13.66

[1] Velocities through the low water outlet for all culverts were calculated using the HYDROCALC HYDRAULICS FOR WINDOWS program developed by Dodson and Associates (Version 2.01, 1996-2010).

[2] The flowrates for all low water outlets are the peak discharges for the respective areas as calculated by subtracting the total flow calculated by HEC-HMS by the flow over the spillway. The total 25-year flowrate discharging
from RCP NORTH is 294.1 cfs / 2 pipes = 147.05 cfs per pipe, and
from P1 is 59.6 cfs / 1 pipe = 59.60 cfs per pipe, and
from P2 is 40.7 cfs / 1 pipe = 40.7 cfs per pipe, and
from P3 is 142.8 cfs / 2 pipe = 71.40 cfs per pipe, and
from P4 is 144.1 cfs / 1 pipe = 144.1 cfs per pipe, and
from P5 is 47.6 cfs / 1 pipe = 47.6 cfs per pipe, and
from RCP SOUTH is 69.6 cfs / 4 pipes = 17.40 cfs per pipe.

The velocity through the low water outlet is larger than the velocity over the spillway, when there is a low water outlet present. 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 5.24 and 5.25).

The minimum riprap length and diameter for each outlet is summarized below. Riprap was not designed for culvert as they discharge into channels or ponds. 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)	Length L x 1.2 (ft)	Rock Diameter (ft)
RCP NORTH	147.05	2 X 48 (DIA)	40	48	0.4
P1	59.60	24 (DIA)	65	78	0.6
P2	40.70	24 (DIA)	22	26	0.8
P3	71.40	2 X 36 (DIA)	29	35	0.3
P4	144.10	48 (DIA)	40	48	0.5
P5	47.60	24 (DIA)	55	66	0.6
RCP SOUTH	17.40	4 X 24 (DIA)	10	12	0.5

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)
RCP NORTH	20.0	22.0
P1	30.0	32.0
P2	12.8	15.0
P3	15.6	18.0
P4	20.0	22.0
RCP SOUTH	8.0	10.0

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.

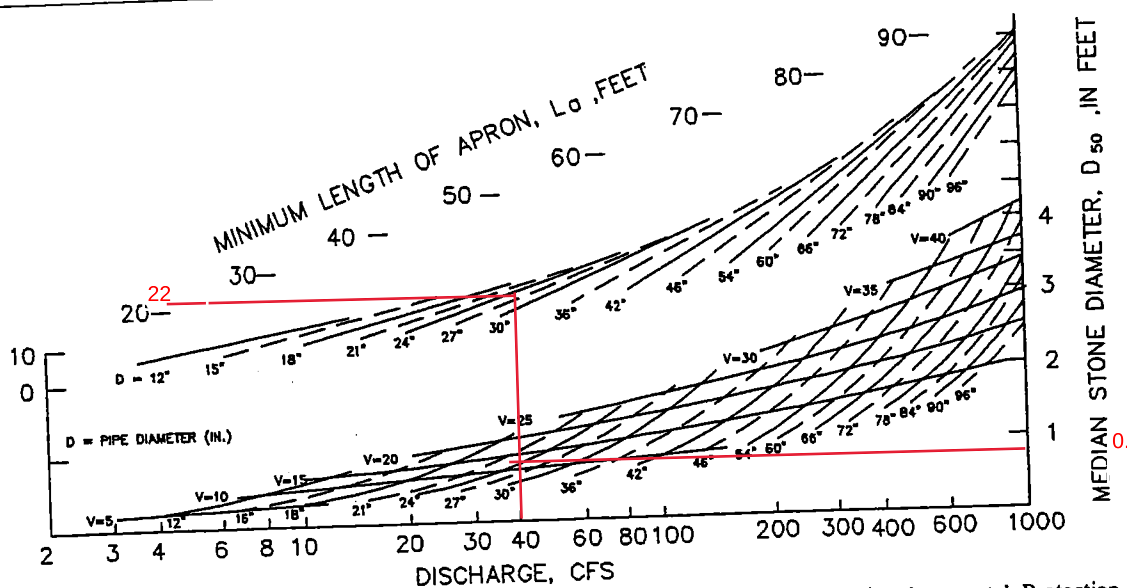


Figure 5.24 Design of outlet protection—minimum tailwater condition, $T_w < 0.5D$ (Environmental Protection Agency, 1976).

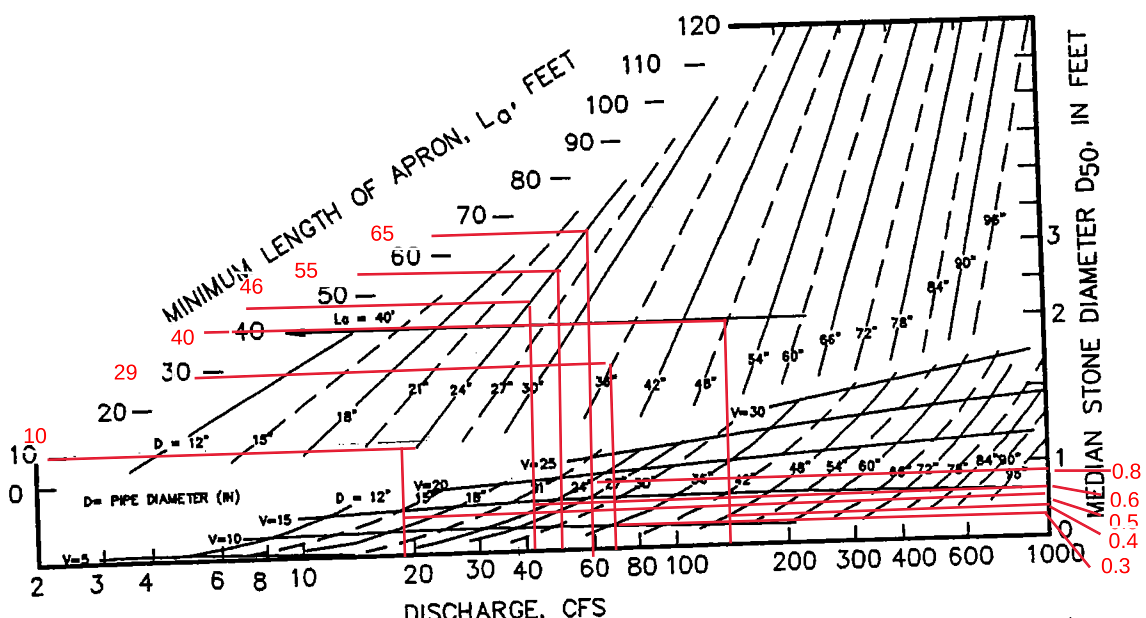


Figure 5.25 Design of outlet protection—maximum tailwater condition, $T_w \geq 0.5D$ (Environmental Protection Agency, 1976).

into the riser 3 ft below its top, what discharge will pass through the four holes with the water level at 1, 2, 4, and 8 ft above the riser? (c) What is the total discharge through the pipe? (d) How might the orifices be sized to provide better stormwater control? (e) Explain whether you would expect two rows (each consisting of four holes) of 8-in.-diameter holes to provide better results? Assume that one row is 2 ft below the riser invert and the other row is 4 ft below the riser invert.

(5.6) A gravel roadway is constructed in a low-lying area such that the roadway is frequently overtopped as a result of severe storms. The roadway is 40 ft wide, and its elevation is 36 ft. (a) If the water level upstream of the roadway is 2 ft above the crest of the roadway, what is the discharge across the roadway? (b) If the roadway is paved, what upstream depth would be required to carry the same flow? (c) Would paving reduce flooding problems?

CULVERT DESIGN

FORT WORTH C&D LANDFILL
0771-356-11-35
CULVERT DESIGN

Required: Design culverts to convey the flow.

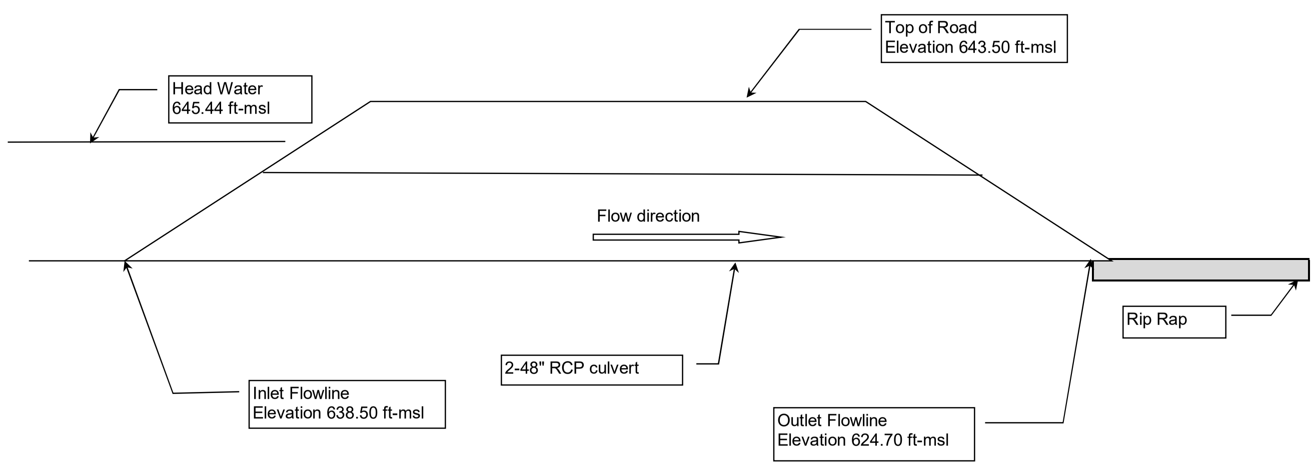
Method: Use HYDROCALC Hydraulics for Windows computer program to determine number and size of the culverts.
Use total 25-year frequency storm event flow estimated by HEC-HMS included in Appendix IIIF-A.

For RCP NORTH culvert

Total Flow=	294.1	cfs
No. of Culverts=	2	
Culvert Span=	--	inches
Culvert Rise=	--	inches
Culvert Diameter=	48	inches

Culvert ID	Culvert Span (ft)	Culvert Rise (ft)	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)	Headwater Inlet Control (ft)	Headwater Outlet Control (ft)	Normal Depth (ft)	Critical Depth (ft)	Depth at Outlet (ft)	Outlet Velocity (fps)
1	--	--	1	3	4	0.013	0.5	176.00	624.70	638.50	147.05	13.50	6.94	4.74	1.67	3.57	1.67	29.51

1. Calculations were performed using the HYDROCALC Hydraulics for Windows program developed by Dodson and Associates (Version 2.0, 1996-2010).
2. Tailwater depth is assumed to be the 25-year, 24-hour storm water surface elevation in P1 (638.20 ft-msl).



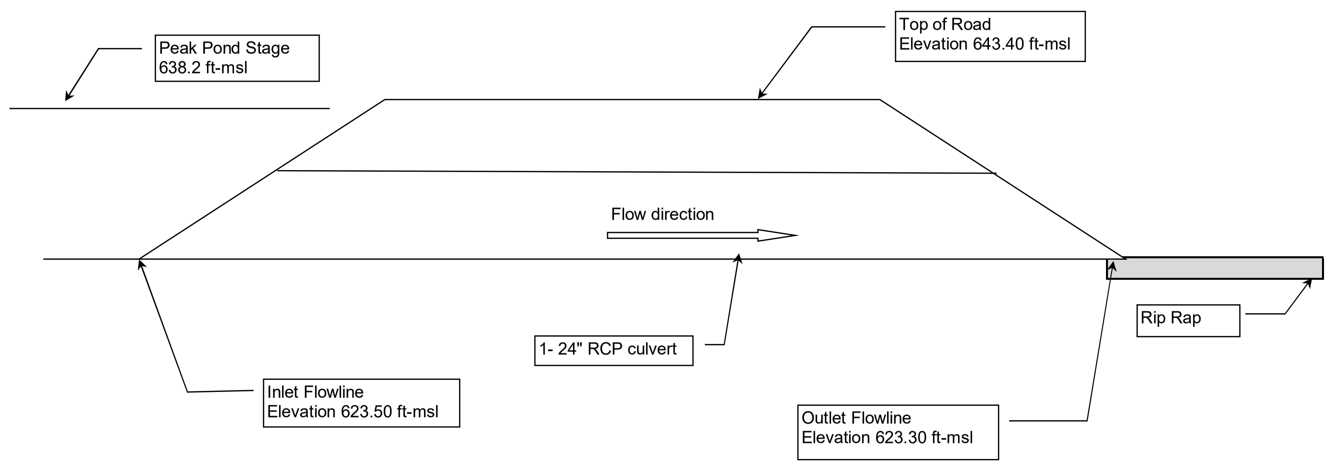
FORT WORTH C&D LANDFILL
0771-356-11-35
CULVERT DESIGN

For P1 culvert

Total Flow= 59.6 cfs
No. of Culverts= 1
Culvert Span= -- inches
Culvert Rise= -- inches
Culvert Diameter= 24 inches

Culvert ID	Culvert Span (ft)	Culvert Rise (ft)	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)	Headwater Inlet Control (ft)	Headwater Outlet Control (ft)	Normal Depth (ft)	Critical Depth (ft)	Depth at Outlet (ft)	Outlet Velocity (fps)
1	--	--	1	3	2	0.013	0.5	109.00	623.30	623.50	59.60	1.00	12.79	12.16	2.00	1.99	2.00	18.97

1. Calculations were performed using the HYDROCALC Hydraulics for Windows program developed by Dodson and Associates (Version 2.0, 1996-2010).
2. Since the peak pond stage in P2 does not reach the outlet elevation, the normal depth was used for the tailwater.



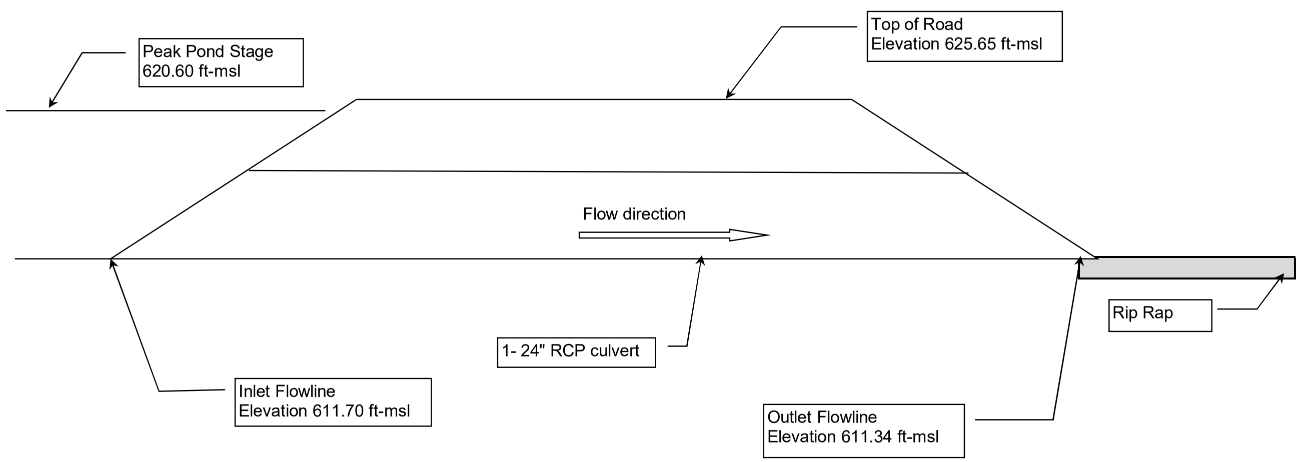
FORT WORTH C&D LANDFILL
0771-356-11-35
CULVERT DESIGN

For P2 culvert

Total Flow= 40.7 cfs
No. of Culverts= 1
Culvert Span= -- inches
Culvert Rise= -- inches
Culvert Diameter= 24 inches

Culvert ID	Culvert Span (ft)	Culvert Rise (ft)	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)	Headwater Inlet Control (ft)	Headwater Outlet Control (ft)	Normal Depth (ft)	Critical Depth (ft)	Depth at Outlet (ft)	Outlet Velocity (fps)
1	--	--	1	3	2	0.013	0.5	69.00	611.34	611.70	40.70	0.61	6.70	5.17	2.00	1.96	2.00	12.96

1. Calculations were performed using the HYDROCALC Hydraulics for Windows program developed by Dodson and Associates (Version 2.0, 1996-2010).
2. Since the peak pond stage in P3 does not reach the outlet elevation, the normal depth was used for the tailwater.

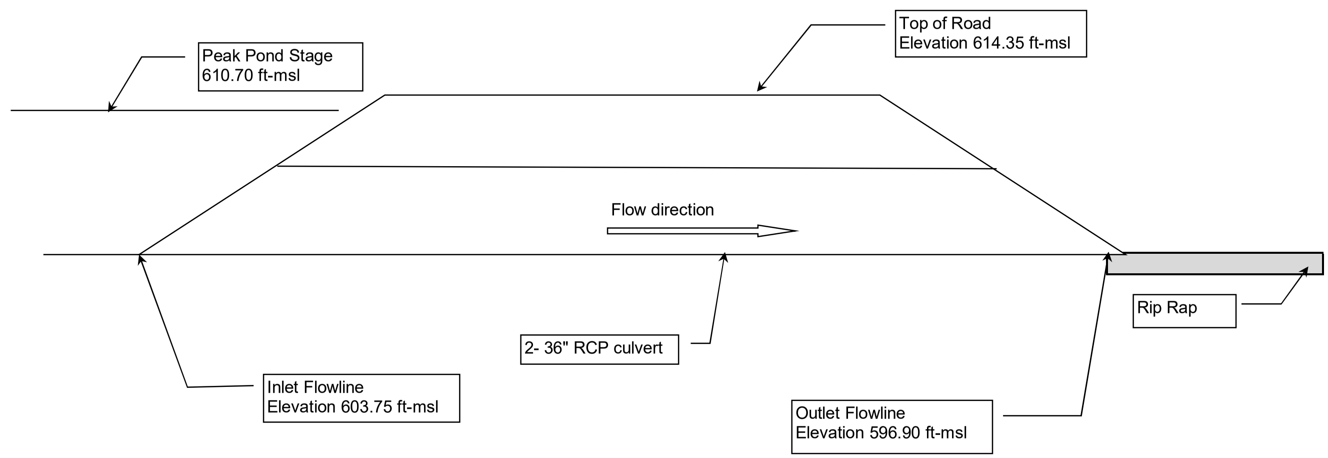


For P3 culvert

Total Flow= 142.8 cfs
No. of Culverts= 2
Culvert Span= -- inches
Culvert Rise= -- inches
Culvert Diameter= 36 inches

Culvert ID	Culvert Span (ft)	Culvert Rise (ft)	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)	Headwater Inlet Control (ft)	Headwater Outlet Control (ft)	Normal Depth (ft)	Critical Depth (ft)	Depth at Outlet (ft)	Outlet Velocity (fps)
1	--	--	1	3	3	0.013	0.5	68.00	596.90	603.75	71.40	3.90	5.15	0.21	1.20	2.68	1.20	27.04

1. Calculations were performed using the HYDROCALC Hydraulics for Windows program developed by Dodson and Associates (Version 2.0, 1996-2010).
2. Tailwater depth is assumed to be the 25-year, 24-hour storm water surface elevation in P4 (600.80 ft-msl).



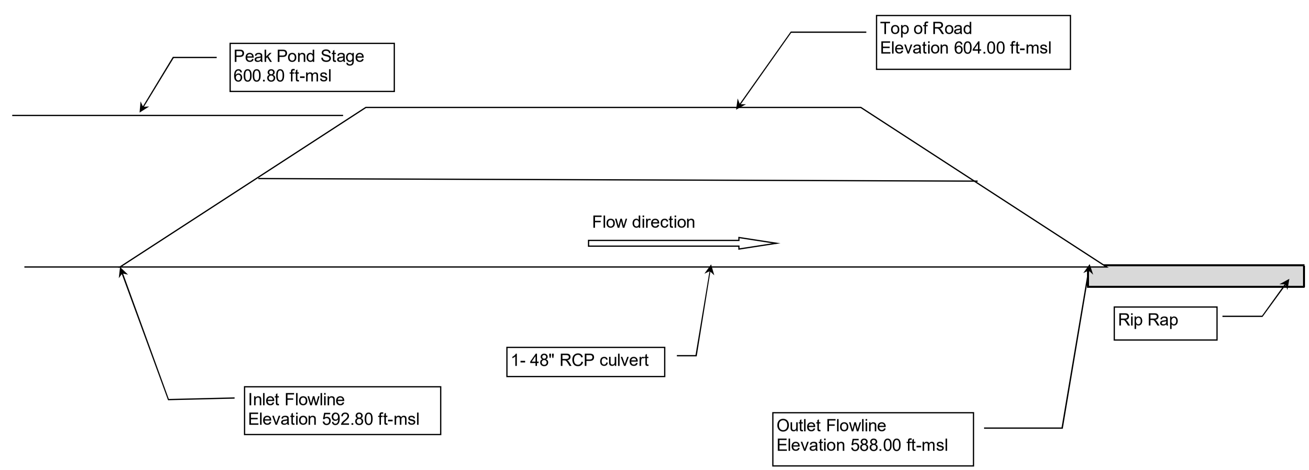
FORT WORTH C&D LANDFILL
0771-356-11-35
CULVERT DESIGN

For P4 culvert

Total Flow= 144.1 cfs
No. of Culverts= 1
Culvert Span= -- inches
Culvert Rise= -- inches
Culvert Diameter= 48 inches

Culvert ID	Culvert Span (ft)	Culvert Rise (ft)	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)	Headwater Inlet Control (ft)	Headwater Outlet Control (ft)	Normal Depth (ft)	Critical Depth (ft)	Depth at Outlet (ft)	Outlet Velocity (fps)
1	--	--	1	3	4	0.013	0.5	75.00	588.00	592.80	144.10	3.40	6.80	0.00	1.75	3.55	1.75	27.25

1. Calculations were performed using the HYDROCALC Hydraulics for Windows program developed by Dodson and Associates (Version 2.01, 1996-2010).
2. Tailwater depth is assumed to be the 25-year, 24-hour storm water surface elevation in P5 (591.40 ft-msl).



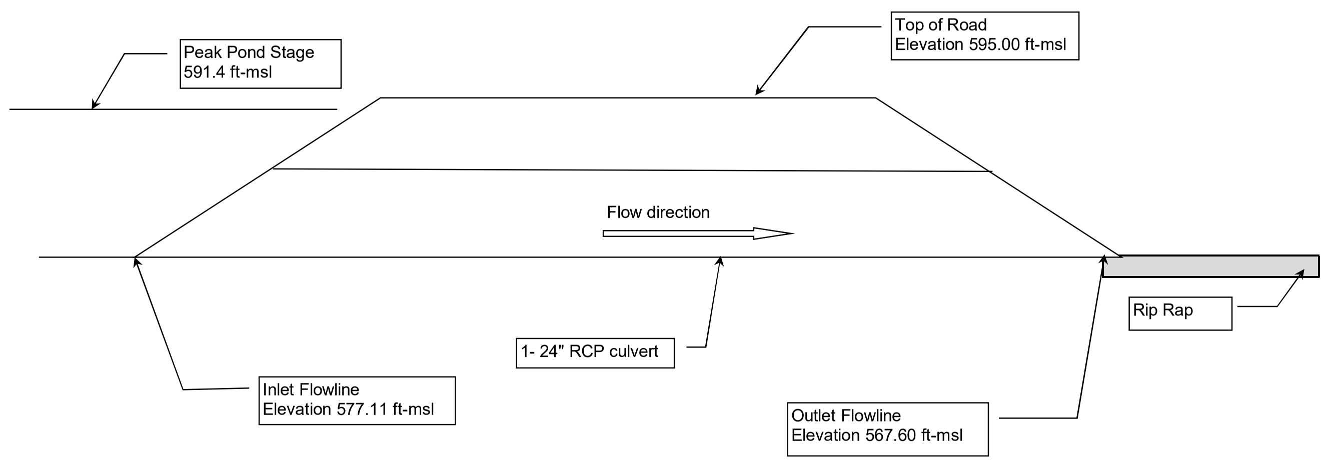
FORT WORTH C&D LANDFILL
0771-356-11-35
CULVERT DESIGN

For P5 culvert

Total Flow=	47.6	cfs
No. of Culverts=	1	
Culvert Span=	--	inches
Culvert Rise=	--	inches
Culvert Diameter=	24	inches

Culvert ID	Culvert Span (ft)	Culvert Rise (ft)	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)	Headwater Inlet Control (ft)	Headwater Outlet Control (ft)	Normal Depth (ft)	Critical Depth (ft)	Depth at Outlet (ft)	Outlet Velocity (fps)
1	--	--	1	3	2	0.013	0.5	134.00	567.60	577.11	47.60	11.79	8.59	13.56	1.34	1.98	2.00	15.15

1. Calculations were performed using the HYDROCALC Hydraulics for Windows program developed by Dodson and Associates (Version 2.0, 1996-2010).
2. Tailwater depth is assumed to be the 100-year storm water surface elevation (588.90 ft-msl) in proposed CLOMR (See Appendix IIIF-G for more information).
3. Head losses due to the flap gates on the culvert outlet are considered negligible due to the high flow rates.



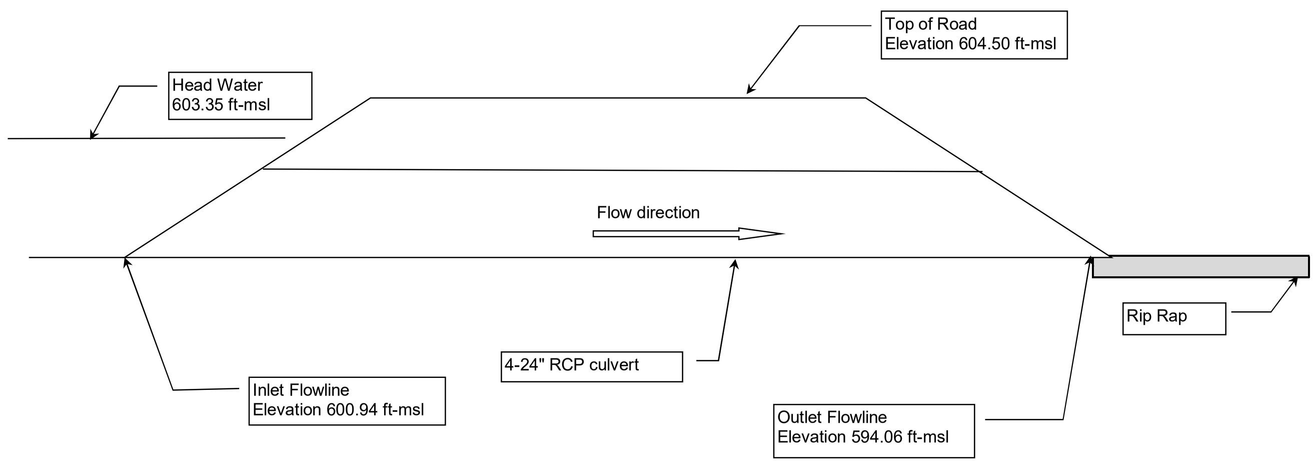
FORT WORTH C&D LANDFILL
0771-356-11-35
CULVERT DESIGN

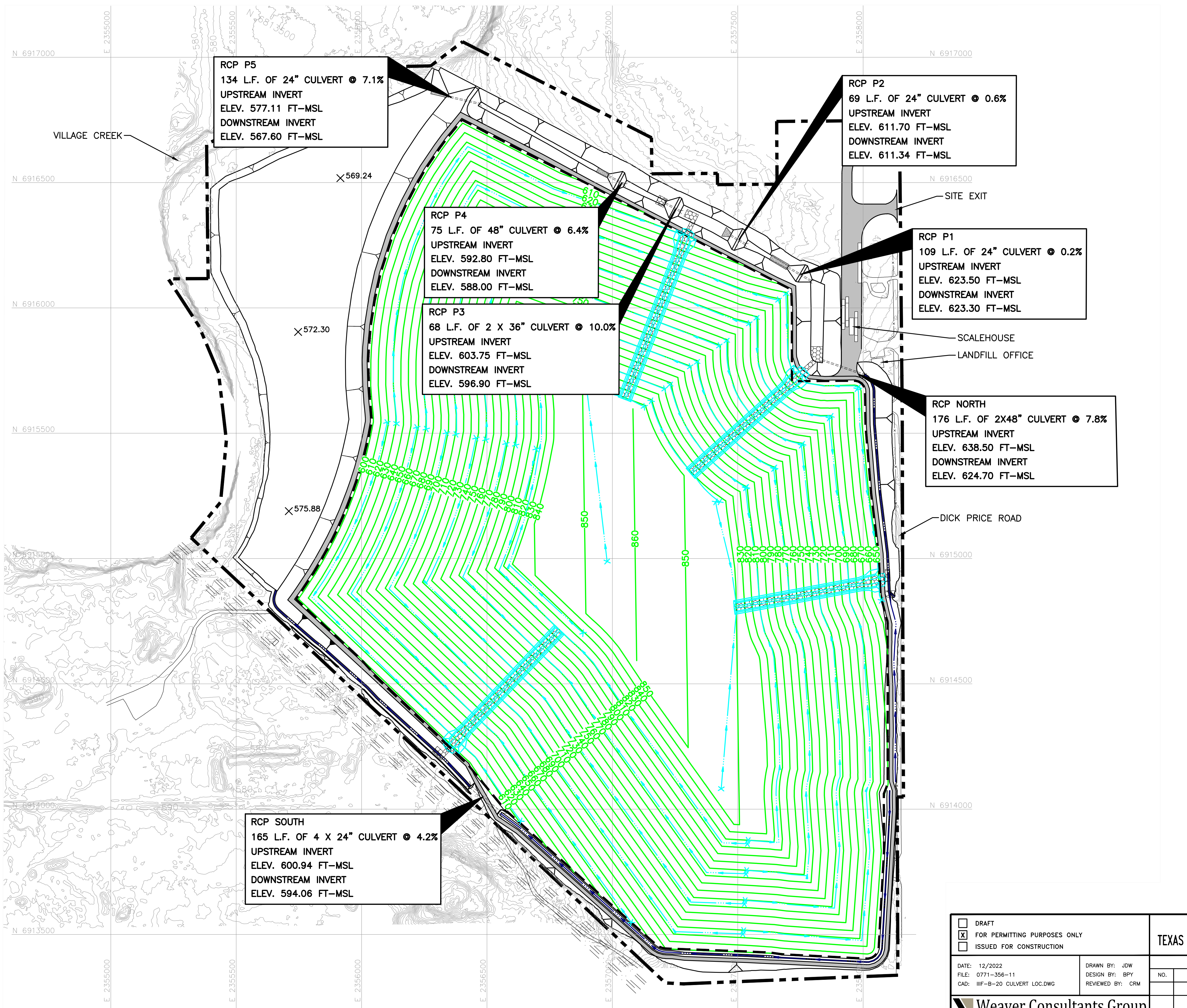
For RCP SOUTH culvert

Total Flow=	69.6	cfs
No. of Culverts=	4	
Culvert Span=	--	inches
Culvert Rise=	--	inches
Culvert Diameter=	24	inches

Culvert ID	Culvert Span (ft)	Culvert Rise (ft)	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)	Headwater Inlet Control (ft)	Headwater Outlet Control (ft)	Normal Depth (ft)	Critical Depth (ft)	Depth at Outlet (ft)	Outlet Velocity (fps)
1	--	--	1	3	2	0.013	0.5	165.00	594.06	600.94	17.40	2.46	2.35	-2.73	0.85	1.50	0.85	13.66

1. Calculations were performed using the HYDROCALC Hydraulics for Windows program developed by Dodson and Associates (Version 2.0, 1996-2010).
2. Tailwater depth is assumed to be the 25-year, 24-hour storm water normal depth in CH5 (2.46 ft).





0 200 400
SCALE IN FEET

LEGEND

- PERMIT BOUNDARY
- LIMIT OF WASTE
- 630 EXISTING CONTOUR
- N 6915000 STATE PLANE COORDINATE
- 560 FINAL COVER CONTOUR
- DRAINAGE LETDOWN
- DRAINAGE SWALE
- PROPOSED PERIMETER CHANNEL
- GABIONS

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS PROVIDED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN 02-17-2022.
2. REFER TO APPENDIX IIIF-SURFACE WATER DRAINAGE PLAN FOR DRAINAGE DESIGN INFORMATION.
3. MAXIMUM FINAL COVER ELEVATION 860 FT-MSL.

STATE OF TEXAS
CHARLES R. MARSH
105073
LICENSED
PROFESSIONAL ENGINEER
02/09/2023

☐ DRAFT
☒ FOR PERMITTING PURPOSES ONLY
☐ ISSUED FOR CONSTRUCTION

DATE: 12/2022
FILE: 0771-356-11
CAD: IIIF-B-20 CULVERT LOC.DWG

DRAWN BY: JDW
DESIGN BY: BPY
REVIEWED BY: CRM

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

PREPARED FOR
TEXAS REGIONAL LANDFILL COMPANY, LP

REVISIONS

NO.	DATE	DESCRIPTION

MAJOR PERMIT AMENDMENT
CULVERT LOCATIONS

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

WWW.WCGRP.COM

DRAWING IIIF-B-20

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O:\0771\356\EXPANSION 2022\PART III\IIIF\IIIF-B\IIIF-B-20 CULVERT PLAN.dwg, jwilson, 1:2

APPENDIX IIIF-C

FINAL COVER EROSION CONTROL STRUCTURE DESIGN

Includes pages IIIF-C-1 through IIIF-C-23

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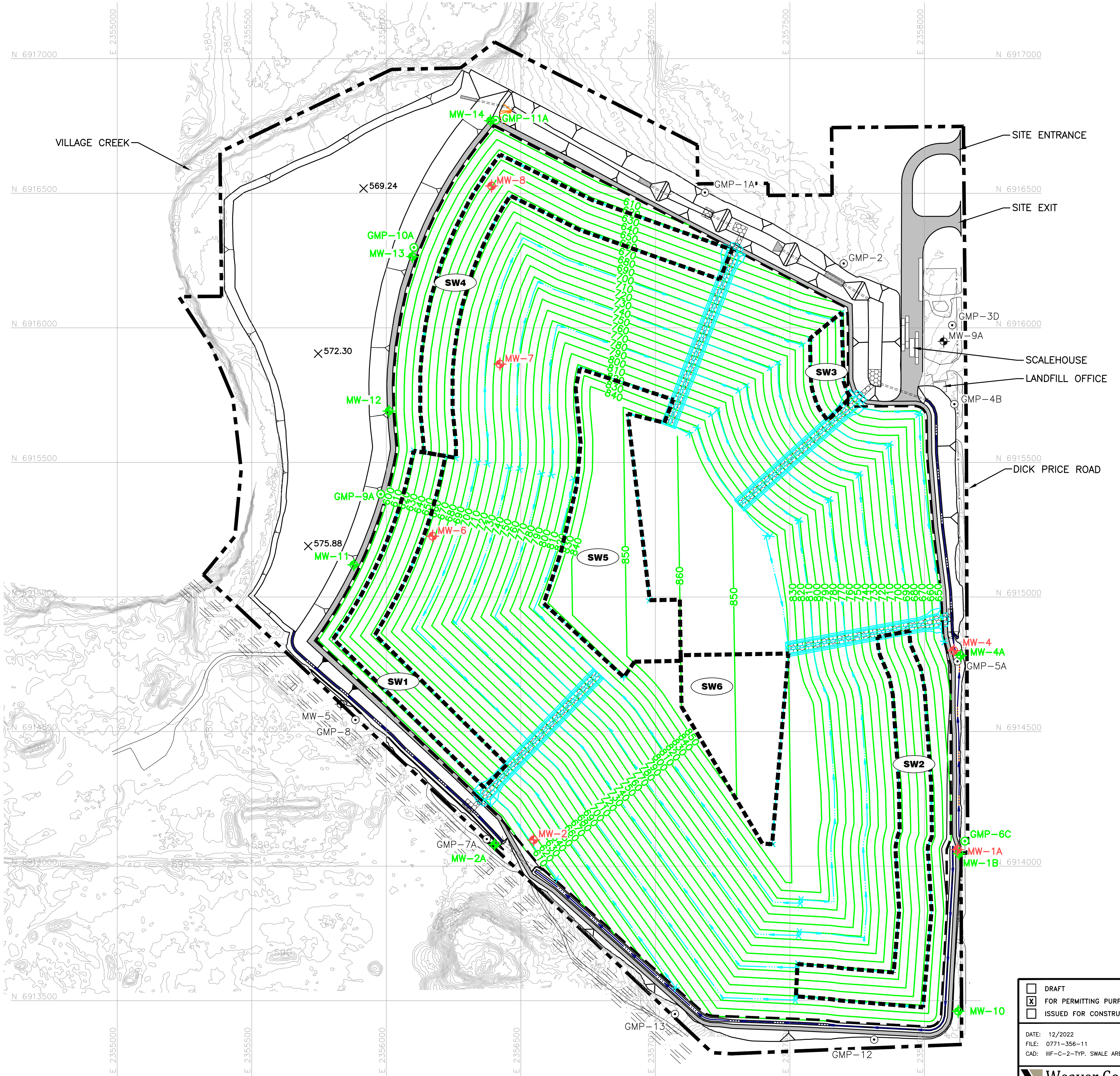
105073
LICENSED
PROFESSIONAL ENGINEER
02/09/2023

CONTENTS

STATE OF TEXAS
CHARLES R. MARSH
105073
LICENSED
PROFESSIONAL ENGINEER
02/09/2023

DRAINAGE SWALE DESIGN

- The drainage swale layout is shown on Drawing IIIF.1 – Drainage Structure Plan. A swale detail is provided on Drawing IIIF.7 – Drainage Details.
- Typical Swale Design Summary:
  - Typical swale drainage areas analyzed are shown on sheet IIIF-C-2.
  - Hydraulic calculations are summarized on page IIIF-C-5.
  - Maximum normal depth is 1.84 feet (Drainage Area SW4).
  - Maximum flow velocity is 3.18 fps (Drainage Area SW4).
  - Vegetation will be established on the swales to protect against erosion.
  - Typical slope conditions (0.5%) are included in this analysis. Additionally, swales with large individual drainage areas were used to conservatively represent all swales leading towards their respective letdown in the design.



0 200 400

SCALE IN FEET

LEGEND

- PERMIT BOUNDARY
- LIMIT OF WASTE
- 630 EXISTING CONTOUR
- N 6915000 STATE PLANE COORDINATE
- 560 FINAL COVER CONTOUR
- DRAINAGE LETDOWN
- DRAINAGE SWALE
- PROPOSED PERIMETER CHANNEL
- GABIONS
- MW-7 EXISTING/PERMITTED GROUNDWATER MONITORING WELL
- MW-9A APPROVED GROUNDWATER MONITORING WELL
- MW-4 PERMITTED GROUNDWATER MONITORING WELL (TO BE ABANDONED)
- GMP-4B PERMITTED GAS MONITORING PROBE
- DRAINAGE DIVIDE
- SW1 DRAINAGE AREA DESIGNATION

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS PROVIDED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN 02-17-2022.
2. REFER TO APPENDIX IIIF-SURFACE WATER DRAINAGE PLAN FOR DRAINAGE DESIGN INFORMATION.
3. MAXIMUM FINAL COVER ELEVATION 860 FT-MSL.

TYPICAL SWALE DRAINAGE AREA DESIGNATION	AREA (ACRES)
SW1	4.07
SW2	4.82
SW3	0.89
SW4	5.46
SW5	7.18
SW6	4.17

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105073
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PROFESSIONAL ENGINEER

02/09/2023

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DATE: 12/2022
FILE: 0771-356-11
CAD: IIIF-C-2-TYP. SWALE AREAS.DWG

DRAWN BY: JDW
DESIGN BY: BPY
REVIEWED BY: CRM

PREPARED FOR

TEXAS REGIONAL LANDFILL COMPANY, LP

REVISIONS

NO.	DATE	DESCRIPTION

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

MAJOR PERMIT AMENDMENT
SWALE DRAINAGE AREAS

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

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DRAWING IIIF-C-2

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O:\0771\356\EXPANSION 2022\PART III\IIIF\IIIF-C\IIIF-C-2 SWALE AREAS.dwg, jwilson, 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, September 2019.
2. NOAA Atlas 14 - Precipitation-Frequency Atlas of the United States, Volume 11, Version 2.0: Texas (U.S. Department of Commerce, National Oceanic and Atmospheric Administration, and National Weather Service, 2018)

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

$$I = \frac{b}{(t_c + d)^e}$$

$b =$ 79.18 From Ref. 2, for Johnson County
$d =$ 10.44 25-year storm event
$e =$ 0.772
t_c is assumed to be 10 min.

$I =$ 7.72 in/hr

Swale	Area[1] (ac)	Flow Rate (cfs)
SW1	4.07	22.0
SW2	4.82	26.0
SW3	0.89	4.8
SW4	5.46	29.5
SW5	7.18	38.8
SW6	4.17	22.5

[1] The total drainage area was conservatively assumed to be contributing to the swale at the analysis point.

Rainfall Intensity-Duration-Frequency Coefficients for Texas

Texas Department of Transportation

Based on "National Oceanic and Atmospheric Administration's (NOAA) Atlas 14
Precipitation-Frequency Atlas of the United States, Volume 11 Version 2.0: Texas" (Perica et al. 2018)

Parameter Selection

1. Select Units
   English
2. Select Methodology
   Annual Maximum Series (AMS)
3. Select County
   TARRANT
4. Select County Zone
   Zone-1
5. Select Time of Concentration (t_C)
   10 Minute

	Design Annual Exceedance Probability (Design Annual Recurrence Interval)						
Coefficient	50% (2-year)	20% (5-year)	10% (10-year)	4% (25-year)	2% (50-year)	1% (100-year)	0.2% (500-year)
e	0.7842	0.7793	0.7759	0.7715	0.7678	0.7643	0.7583
b	44.1286	57.0870	66.7228	79.1811	88.1558	97.2910	121.1438
d (min)	10.0200	10.2377	10.3432	10.4421	10.4601	10.5378	11.1141
Intensity (inches/hour)	4.21	5.48	6.44	7.72	8.68	9.66	11.99

Note: Tarrant County has 1 rainfall zone.

FORT WORTH C&D LANDFILL
0771-356-11-35
SWALE ANALYSIS

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	22.0	0.005	0.03	3.0	2.5	0	1.65	2.95	0.574	0.14	1.78	7.45	9.05
SW2	26.0	0.005	0.03	3.0	2.5	0	1.75	3.08	0.580	0.15	1.90	8.44	9.64
SW3	4.8	0.005	0.03	3.0	2.5	0	0.93	2.01	0.521	0.06	0.99	2.38	5.12
SW4	29.5	0.005	0.03	3.0	2.5	0	1.84	3.18	0.858	0.16	1.99	9.28	10.10
SW5	38.8	0.005	0.03	20.0	2.5	0	1.19	2.45	0.562	0.09	1.28	15.82	26.68
SW6	22.5	0.005	0.03	20.0	2.5	0	0.96	2.14	0.544	0.07	1.03	10.53	22.01

Note: Calculations were performed using the HYDROCALC HYDRAULICS program developed by Dodson and Associates (Version 2.01, 1996-2010).

Maximum flow depth is 1.84 ft < 2.0 ft (swale height).
Design is okay.

Example Calculation: Calculate the normal depth for the swale for drainage area SW1 (See IIIF-C-2)

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 =	22.0	cfs	(From page IIIF-C-3)
S =	0.005	ft/ft	
b =	0	ft	
z_r =	2.5	(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

$$R = \frac{bd + 1/2d^2(z_r + z_l)}{b + d((z_l^2 + 1)^{0.5} + (z_r^2 + 1)^{0.5})}$$

$$A_f = bd + 1/2d^2(z_r + z_l)$$

assume: d = 1.65 ft

R = 0.77 ft

A_f = 7.45 sf

solve for Q: $Q =$ 22.0

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

$Q = VA \Rightarrow \quad V = Q/A$

$V =$ 2.95 ft/s

$T = b + d(z_l + z_r)$

$T =$ 9.05 ft

$$F_r = \frac{V}{(gA/T`)^{0.5}}$$

$F_r =$ 0.574

$$\text{Velocity Head} = \frac{V^2}{2g}$$

Velocity Head = 0.14 ft

Energy Head = water elevation + velocity head

Energy Head = 1.78 ft

DRAINAGE LETDOWN (OR CHUTE) DESIGN

Chute Design

The letdown structures are designed using gabions, FML or other approved alternative provided so that the alternative system will provide adequate tractive stress, be geotechnically stable, and meet the hydraulic sizing criteria set forth in this appendix. Additional materials may be used for chute lining, provided it meets the design criteria in this appendix and relevant construction details are provided. Bedding for the gabions will be prepared subgrade soil overlain by 8 oz/sy geotextile (refer to Drawing IIIF.7). The gabions or FML 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 thirty three percent slopes in the low-water crossings over the perimeter road.

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 pages IIIF-C-9 and IIIF-C-10, and the calculation procedure is provided on pages IIIF-C-11 and IIIF-C-12.
- Chute layouts and drainage areas are shown on Sheet IIIF-C-13.
- The chute energy dissipater sizing calculation procedure is provided on pages IIIF-C-14 through IIIF-C-18.
- FML Anchor Trench Design calculations are provided on Pages IIIF-C-19 through IIF-C-23.
- Additional stormwater details are included on Drawings IIIF.7 through IIIF.12.

FORT WORTH C&D LANDFILL
0771-356-11-35
CHUTE ANALYSIS
NORMAL DEPTH CALCULATIONS FOR
GABION LINED CHUTES

Drainage Area	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)
SIDESLOPE AREAS													
LD1	212.1	0.33	0.04	3	3	8	1.04	18.30	3.577	5.20	6.24	11.59	14.25
LD2	188.8	0.33	0.04	3	3	8	0.98	17.67	3.549	4.85	5.83	10.68	13.86
LD3	53.6	0.33	0.04	3	3	8	0.48	11.82	3.227	2.17	2.65	4.54	10.88
LD4	197.2	0.33	0.04	3	3	8	1.00	17.89	3.555	4.97	5.97	11.03	14.01

Drainage Area	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)
LOW WATER CROSSING (2%) AREAS													
LD1	212.1	0.02	0.04	8	8	30	1.11	4.90	0.909	0.37	1.49	43.24	47.79
LD2	188.8	0.02	0.04	8	8	26	1.12	4.84	0.905	0.36	1.48	39.00	43.86
LD3	53.6	0.02	0.04	8	8	8	0.92	3.81	0.853	0.23	1.14	14.07	22.68
LD4	197.2	0.02	0.04	8	8	28	1.10	4.85	9.050	0.37	1.47	10.68	45.67

Note: Calculations were performed using the HYDROCALC HYDRAULICS for Windows program developed by Dodson and Associates (Version 2.01, 1996-2010).

FORT WORTH C&D LANDFILL
0771-356-11-35
CHUTE ANALYSIS
NORMAL DEPTH CALCULATIONS FOR
FML LINED CHUTES

Drainage Area	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)
SIDESLOPE AREAS													
LD1	212.1	0.33	0.01	2	2	8	0.49	48.65	12.951	36.77	37.26	4.36	9.94
LD2	188.8	0.33	0.01	2	2	8	0.45	46.71	12.831	33.90	34.35	4.04	9.82
LD3	53.6	0.33	0.01	2	2	8	0.22	29.49	11.480	13.52	13.73	1.82	8.86
LD6	197.2	0.33	0.01	2	2	8	0.47	47.42	12.876	37.95	35.42	4.16	9.86

Drainage Area	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)
LOW WATER CROSSING (2%) AREAS													
LD1	212.1	0.02	0.04	8	8	30	1.11	4.9	0.909	0.37	1.49	43.24	47.79
LD2	188.8	0.02	0.04	8	8	26	1.12	4.84	0.905	0.36	1.48	39	43.86
LD3	53.6	0.02	0.04	8	8	8	0.92	3.81	0.853	0.23	1.14	14.07	22.68
LD4	197.2	0.02	0.04	8	8	28	1.10	4.85	9.05	0.37	1.47	10.68	45.67

Note: Calculations were performed using the HYDROCALC HYDRAULICS for Windows program developed by Dodson and Associates (Version 2.01, 1996-2010).

Example Calculation: Calculate the normal depth for the chute for the 33% slope portion of drainage area LD1.

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 = z-ratio (ratio of run to rise for channel sideslope)
A_f = flow area, sf
g = gravitational acceleration = 32.2 ft/s^2
T = top width of flow, ft
d = normal depth of chute, ft

The program uses an iterative process to calculate the normal depth of the chute to satisfy Manning's Equation

$$Q = \frac{1.486}{n} A R^{0.67} S^{0.5}$$

Design Inputs:

Q_d =	212.1	cfs (from HEC-HMS analysis, Appendix IIIF-A)
S =	0.33	ft/ft
b =	8	ft
z =	3	(H) : 1 (V)
n =	0.04	

Step 1 - Based on the geometry of the chute cross-section, solve for R and A_f

$$R = \frac{bd + zd^2}{b + 2d(z^2 + 1)^{0.5}}$$

$$A_f = bd + zd^2$$

assume: d = 1.04 ft

R = 0.793 ft

A_f = 11.59 sf

solve for Q: Q = 212.1 cfs

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

$Q = VA \Rightarrow \quad V = Q/A$

$V = 18.30$ ft/s

$T = b + 2(z \times d)$

$T = 14.25$ ft

$$F_r = \frac{V}{(gA/T)^{0.5}}$$

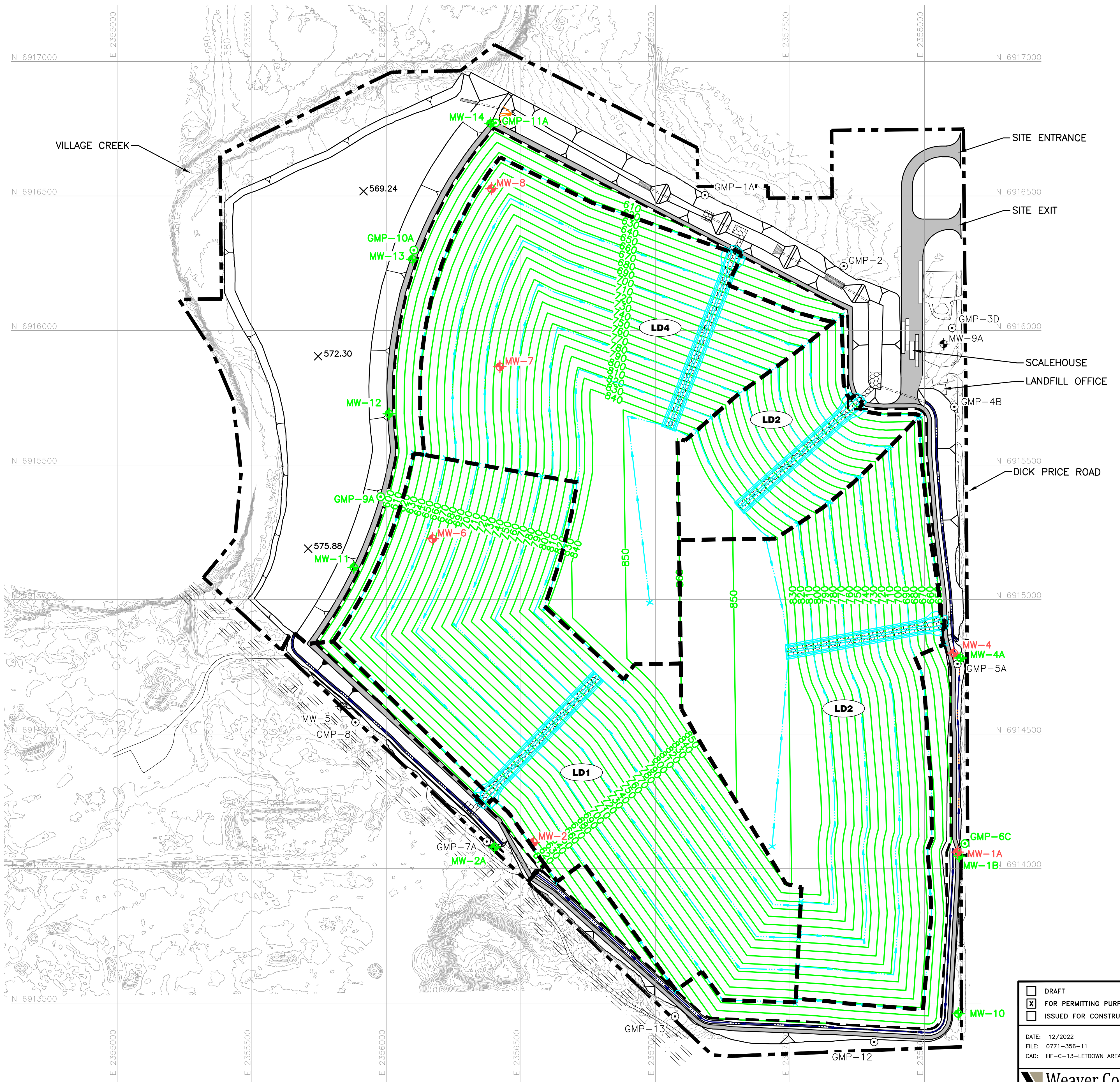
$F_r = 3.577$

$$\text{Velocity Head} = \frac{V^2}{2g}$$

Velocity Head = 5.20 ft

Energy Head = water elevation + velocity head

Energy Head = 6.24 ft



0 200 400
SCALE IN FEET

LEGEND

- PERMIT BOUNDARY
- LIMIT OF WASTE
- 630 EXISTING CONTOUR
- N 6915000 STATE PLANE COORDINATE
- 560 FINAL COVER CONTOUR
- DRAINAGE LETDOWN
- DRAINAGE SWALE
- PROPOSED PERIMETER CHANNEL
- GABIONS
- MW-7 EXISTING/PERMITTED GROUNDWATER MONITORING WELL
- MW-9A APPROVED GROUNDWATER MONITORING WELL
- MW-4 PERMITTED GROUNDWATER MONITORING WELL (TO BE ABANDONED)
- GMP-4B PERMITTED GAS MONITORING PROBE
- DRAINAGE DIVIDE
- LD1 DRAINAGE AREA DESIGNATION

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS PROVIDED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN 02-17-2022.
2. REFER TO APPENDIX IIIF-SURFACE WATER DRAINAGE PLAN FOR DRAINAGE DESIGN INFORMATION.
3. MAXIMUM FINAL COVER ELEVATION 860 FT-MSL.

DRAINAGE AREA NO.	AREA (ACRES)
LD1	35.42
LD2	32.15
LD3	8.98
LD4	33.08

STATE OF TEXAS
CHARLES R. MARSH
105073
LICENSED
PROFESSIONAL ENGINEER
02/09/2023

☐ DRAFT
☒ FOR PERMITTING PURPOSES ONLY
☐ ISSUED FOR CONSTRUCTION

DATE: 12/2022
FILE: 0771-356-11
CAD: IIIF-C-13-LETDOWN AREAS.DWG

DRAWN BY: JDW
DESIGN BY: BPY
REVIEWED BY: CRM

PREPARED FOR
TEXAS REGIONAL LANDFILL COMPANY, LP

REVISIONS

NO.	DATE	DESCRIPTION

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

MAJOR PERMIT AMENDMENT LETDOWN STRUCTURE DRAINAGE AREAS

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

WWW.WCGRP.COM

DRAWING IIIF-C-13

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O:\0771\356\EXPANSION 2022\PART III\IIIF\IIIF-C\IIIF-C-13 LETDOWN AREAS.dwg, jwilson, 1:2

Required: Determine the hydraulic properties for the grouted ripraps as energy letdown structures (chutes).

Method:

1. Calculate the design flow rate of the chute section.
2. Estimate the normal and flow velocity from Hydrocalc using calculated design flow rate.
3. Calculate the critical depth and critical flow velocity.
4. Calculate the height of the roughness element and spacing between the rows of the roughness elements.
5. Calculate the total length of roughness elements.

References:

1. Henry M. Morris, *Hydraulic Dissipation in Steep, Rough Channels* , Bulletin19, Research Division, Virginia Polytechnic Institute, 1968.
2. "Open Channel Hydraulics" by V.T. Chow.
3. "Hydraulic Design of Energy Dissipators for Culverts and Channels", FHWA Hydraulics Engineering Circular Number 14, Third Edition.
4. "Hydraulic Considerations for Corrugated Plastic Pipes" Plastic Pipe Institute.
5. "Reclamation Managing Water in the West" Erosion and Sedimentation Manual. US Department of the Interior Bureau of Reclamation, November 2006.
6. Fort Bend County, Texas, Drainage District "Drainage Criteria Manual", 2nd Revision, February, 2011. Interim Atlas 14 Drainage Criteria Manual and Minimum Slab Elevation Criteria December, 2019.

Solution:

The design of energy dissipators for the 33.3 percent sideslope is based on tumbling flow in the chute. Tumbling flow consists of a series of hydraulic jumps on overfalls that maintain the critical velocity in the chute.

1. For Chute LD1 (For the Upper Portion of a FML Chute):

1.A Design flow rates for energy dissipation.

According to the definition of the unit flow rate,

q = Q/b

Where:
Q = Design flow rate for channel, cfs
b = Bottom width of chute, ft
q = Unit flowrate, cfs/ft of chute width

Q = 212.1 cfs
b = 8 ft

q = 26.51 cfs/ft

1.B. Estimate the normal depth and flow velocity from Hydrocalc using the design flow rate and appropriate Manning's coefficient.

Where:
n = Manning's roughness coefficient
S = channel slope, ft/ft
b = Width of the channel, ft
z = z-ratio (ratio of run to rise for channel sideslope) for side slope
d = Normal Depth of the channel
v = Flow Velocity in the channel

Q = 212.1 cfs
n = 0.01
S = 0.33 ft/ft
z = 2 ft/ft
b = 8 ft

From Hydrocalc

d = 0.49 ft
v = 48.65 ft/sec

1.C For Chute LD1 (For the Lower Portion of the Chute):

Design flow rates for energy dissipation.

According to the definition of the unit flow rate,

q = Q/b

Where:
Q = Design flow rate for channel, cfs
b = Bottom width of chute, ft
q = Unit flowrate, cfs/ft of chute width

Q = 212.1 cfs
b = 30 ft

q = 7.07 cfs/ft

2. Estimate the normal and flow velocity due to the roughness elements from Hydrocalc using flow rate and appropriately adjusted Manning's coefficient.

The roughness coefficient can be calculated from Equation 5-12 from Reference 2

$n = (n_0+n_1+n_2+n_3+n_4)\ m_5$ (Equation 5-12, Reference 2)

Where:	n_0	basic n value for straight, uniform, smooth channel based on material = 0.025	(Reference 2, Page 111, Table 5-6)
	n_1	value added for surface irregularities = 0.01	(Reference 2, Page 109, Table 5-5)
	n_2	value added for variation in channel cross section= 0.0	(Reference 2, Page 109, Table 5-5)
	n_3	value added for obstructions = 0.015	(Reference 2, Page 109, Table 5-5)
	n_4	value added for vegetation and flow conditions = 0.001	(Reference 2, Page 109, Table 5-5)
	m_5	correction factor for meandering of channel =1.0	(Reference 2, Page 109, Table 5-5)

n = (0.025+0.01+0.0+0.015+0.001)*1.0
n = 0.055

Therefore:
Q = 212.1 cfs
n = 0.055
S = 0.33 ft/ft
z = 3 ft/ft
b = 30 ft

From Hydrocalc

d = 0.62 ft
v = 10.81 ft/sec

3. Calculate the critical depth and critical flow velocity.

$Y_c = (q2/g)^{1/3}$ (Reference 3, Equation 7.1)
$V_c = (gq)^{1/3}$ (Reference 3)

Where:
Y_c = Critical depth, ft
q = Unit flowrate, cfs/ft of channel width
g = Acceleration due to gravity = 32.2 ft/s^2
V_c = Critical velocity, ft/s

q = 7.07 cfs

Y_c = 1.16 ft
V_c = 6.11 ft/s

4. Calculate the height of the roughness element and spacing between the rows of the roughness elements.

$h = Y_c/((3-3.7S)^{(2/3)})$ (Reference 3, Equation 7.2)

Where:
Y_c = Critical depth, ft
S = Channel slope, ft/ft
h = Element height, ft

S = 0.33 ft/ft

h = 0.79 ft
h = 9.5 in

$h_{provided}$ = 12.0 in

$h_{provided}$ > h, so the design is adequate.

5. Calculate the total length of roughness elements.

L = 8.5*h (Reference 3)

Where:
L = Spacing between the roughness elements, ft
h = Element height, ft
L_{Total} = Total length of roughened section, ft

L = 7.29 ft

The spacing and height of the roughness elements are designed based on 5 rows of roughness elements. (Reference 3)

$L_{total\ (recommended)}$ = L5

$L_{total\ (recommended)}$ = 36.5

$L_{total(provided)}$ = 40.00 ft

$L_{total(provided)} \geq L_{total\ (recommended)}$, so the design is adequate.

The following table summarizes the calculations for gabion chutes.

Upper Portion of Chutes

Chute	[1]Q	W_{Design}	q	n-value	Bottom Slope	Side Slope	Normal Depth	Flow Velocity
	(cfs)	(ft)	(cfs/ft)		(ft/ft)	(ft/ft)	(ft)	(ft/sec)
LD1	212.1	8	26.51	0.04	0.33	3	1.04	18.30
LD2	188.8	8	23.60	0.04	0.33	3	0.98	17.67
LD3	53.6	8	6.70	0.04	0.33	3	0.48	11.82
LD4	197.2	8	24.65	0.04	0.33	3	1.00	17.89

Lower Portion of Chutes

Chute	[1]Q	W_{Design}	q	n-value	Bottom Slope	Side Slope	Normal Depth	Flow Velocity	Y_c	V_c	h	L (=9.25h)	h_{Design}	[2]L_{Total} (Recommended)	$W_{Provided}$	$h_{Provided}$	L_{Total} (Provided)
	(cfs)	(ft)	(cfs/ft)		(ft/ft)	(ft/ft)	(ft)	(ft/sec)	(ft)	(fps)	(ft)	(ft)	(in)	(ft)	(ft)	(in)	(ft)
LD1	212.1	30	7.07	0.055	0.33	3	0.62	10.81	1.16	6.11	0.79	7.3	9.4	36.4	30	12.0	40.0
LD2	188.8	26	7.26	0.055	0.33	3	0.63	10.84	1.18	6.16	0.80	7.4	9.6	37.1	26	12.0	40.0
LD3	53.6	8	6.70	0.055	0.33	3	0.58	9.55	1.12	6.00	0.76	7.0	9.1	35.2	8	12.0	40.0
LD4	197.2	28	7.04	0.055	0.33	3	0.61	10.75	1.15	6.10	0.79	7.3	9.4	36.3	28	12.0	40.0

1. The flowrates were reproduced from Appendix IIIF-A.
2. Total length of the roughened section was calculated based on FHWA recommendation of 5 rows of roughened elements.

Required: Provide topslope and sideslope anchor trench design for a geomembrane-lined letdown structure (or chute).

Method:

1. Design anchor trench spacing and depths.
2. Design upstream end anchor trench.

Assumptions:

1. The geomembrane-lined chute will transition to its maximum width for the energy dissipater design where maximum total flow for chute is expected to occur.
2. Proposed chutes will convey runoff from the following chute drainage area:

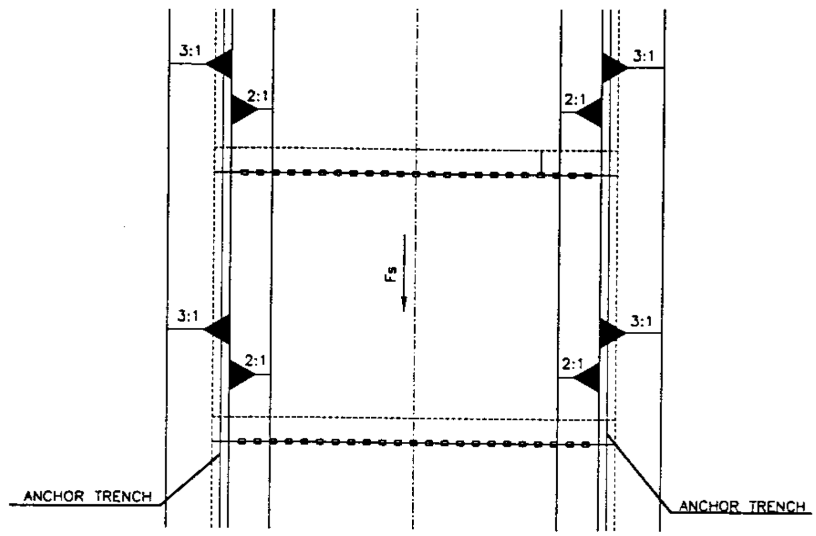
Proposed Chute	Chute Drainage Areas	25-year Total Flow (cfs)[1]
1	LD1	212.1
2	LD2	188.8
3	LD3	53.6
4	LD4	197.2

[1] From HEC-HMS Analysis, Appendix IIIF-A

References:

1. Gamelsky, S.G., *Innovations in Stormwater Management for Landfill Closure* Technical Paper
2. Koerner, R.M., *Designing with Geosynthetics* , 5th Edition, Prentice-Hall, Inc, 2005.
3. Morris, H.M., *Hydraulics of Energy Dissipators in Steep Rough Channels* , Bulletin 19, Research Division, Virginia Polytechnic Institute, Blacksburg, Virginia.

Design anchor trench spacing and depths.



Shear force pulling on geomembrane due to water:

The shear force acting on the geomembrane per square foot of water in the chute:

$T = \gamma_w \times D \times S$ where: γ_w = unit weight of water (lb/cf)
D = maximum water depth (ft)
S = hydraulic gradient (ft/ft)

Shear force acting on the geomembrane per foot of anchor trench:

$F_{s1} = T \times P$

where:
P = wetted perimeter of the chute = $(W + 2 \times (a^2+D^2)^{1/2})$
$a = h \times D$ = horizontal distance from bottom of chute to the depth submerged on the sideslopes
h = Slope of sidewalls = 2 (H) : 1 (V)
W = Minimum bottom width of flow = 8 ft

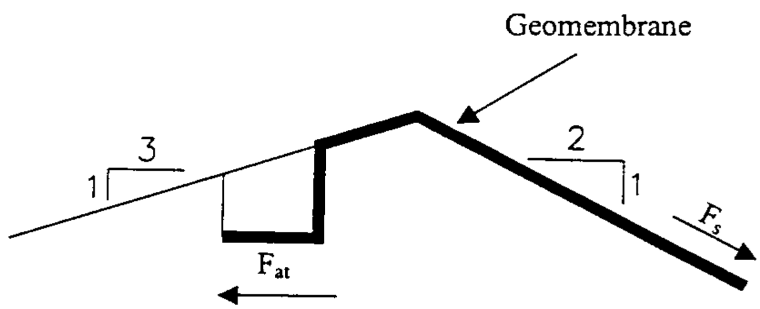
Conservatively, the maximum calculated water depth in the chutes will be used to verify the design. Thus, the water depth in the narrowest part of the chute with the highest depth will be used.

Letdown	Maximum Water Depth (ft)[1]	Hydraulic Gradient (ft/ft)	T (lb/sf)	a (ft)	F_{s1} (lb/ft)
LD1	0.49	0.33	10.18	0.98	104
LD2	0.45	0.33	9.35	0.9	94
LD3	0.22	0.33	4.57	0.44	41
LD4	0.47	0.33	9.77	0.94	99

[1] See design depths on page IIIF-C-9 and IIIF-C-10

Pullout Resistance from Edges, F_{at1} [1]

Assuming pullout only opposed by trench (conservative assumption)



$F_{at} = 2[\{K_o\gamma(D/2)\}\{\tan\zeta\}\{D\} + \{\gamma D\}\{\tan\zeta\}\{w\}]$ (Ref 3)

where: ζ = interface friction angle
K_o = 1 - sin ζ
γ = unit weight of soil (lb/cf)
D = depth of anchor trench (ft)
w = bottom width of anchor trench (ft)

soil friction angle =	21	degrees	(CL,SC)
soil/geomembrane friction angle =	18.2	degrees	
unit weight =	112	lb/ft^3	
depth of anchor trench =	1	ft	
bottom width of anchor trench =	1	ft	

[1]See detail D10 - Anchor Trench Type 2 on Drawing IIIF.9 for dimensions.

K_o = 0.64

F_{at1} =	114	lb/ft width on one side

Factor of Safety = $2F_{at1}/F_{s1}$ =	$\frac{227}{104}$	FS =	2.2

3. **Upstream End Anchor Trench Design**

Shear force pulling on geomembrane due to water:

F_{s2} = T x A

where: T = Maximum shear force acting on the geomembrane per square foot of water in the chute (lb/sf)

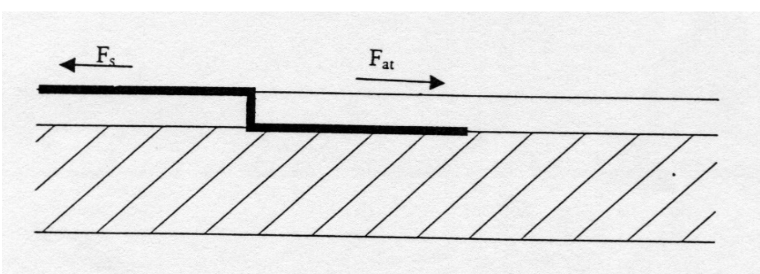
A = area of geomembrane at the top of the chute (ft^2)

Area of geomembrane at top of chute = 116 ft x 17 ft = 1,972 sf

Conservatively, use the maximum shear force per square foot calculated in Part 2

F_{s2} =	20,079	lbs

Pullout resistance of upstream end, F_{at2} [2]



$F_{at} = 2[\{K_o\gamma(D/2)\}\{\tan\zeta\}\{D\} + \{\gamma D\}\{\tan\zeta\}\{w\}]$ (Ref 3)

where:
ζ = interface friction angle
K_o = 1 - sin ζ
γ = unit weight of soil (lb/cf)
D = depth of anchor trench (ft)
w = bottom width of anchor trench (ft)

friction angle =	18.2	degrees
anchor trench soil unit weight =	112	lb/ft^3
depth of anchor trench =	0.5	ft
bottom width of anchor trench =	3	ft

K_o = 0.69

F_{at2}= 117 lb/ft width

Total End Anchor Length (L_T) [4] = 150 ft

F_{pr} = Pullout Resistance (End) = F_{at2} x L_T = 17,520 lbs

Factor of Safety = F_{pr}/F_{s2} = $\frac{17,520}{20,079}$ FS = 0.9

Summary of Results

Side Anchor Trench Pullout resistance:

FS = $\frac{2F_{AT2}}{F_{S1}}$ ==> FS = 2.2

Upstream End Anchor Trench Pullout resistance:

FS = $\frac{F_{pr}}{F_{s2}}$ ==> FS = 0.9

As it is stated on page 557 of Reference 3, the typical factors of safety for the proposed anchor trenches are between 0.7 to 5.0. Therefore, the design is acceptable.

APPENDIX IIIF-D

EROSION LAYER EVALUATION

Includes pages IIIF-D-1 through IIIF-D-36

STATE OF TEXAS

CHARLES R. MARSH

105073

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PROFESSIONAL ENGINEER

02/09/2023

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 Fort Worth C&D 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 Tarrant County, the approximate depth of frost penetration is approximately 6 inches (see IIIF-D-10). 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 5 percent topslope and 33 percent side slope require a minimum of 6.144 inches and 7.388 inches, respectively, for the erosion layer. These USLE requirements include the 6-inch minimum required by regulations. Conservatively, a 12-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, ACB, or FML to prevent erosion given that the velocities in the letdowns are over 5 ft/sec.

   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 nonerosive velocity is defined as 5.0 ft/sec or less. Calculated sheet flow velocities range from 0.30 to 0.44 ft/sec for topslope and sideslope cases. The supporting calculations are presented on pages IIIF-D-20 through IIIF-D-28.

Channelized flow for drainage swales is also calculated to be less than permissible nonerosive velocities. Calculated channelized flow velocities range from 2.01 to 3.18 ft/sec for the drainage swales. The supporting calculations are presented on pages IIIF-C-3 through IIIF-C-7.

4. Vegetation for the site will be native and introduced grasses with root depths of 6 inches to 8 inches. The erosion layer shall also include a mixture of Bermuda, vetch, rye, wheat grass, wild flowers, and flowering plants. The seeding is specified on the attached pages IIIF-D-29 through IIIF-D-35. The seeding included on pages IIIF-D-29 through IIIF-D-36 is specified by TxDOT for temporary and permanent erosion control for Tarrant County, Texas (Fort Worth District).

5. 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.

6. Slope stability information is included in Appendix IIIM.

Required: Determine expected soil loss and minimum thickness for the erosion layer.

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, National Resource Conservation Service, Web Soil Survey for Tarrant County, Texas (http://websoilsurvey.nrcs.usda.gov).
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 Tarrant County is:

R = 290

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 consist of clay with high organic content. Clean compost as a soil amendment may be added to final cover top soil as necessary to protect against erosion. Therefore, the following is a K value for the site.

K = 0.25

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 side slope and top slope conditions.
See sheet IIIF-D-7 for the locations of the slopes analyzed.

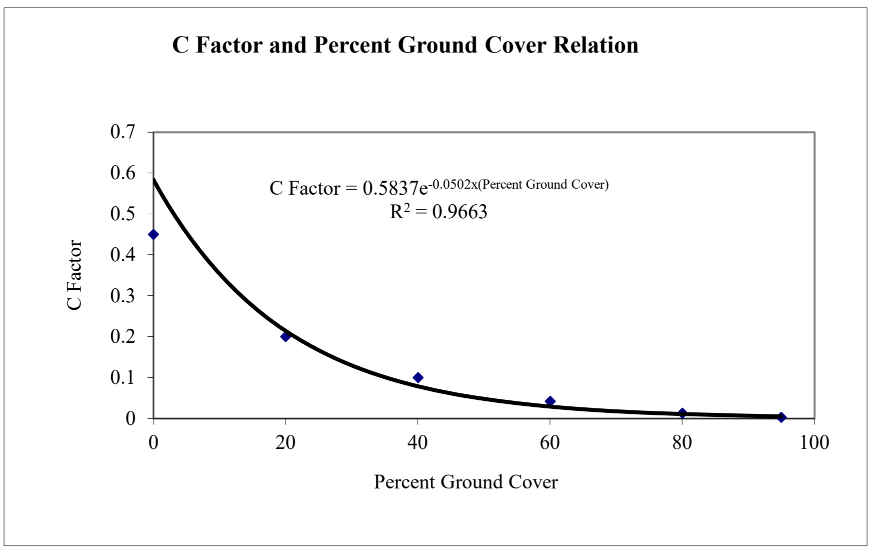
Case 1.	Typical Top Slope			Case 2.	Longest Top Slope		
	slope =	5	%		slope =	5	%
	length =	377	ft		length =	503	ft
Case 3.	Typical Side Slope			Case 4.	Longest Side Slope (25%)		
		33.3	%			33.3	%
		120	ft			127	ft

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. Typical Top Slope	5	377	1.04
2. Longest Top Slope	5	503	1.20
3. Typical Side Slope	33.3	120	10.00
4. Longest Side Slope	33.3	127	11.00

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= $0.5837e^{(-0.0502x90)}$
C Factor= 0.0064 (for 90% 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)
1. Typical Top Slope 5% slope 377 ft length	290	0.25	1.04	0.0064	1.00	0.48
2. Longest Top Slope 5% slope 503 ft length	290	0.25	1.20	0.0064	1.00	0.55
3. Typical Side Slope 33% slope 120 ft length	290	0.25	10.00	0.0064	1.00	4.62
4. Longest Side Slope 33% slope 127 ft length	290	0.25	11.00	0.0064	1.00	5.08

Note: Erosion layer will be maintained to provide 90% ground cover.

3. Erosion layer thickness calculations:

$$T_{el} = 6\text{in} + \frac{AYF(2000\text{lb/ton})(12\text{in/ft})}{w(43{,}560\text{sf/ac})}$$

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)

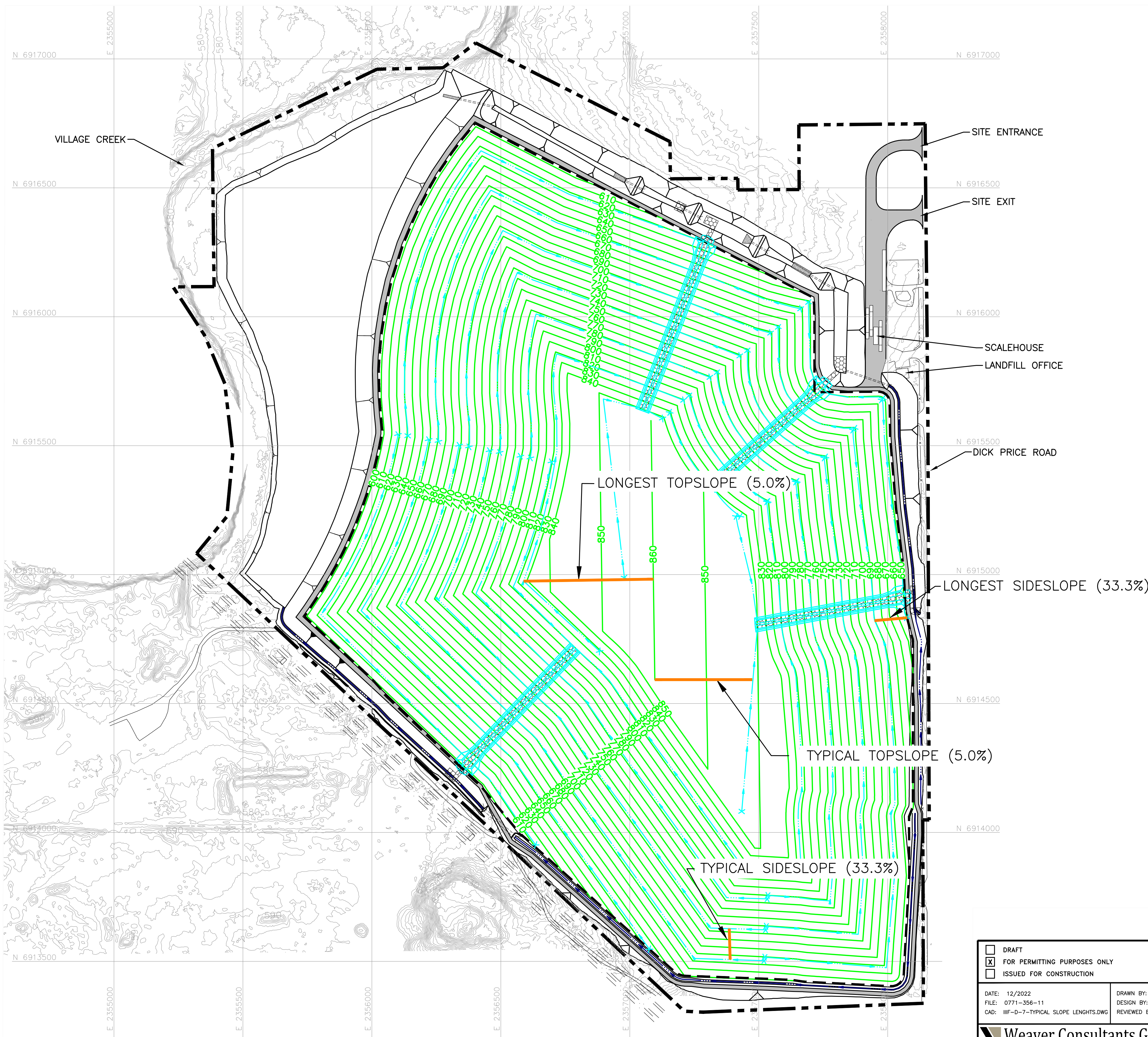
Y = 30 yr
F = 2
w = 110 pcf

1. Typical Top Slope Thickness:		
T_{el}, Required thickness[1] =	6.144	in
Total estimated soil loss =	0.144	in
Minimum Specified thickness =	12.000	in
2. Longest Top Slope Thickness:		
T_{el}, Required thickness[1] =	6.167	in
Total estimated soil loss =	0.167	in
Minimum Specified thickness =	12.000	in
3. Typical Sideslope Thickness:		
T_{el}, Required thickness[1] =	7.388	in
Total estimated soil loss =	1.388	in
Minimum Specified thickness =	12.000	in
4. Longest Sideslope Thickness:		
T_{el}, Required thickness[1] =	7.526	in
Total estimated soil loss =	1.526	in
Minimum Specified thickness =	12.000	in

Note: [1]Required thicknesses include 6 inch minimum required and estimated soil loss.

4. Summary:

Calculated erosion losses are shown in Step 2 above.
The erosion layer will be a minimum of 12 inches thick.
As shown above, this is a conservative design considering
the maximum expected soil loss for a 30 year period is 1.526 inches.



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0 200 400

SCALE IN FEET

LEGEND

- PERMIT BOUNDARY
- LIMIT OF WASTE
- 630 EXISTING CONTOUR
- N 6915000 STATE PLANE COORDINATE
- 560 FINAL COVER CONTOUR
- DRAINAGE LETDOWN
- DRAINAGE SWALE
- PROPOSED PERIMETER CHANNEL
- GABIONS

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS PROVIDED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN 02-17-2022.

STATE OF TEXAS
CHARLES R. MARSH
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LICENSED
PROFESSIONAL ENGINEER
02/09/2023

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☒ FOR PERMITTING PURPOSES ONLY	TEXAS REGIONAL LANDFILL COMPANY, LP
☐ ISSUED FOR CONSTRUCTION	

DATE: 12/2022	DRAWN BY: JDW
FILE: 0771-356-11	DESIGN BY: BPY
CAD: IIIF-D-7-TYPICAL SLOPE LENGHTS.DWG	REVIEWED BY: CRM

Weaver Consultants Group

TBPE REGISTRATION NO. F-3727

REVISIONS

NO.	DATE	DESCRIPTION

EROSION LAYER EVALUATION TYPICAL SLOPE LENGTHS

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

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DRAWING IIIF-D-7

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O:\0771\356\EXPANSION 2022\PART III\IIIF\IIIF-D\IIIF-D-7 TYPICAL SLOPE LENGHTS.dwg, jwilson, 1:2

SOIL LOSS ESTIMATE SUMMARY TABLE

Case	Slope (%)	Length (ft)	L_s	Percent Ground Cover	C Factor	A (tons/ac/yr)
Top Slope	5	377	1.04	60	0.042	3.2
Top Slope	5	377	1.04	70	0.017	1.3
Top Slope	5	377	1.04	80	0.013	1.0
Top Slope	5	377	1.04	90	0.0064	0.5
Top Slope	5	503	1.20	60	0.042	3.7
Top Slope	5	503	1.20	70	0.017	1.5
Top Slope	5	503	1.20	80	0.013	1.1
Top Slope	5	503	1.20	90	0.0064	0.6
Side Slope	33.3	120	10.00	60	0.042	30.5
Side Slope	33.3	120	10.00	70	0.017	12.3
Side Slope	33.3	120	10.00	80	0.013	9.4
Side Slope	33.3	120	10.00	90	0.0064	4.6
Side Slope	33.3	127	11.00	60	0.042	33.5
Side Slope	33.3	127	11.00	70	0.017	13.6
Side Slope	33.3	127	11.00	80	0.013	10.4
Side Slope	33.3	127	11.00	90	0.0064	5.1

United States Environmental Protection Agency
Solid Waste and Emergency Response (5305)
EPA530-R-93-017
November 1993
www.epa.gov/osw

Solid Waste Disposal Facility Criteria

Technical Manual

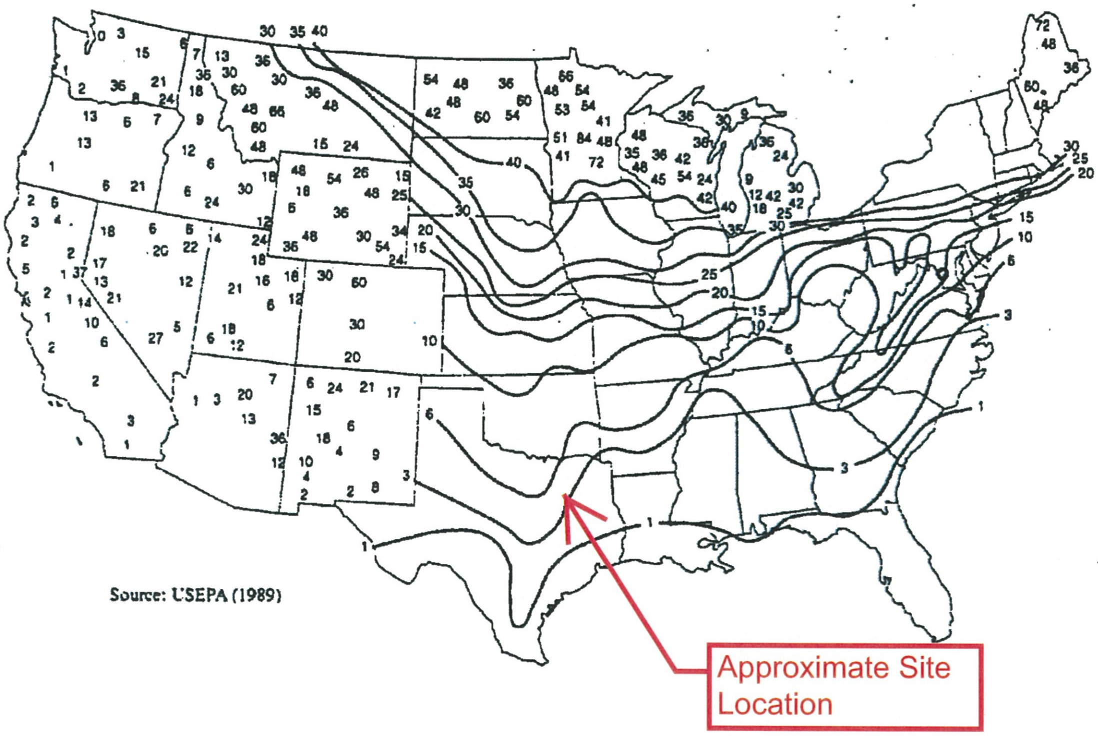
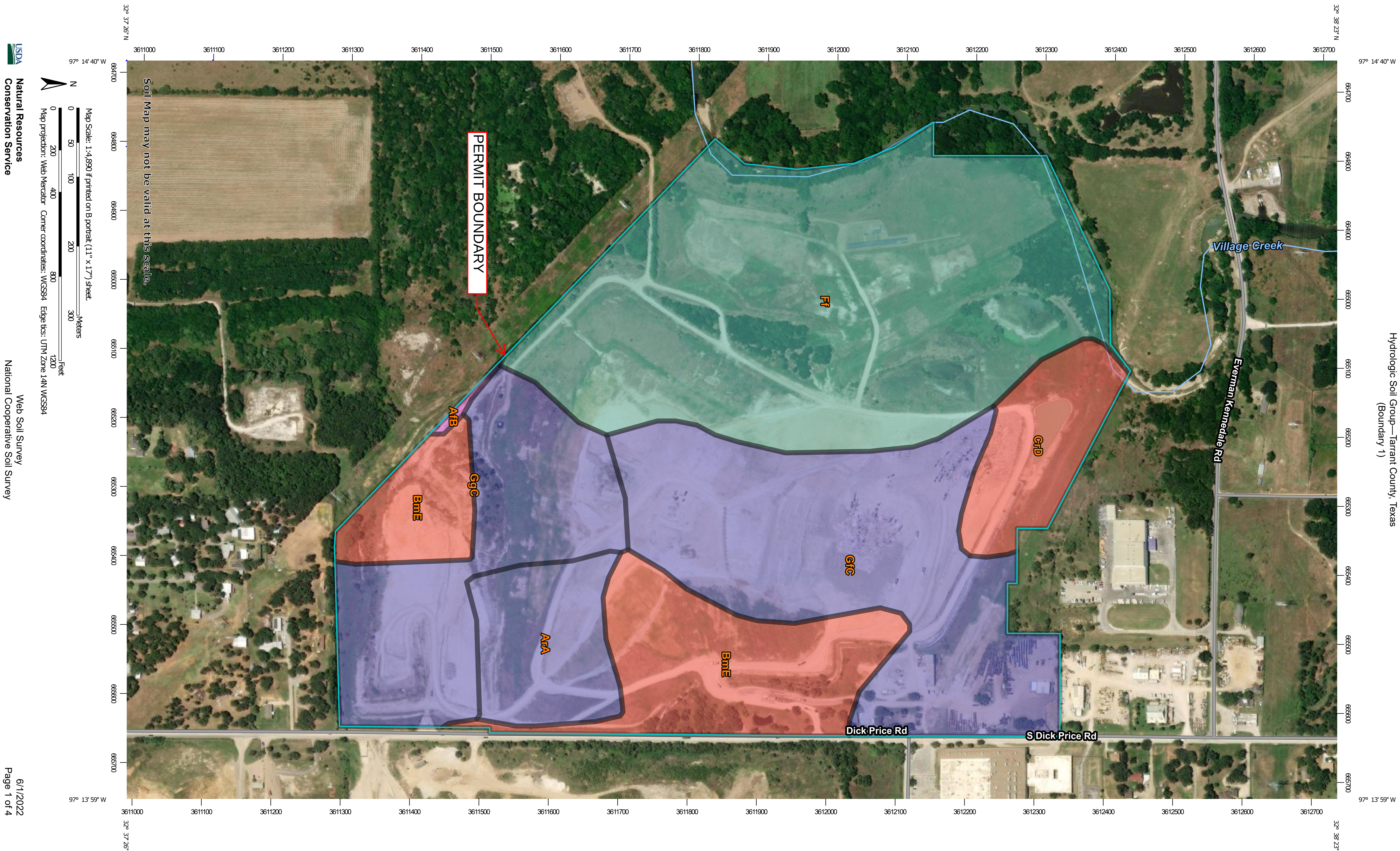


Figure 6-4
Regional Depth of Frost Penetration in Inches



Hydrologic Soil Group—Tarrant County, Texas
(Boundary 1)
Soil Map may not be valid at this scale.
PERMIT BOUNDARY
Ff
GfC
GrD
GgC
BmE
BmE
ArA
AfB
Village Creek
Everman Kennedale Rd
Dick Price Rd
S Dick Price Rd
Map Scale: 1:4,890 if printed on B portrait (11" x 17") sheet.
Meters
0 50 100 200 300
Feet
0 200 400 800 1200
Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 14N WGS84
97° 14' 40" W
97° 13' 59" W
32° 37' 26" N
32° 38' 23" N
Natural Resources
Conservation Service
Web Soil Survey
National Cooperative Soil Survey
6/1/2022
Page 1 of 4

MAP LEGEND

Area of Interest (AOI)

Area of Interest (AOI)

Soils

Soil Rating Polygons

- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Soil Rating Lines

- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Soil Rating Points

- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Water Features

- Streams and Canals

Transportation

- Rails
- Interstate Highways
- US Routes
- Major Roads
- Local Roads

Background

- Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:20,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Tarrant County, Texas
Survey Area Data: Version 19, Sep 10, 2021

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Jan 18, 2020—Nov 15, 2020

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Hydrologic Soil Group

Map unit symbol	Map unit name	Rating	Acres in AOI	Percent of AOI
AfB	Arents, frequently flooded	A	0.3	0.2%
ArA	Arents, loamy	B	11.2	6.1%
BmE	Birome-Aubrey-Rayex complex, 5 to 15 percent slopes	D	26.2	14.2%
CrD	Crosstell fine sandy loam, 3 to 8 percent slopes	D	10.3	5.6%
Ff	Frio clay loam, 0 to 1 percent slopes, frequently flooded	C	69.7	37.8%
GfC	Gasil fine sandy loam, 3 to 8 percent slopes	B	42.3	22.9%
GgC	Gasil sandy clay loam, graded, 1 to 5 percent slopes	B	24.3	13.2%
Totals for Area of Interest			**184.4**	**100.0%**

Description

Hydrologic soil groups are based on estimates of runoff potential. Soils are assigned to one of four groups 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 rate of water transmission.

Group B. Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately 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 a 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 that 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 and the second is for undrained areas. Only the soils that in their natural condition are in group D are assigned to dual classes.

Rating Options

Aggregation Method: Dominant Condition

Component Percent Cutoff: None Specified

Tie-break Rule: Higher

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 = 0.25		

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.

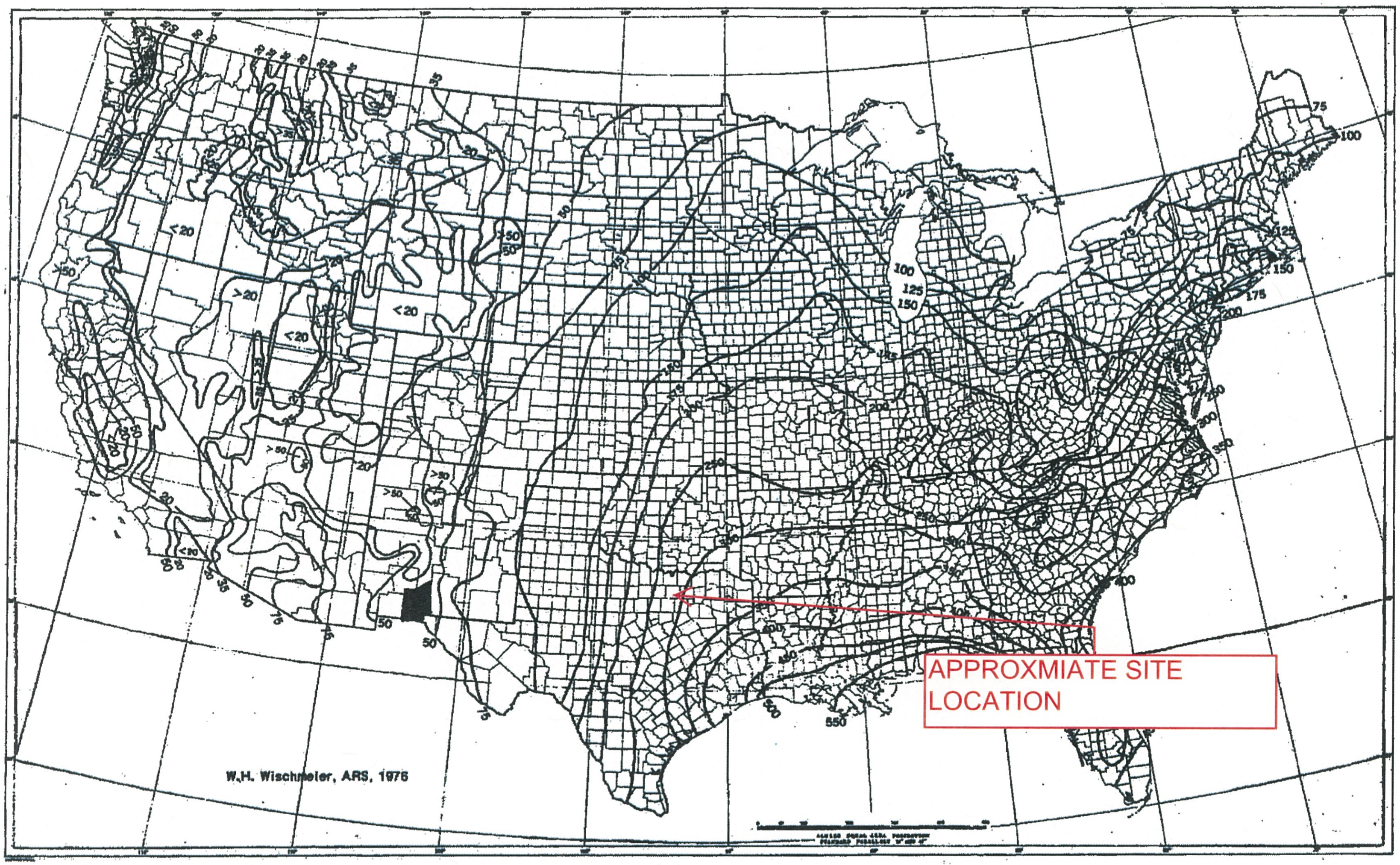


Figure 3-1.—Average annual values of the R factor.

tion and developmental areas can be obtained from table 5 if good judgment is exercised in comparing the surface conditions with those of agricultural conditions specified in lines of the table. Time intervals analogous to cropstage periods will be defined to begin and end with successive construction or management activities that appreciably change the surface conditions. The procedure is then similar to that described for cropland.

Establishing vegetation on the denuded areas as quickly as possible is highly important. A good sod has a C value of 0.01 or less (table 5-B), but such a low C value can be obtained quickly only by laying sod on the area, at a substantial cost. When grass or small grain is started from seed, the probable soil loss for the period while cover is developing can be computed by the procedure outlined for estimating cropstage-period soil losses. If the seeding is on topsoil, without a mulch, the soil loss ratios given in line 141 of table 5 are appropriate for cropstage C values. If the seeding is on a desurfaced area, where residual effects of prior vegetation are no longer significant, the ratios for periods SB, 1 and 2 are 1.0, 0.75 and 0.50, respectively, and line 141 applies for cropstage 3. When the seedbed is protected by a mulch, the pertinent mulch factor from the upper curve of figure 6 or table 9 is applicable until good canopy cover is attained. The combined effects of vegetative mulch and low-growing canopy are given in figure 7. When grass is established in small grain, it can usually be evaluated as established meadow about 2 mo after the grain is cut.

C Values for Pasture, Range, and Idle Land

Factor C for a specific combination of cover conditions on these types of land may be obtained from table 10 (57). The cover characteristics that must be appraised before consulting this table are defined in the table and its footnotes. Cropstage periods and EI monthly distribution data are generally not necessary where perennial vegetation has become established and there is no mechanical disturbance of the soil.

Available soil loss data from undisturbed land were not sufficient to derive table 10 by direct comparison of measured soil loss rates, as was done for development of table 5. However, analyses of the assembled erosion data showed that the research information on values of C can be extended to completely different situations by combining subfactors that evaluate three separate and distinct, but interrelated, zones of influence: (a) vegetative cover in direct contact with the soil surface, (b) canopy cover, and (c) residual and tillage effects.

Subfactors for various percentages of surface cover by mulch are given by the upper curve of

TABLE 10.—*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]	Type[4]	Percent ground cover					
			0	20	40	60	80	95+
No appreciable canopy		G	0.45	0.20	0.10	0.042	0.013	0.003
		W	.45	.24	.15	.091	.043	.011
Tall weeds or short brush with average drop fall height of 20 in	25	G	.36	.17	.09	.038	.013	.003
		W	.36	.20	.13	.083	.041	.011
	50	G	.26	.13	.07	.035	.012	.003
		W	.26	.16	.11	.076	.039	.011
	75	G	.17	.10	.06	.032	.011	.003
		W	.17	.12	.09	.068	.038	.011
Appreciable brush or bushes, with average drop fall height of 6½ ft	25	G	.40	.18	.09	.040	.013	.003
		W	.40	.22	.14	.087	.042	.011
	50	G	.34	.16	.08	.038	.012	.003
		W	.34	.19	.13	.082	.041	.011
	75	G	.28	.14	.08	.036	.012	.003
		W	.28	.17	.12	.078	.040	.011
Trees, but no appreciable low brush. Average drop fall height of 13 ft	25	G	.42	.19	.10	.041	.013	.003
		W	.42	.23	.14	.089	.042	.011
	50	G	.39	.18	.09	.040	.013	.003
		W	.39	.21	.14	.087	.042	.011
	75	G	.36	.17	.09	.039	.012	.003
		W	.36	.20	.13	.084	.041	.011

[1] The listed C values assume that the vegetation and mulch are randomly distributed over the entire area.

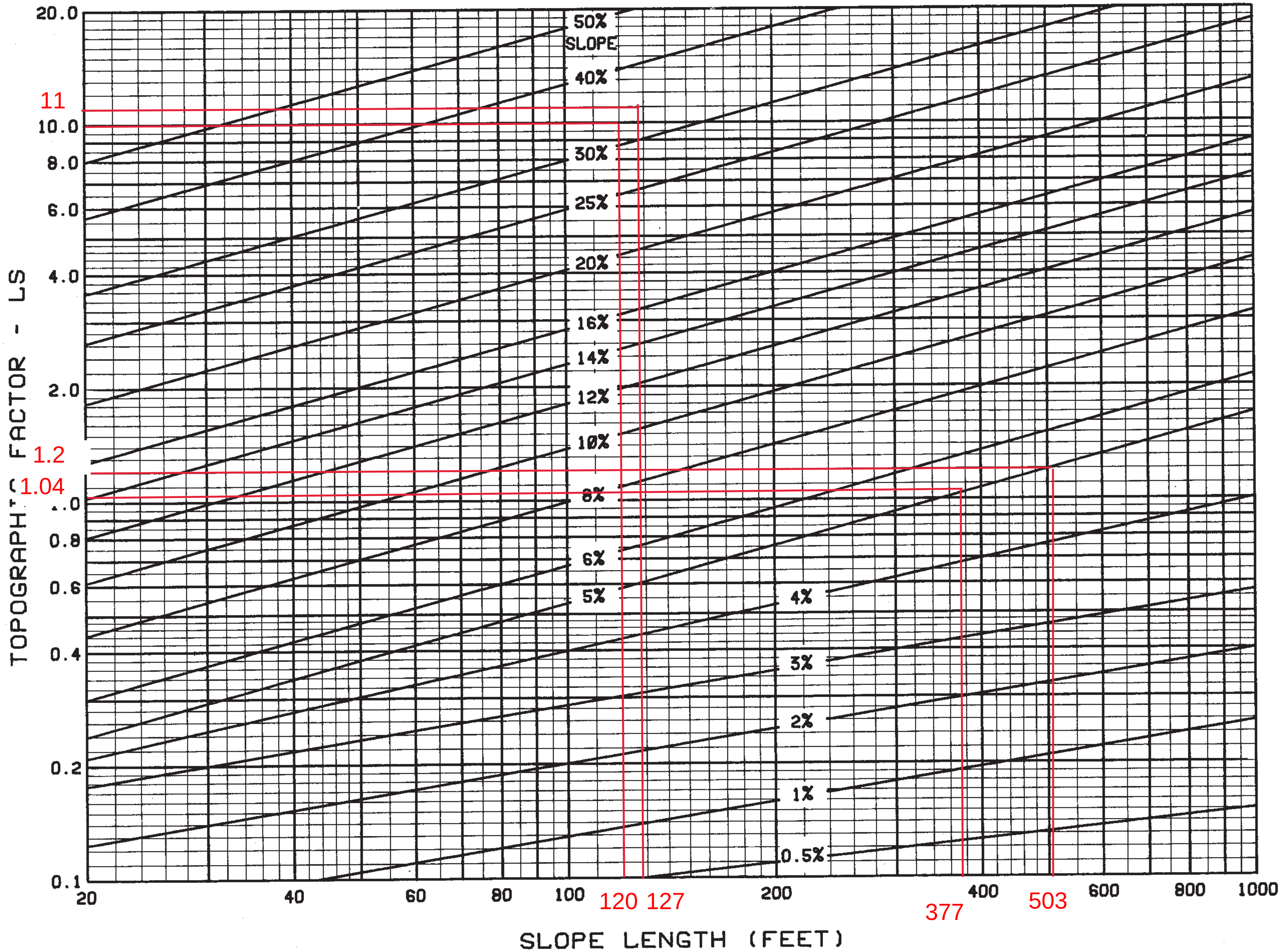
[2] Canopy height is measured as the average fall height of water drops falling from the canopy to the ground. Canopy effect is inversely proportional to drop fall height and is negligible if fall height exceeds 33 ft.

[3] Portion of total-area surface that would be hidden from view by canopy in a vertical projection (a bird's-eye view).

[4] G: cover at surface is grass, grasslike plants, decaying compacted duff, or litter at least 2 in deep.

W: cover at surface is mostly broadleaf herbaceous plants (as weeds with little lateral-root network near the surface) or undecayed residues or both.

FIGURE 4.—Slope-effect chart (topographic factor, LS). LS = $(\lambda/72.6)^m (65.41 \sin^2\theta + 4.56 \sin\theta + 0.065)$ where λ = slope length in feet; θ = angle of slope; and m = 0.2 for gradients < 1 percent, 0.3 for 1 to 3 percent slopes, 0.4 for 3.5 to 4.5 percent slopes, and 0.5 for slopes of 5 percent or steeper.



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. NOAA Atlas 14 - Precipitation-Frequency Atlas of the United States, Volume 11, Version 2.0: Texas
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.

Case 1. Typical top slope

slope =	0.05	ft/ft
length =	377	ft

Case 2. Longest top slope

slope =	0.05	ft/ft
length =	503	ft

Case 3. Typical side slope

slope =	0.33	ft/ft
length =	120	ft

Case 4. Longest side slope

slope =	0.33	ft/ft
length =	127	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)

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

Determine $P_{2,24}$:

$P_{2,24}$ = 3.91 in (ref 2)

Calculate t_c:

Case 1:

n =	0.24	
L =	377	
$P_{2,24}$ =	3.91	
S =	0.05	
t_c =	0.43	hr
	25.9	min

Case 2:

n =	0.24	
L =	503	
$P_{2,24}$ =	3.91	
S =	0.05	
t_c =	0.54	hr
	32.6	min

Case 3:

n =	0.24	
L =	120	
$P_{2,24}$ =	3.91	
S =	0.33	
t_c =	0.08	hr
	4.87	min

Case 4:

n =	0.24	
L =	127	
$P_{2,24}$ =	3.91	
S =	0.33	
t_c =	0.08	hr
	4.87	min

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.

NOAA Atlas 14, Volume 11, Version 2
Location name: Fort Worth, Texas, USA*
Latitude: 32.6287°, Longitude: -97.2423°
Elevation: 592.11 ft**
* source: ESRI Maps
** source: USGS

POINT PRECIPITATION FREQUENCY ESTIMATES

Sanja Perica, Sandra Pavlovic, Michael St. Laurent, Carl Trypaluk, Dale Unruh, Orlan Wilhite

NOAA, National Weather Service, Silver Spring, Maryland

PF_tabular | PF_graphical | Maps_&_aerials

PF tabular

PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches)[1]

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.408 (0.309-0.539)	0.479 (0.365-0.626)	0.594 (0.452-0.780)	0.689 (0.517-0.918)	0.822 (0.598-1.13)	0.925 (0.654-1.30)	1.03 (0.710-1.48)	1.14 (0.766-1.68)	1.29 (0.838-1.97)	1.41 (0.891-2.20)
10-min	0.653 (0.495-0.863)	0.767 (0.585-1.00)	0.952 (0.725-1.25)	1.11 (0.830-1.47)	1.32 (0.960-1.81)	1.49 (1.05-2.09)	1.65 (1.14-2.38)	1.83 (1.23-2.70)	2.06 (1.34-3.13)	2.23 (1.41-3.48)
15-min	0.814 (0.616-1.08)	0.954 (0.728-1.25)	1.18 (0.899-1.55)	1.37 (1.03-1.82)	1.63 (1.19-2.23)	1.83 (1.30-2.58)	2.04 (1.41-2.94)	2.26 (1.52-3.34)	2.56 (1.66-3.90)	2.79 (1.76-4.35)
30-min	1.13 (0.857-1.49)	1.32 (1.01-1.73)	1.64 (1.25-2.15)	1.90 (1.42-2.53)	2.26 (1.64-3.08)	2.53 (1.79-3.55)	2.81 (1.94-4.05)	3.12 (2.09-4.60)	3.53 (2.30-5.39)	3.86 (2.44-6.04)
60-min	1.47 (1.11-1.94)	1.72 (1.32-2.25)	2.14 (1.63-2.81)	2.48 (1.86-3.31)	2.96 (2.15-4.05)	3.33 (2.35-4.67)	3.71 (2.56-5.34)	4.12 (2.77-6.09)	4.69 (3.05-7.17)	5.15 (3.26-8.05)
2-hr	1.80 (1.37-2.36)	2.13 (1.63-2.76)	2.67 (2.04-3.48)	3.12 (2.36-4.13)	3.77 (2.75-5.11)	4.27 (3.04-5.93)	4.79 (3.32-6.83)	5.37 (3.63-7.84)	6.18 (4.03-9.31)	6.83 (4.34-10.5)
3-hr	1.99 (1.53-2.60)	2.38 (1.83-3.06)	3.00 (2.30-3.88)	3.53 (2.67-4.64)	4.28 (3.14-5.77)	4.87 (3.48-6.74)	5.51 (3.83-7.80)	6.19 (4.20-8.98)	7.17 (4.69-10.7)	7.95 (5.07-12.2)
6-hr	2.35 (1.81-3.05)	2.84 (2.19-3.61)	3.59 (2.78-4.62)	4.25 (3.24-5.54)	5.19 (3.84-6.95)	5.95 (4.27-8.15)	6.75 (4.72-9.46)	7.63 (5.19-10.9)	8.86 (5.83-13.1)	9.86 (6.31-14.9)
12-hr	2.77 (2.15-3.56)	3.35 (2.60-4.22)	4.25 (3.31-5.42)	5.03 (3.87-6.51)	6.15 (4.57-8.15)	7.04 (5.09-9.55)	7.98 (5.61-11.1)	9.01 (6.17-12.8)	10.5 (6.91-15.3)	11.6 (7.48-17.3)
24-hr	3.24 (2.52-4.12)	3.91 (3.06-4.90)	4.97 (3.89-6.29)	5.88 (4.54-7.54)	7.17 (5.36-9.41)	8.20 (5.95-11.0)	9.28 (6.56-12.7)	10.5 (7.20-14.6)	12.1 (8.05-17.4)	13.5 (8.70-19.8)
2-day	3.76 (2.95-4.75)	4.52 (3.56-5.62)	5.72 (4.52-7.19)	6.76 (5.26-8.60)	8.24 (6.19-10.7)	9.41 (6.88-12.5)	10.7 (7.57-14.4)	12.0 (8.30-16.6)	13.9 (9.29-19.8)	15.5 (10.0-22.4)
3-day	4.10 (3.23-5.15)	4.92 (3.90-6.10)	6.23 (4.93-7.79)	7.35 (5.74-9.31)	8.94 (6.75-11.6)	10.2 (7.48-13.5)	11.5 (8.23-15.5)	13.0 (9.03-17.8)	15.1 (10.1-21.2)	16.8 (10.9-24.0)
4-day	4.34 (3.44-5.45)	5.22 (4.14-6.45)	6.60 (5.24-8.22)	7.79 (6.10-9.82)	9.48 (7.17-12.2)	10.8 (7.95-14.2)	12.2 (8.75-16.4)	13.8 (9.59-18.8)	16.0 (10.7-22.4)	17.8 (11.6-25.3)
7-day	4.88 (3.88-6.07)	5.86 (4.67-7.18)	7.40 (5.91-9.16)	8.73 (6.87-10.9)	10.6 (8.08-13.6)	12.1 (8.97-15.8)	13.7 (9.86-18.2)	15.5 (10.8-20.9)	17.9 (12.1-24.8)	19.9 (13.0-28.0)
10-day	5.34 (4.26-6.62)	6.39 (5.12-7.82)	8.06 (6.46-9.94)	9.49 (7.50-11.8)	11.5 (8.80-14.7)	13.2 (9.75-17.0)	14.9 (10.7-19.6)	16.7 (11.7-22.5)	19.4 (13.1-26.6)	21.5 (14.1-30.0)
20-day	6.91 (5.55-8.50)	8.11 (6.57-9.89)	10.0 (8.12-12.3)	11.7 (9.29-14.4)	13.9 (10.7-17.5)	15.7 (11.7-20.1)	17.5 (12.7-22.8)	19.6 (13.8-25.9)	22.4 (15.2-30.4)	24.8 (16.3-34.1)
30-day	8.21 (6.63-10.1)	9.54 (7.78-11.6)	11.7 (9.52-14.3)	13.5 (10.8-16.6)	16.0 (12.3-20.0)	17.9 (13.3-22.7)	19.8 (14.4-25.6)	22.0 (15.5-28.9)	25.1 (17.1-33.7)	27.6 (18.2-37.6)
45-day	9.97 (8.08-12.2)	11.6 (9.48-14.0)	14.2 (11.6-17.3)	16.4 (13.2-20.1)	19.4 (15.0-24.1)	21.7 (16.2-27.3)	24.0 (17.5-30.8)	26.5 (18.8-34.6)	30.0 (20.5-40.0)	32.8 (21.7-44.4)
60-day	11.5 (9.36-14.0)	13.4 (11.0-16.2)	16.5 (13.5-20.0)	19.1 (15.3-23.3)	22.6 (17.5-28.0)	25.3 (19.0-31.8)	28.0 (20.5-35.8)	30.9 (21.9-40.1)	34.9 (23.8-46.2)	37.9 (25.2-51.0)

[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

Rainfall Intensity-Duration-Frequency Coefficients for Texas

Texas Department of Transportation

Based on "National Oceanic and Atmospheric Administration's (NOAA) Atlas 14
Precipitation-Frequency Atlas of the United States, Volume 11 Version 2.0: Texas" (Perica et al. 2018)

Parameter Selection

1. Select Units
   English
2. Select Methodology
   Annual Maximum Series (AMS)
3. Select County
   TARRANT
4. Select County Zone
   Zone-1
5. Select Time of Concentration (t_C)
   10 Minute

	Design Annual Exceedance Probability (Design Annual Recurrence Interval)						
Coefficient	50% (2-year)	20% (5-year)	10% (10-year)	4% (25-year)	2% (50-year)	1% (100-year)	0.2% (500-year)
e	0.7842	0.7793	0.7759	0.7715	0.7678	0.7643	0.7583
b	44.1286	57.0870	66.7228	79.1811	88.1558	97.2910	121.1438
d (min)	10.0200	10.2377	10.3432	10.4421	10.4601	10.5378	11.1141
Intensity (inches/hour)	4.21	5.48	6.44	7.72	8.68	9.66	11.99

Note: Tarrant County has 1 rainfall zone.

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 Tarrant County = 79.18
- d = constant for Tarrant County = 10.44
- e = constant for Tarrant County = 0.770
- 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 sq. ft/acre = A in acres

Case 1:

C =	0.7	
t_c =	25.87	min
i =	4.98	in/hr
Length:	377.00	ft
A	0.0087	ac
Q =	0.030	cfs

Case 2:

C =	0.7	
t_c =	32.58	min
i =	4.37	in/hr
Length:	503.00	ft
A	0.0115	ac
Q =	0.035	cfs

Case 3:

C =	0.7	
t_c =	4.87	min
i =	9.69	in/hr
Length:	120.00	ft
A	0.0028	ac
Q =	0.019	cfs

Case 4:

C =	0.7	
t_c =	4.87	min
i =	9.69	in/hr
Length:	127.00	ft
A	0.0029	ac
Q =	0.019	cfs

FORT WORTH C&D LANDFILL
0771-356-11-35
SHEET FLOW VELOCITY

Approximate depth of flow:

Using Manning's Equation

$V = (1.49/n)\ y^{0.67}\ S^{0.5}$

$Q = VA \Rightarrow V = Q/A$

$A = y \times 1$ (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

$y = (Qn/1.49\ S^{0.5})^{1/1.67}$

$y = (Qn/1.49S^{0.5})^{0.6}$

Case 1:

Q =	0.030	cfs
n =	0.24	
S =	0.05	ft/ft
y =	0.101	ft

Case 2:

Q =	0.035	cfs
n =	0.24	
S =	0.05	ft/ft
y =	0.111	ft

Case 3:

Q =	0.019	cfs
n =	0.24	
S =	0.33	ft/ft
y =	0.043	ft

Case 4:

Q =	0.019	cfs
n =	0.24	
S =	0.33	ft/ft
y =	0.043	ft

Determine sheet flow velocity:

V = Q/A (assume unit flow width for the flow area, A)

Case 1:

Q =	0.030	cfs
A =	0.101	sf
V =	0.30	ft/s

Case 2:

Q =	0.035	cfs
A =	0.111	sf
V =	0.32	ft/s

Case 3:

Q =	0.019	cfs
A =	0.043	sf
V =	0.44	ft/s

Case 4:

Q =	0.019	cfs
A =	0.043	sf
V =	0.44	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.

Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges

Adopted by the
Texas Department of Transportation

November 1, 2014

Item 164
Seeding for Erosion Control

1. DESCRIPTION

Provide and install temporary or permanent seeding for erosion control as shown on the plans or as directed.

2. MATERIALS

2.1. **Seed**. Provide seed from the previous season's crop meeting the requirements of the Texas Seed Law, including the testing and labeling for pure live seed (PLS = Purity × Germination). Furnish seed of the designated species, in labeled unopened bags or containers to the Engineer before planting. Use within 12 mo. from the date of the analysis. When Buffalograss is specified, use seed that is treated with KNO_3 (potassium nitrate) to overcome dormancy.

Use Tables 1–4 to determine the appropriate seed mix and rates as specified on the plans. If a plant species is not available by the producers, the other plant species in the recommended seed mixture will be increased proportionally by the PLS/acre of the missing plant species.

Table 1
Permanent Rural Seed Mix

District and Planting Dates	Clay Soils Species and Rates (lb. PLS/acre)		Sandy Soils Species and Rates (lb. PLS/acre)	
1 (Paris) Feb. 1–May 15	Green Sprangletop Sideoats Grama (Haskell) Bermudagrass Little Bluestem (Native) Illinois Bundleflower	0.3 3.2 1.8 1.7 1.0	Green Sprangletop Bermudagrass Bahiagrass (Pensacola) Sand Lovegrass Weeping Lovegrass (Ermelo) Partridge Pea	0.3 1.5 6.0 0.6 0.8 1.0
2 (Ft. Worth) Feb. 1–May 15	Green Sprangletop (Van Horn) Sideoats Grama (Haskell) Texas Grama (Atascosa) Hairy Grama (Chaparral) Shortspike Windmillgrass (Welder) Little Bluestem (OK Select) Purple Prairie Clover (Cuero) Engelmann Daisy (Eldorado) Illinois Bundleflower Awnless Bushsunflower (Plateau)	1.0 1.0 1.0 0.4 0.2 0.8 0.6 0.75 1.3 0.2	Green Sprangletop (Van Horn) Hooded Windmillgrass (Mariah) Shortspike Windmillgrass (Welder) Hairy Grama (Chaparral) Slender Grama (Dilley) Sand Lovegrass (Mason) Sand Dropseed (Borden County) Partridge Pea (Comanche) Little Bluestem (OK Select) Englemann Daisy (Eldorado) Purple Prairie Clover	1.0 0.2 0.2 0.4 1.0 0.2 0.2 0.6 0.8 0.75 0.3
3 (Wichita Falls) Feb. 1–May 15	Green Sprangletop (Van Horn) Sideoats Grama (Haskell) Texas Grama (Atascosa) Hairy Grama (Chaparral) Shortspike Windmillgrass (Welder) Little Bluestem (OK Select) Blue Grama (Hachita) Western Wheatgrass (Barton) Galleta Grass (Viva) Engelmann Daisy (Eldorado) Awnless Bushsunflower (Plateau)	0.6 1.0 1.0 0.4 0.2 0.8 0.4 1.2 0.6 0.75 0.2	Green Sprangletop (Van Horn) Hooded Windmillgrass (Mariah) Shortspike Windmillgrass (Welder) Hairy Grama (Chaparral) Sand Lovegrass (Mason) Sand Dropseed (Borden County) Partridge Pea (Comanche) Little Bluestem (OK Select) Englemann Daisy (Eldorado) Purple Prairie Clover (Cuero)	1.0 0.2 0.2 0.4 0.2 0.2 0.6 0.8 0.75 0.3
4 (Amarillo) Feb. 15–May 15	Green Sprangletop Sideoats Grama (Haskell) Blue Grama (Hachita) Buffalograss (Texoka) Illinois Bundleflower	0.3 3.6 1.2 1.6 1.0	Green Sprangletop Weeping Lovegrass (Ermelo) Blue Grama (Hachita) Sand Dropseed (Borden Co.) Sand Bluestem Purple Prairie Clover	0.3 0.8 1.0 0.3 1.8 0.5

Table 1 (continued)
Permanent Rural Seed Mix

District and Planting Dates	Clay Soils Species and Rates (lb. PLS/acre)		Sandy Soils Species and Rates (lb. PLS/acre)	
5 (Lubbock)	Green Sprangletop	0.3	Green Sprangletop	0.3
Feb. 15–May 15	Sideoats Grama (El Reno)	3.6	Weeping Lovegrass (Ermelo)	0.8
	Blue Grama (Hachita)	1.2	Blue Grama (Hachita)	1.0
	Buffalograss (Texoka)	1.6	Sand Dropseed (Borden Co.)	0.3
	Illinois Bundleflower	1.0	Sand Bluestem	1.8
			Purple Prairie Clover	0.5
6 (Odessa)	Green Sprangletop (Van Horn)	1.0	Green Sprangletop (Van Horn)	1.0
Feb. 1–May 15	Sideoats Grama (South Texas)	1.0	Hooded Windmillgrass (Mariah)	0.2
	Blue Grama (Hachita)	0.4	Blue Grama (Hachita)	0.4
	Galleta Grass (Viva)	0.6	Hairy Grama (Chaparral)	0.4
	Shortspike Windmillgrass (Welder)	0.2	Sand Lovegrass (Mason)	0.2
	Pink Pappusgrass (Maverick)	0.6	Sand Dropseed (Borden County)	0.2
	Alkali Sacaton (Saltalk)	0.2	Indian Ricegrass (Rim Rock)	1.6
	Plains Bristlegrass (Catarina Blend)	0.2	Sand Bluestem (Cottle County)	1.2
	False Rhodes Grass (Kinney)	0.1	Little Bluestem (Pastura)	0.8
	Whiplash Pappusgrass (Webb)	0.6	Purple Prairie Clover (Cuero)	0.3
	Arizona Cottontop (La Salle)	0.2		
7 (San Angelo)	Green Sprangletop (Van Horn)	1.0	Green Sprangletop (Van Horn)	1.0
Feb. 1–May 1	Sideoats Grama (Haskell)	1.0	Hooded Windmillgrass (Mariah)	0.2
	Texas Grama (Atascosa)	1.0	Shortspike Windmillgrass (Welder)	0.2
	Hairy Grama (Chaparral)	0.4	Hairy Grama (Chaparral)	0.4
	Shortspike Windmillgrass (Welder)	0.2	Sand Lovegrass (Mason)	0.2
	Little Bluestem (OK Select)	0.4	Sand Dropseed (Borden County)	0.2
	Blue Grama (Hachita)	0.4	Sand Bluestem (Cottle County)	1.2
	Western Wheatgrass (Barton)	1.2	Partridge Pea (Comanche)	0.6
	Galleta Grass (Viva)	0.6	Little Bluestem (OK Select)	0.8
	Engelmann Daisy (Eldorado)	0.75	Englemann Daisy (Eldorado)	0.75
	Illinois Bundleflower (Sabine)	1.0	Purple Prairie Clover (Cuero)	0.3
8 (Abilene)	Green Sprangletop (Van Horn)	1.0	Green Sprangletop (Van Horn)	1.0
Feb. 1–May 15	Sideoats Grama (Haskell)	1.0	Hooded Windmillgrass (Mariah)	0.2
	Texas Grama (Atascosa)	1.0	Shortspike Windmillgrass (Welder)	0.2
	Hairy Grama (Chaparral)	0.4	Hairy Grama (Chaparral)	0.4
	Shortspike Windmillgrass (Welder)	0.2	Sand Lovegrass (Mason)	0.2
	Little Bluestem (OK Select)	0.4	Sand Dropseed (Borden County)	0.2
	Blue Grama (Hachita)	0.4	Sand Bluestem (Cottle County)	1.2
	Western Wheatgrass (Barton)	1.2	Partridge Pea (Comanche)	0.6
	Galleta Grass (Viva)	0.6	Little Bluestem (OK Select)	0.8
	Engelmann Daisy (Eldorado)	0.75	Englemann Daisy (Eldorado)	0.75
	Illinois Bundleflower (Sabine)	1.0	Purple Prairie Clover (Cuero)	0.3
9 (Waco)	Green Sprangletop (Van Horn)	1.0	Green Sprangletop (Van Horn)	1.0
Feb. 1–May 15	Sideoats Grama (Haskell)	1.0	Hooded Windmillgrass (Mariah)	0.2
	Texas Grama (Atascosa)	1.0	Shortspike Windmillgrass (Welder)	0.2
	Hairy Grama (Chaparral)	0.4	Hairy Grama (Chaparral)	0.4
	Shortspike Windmillgrass (Welder)	0.2	Slender Grama (Dilley)	1.0
	Little Bluestem (OK Select)	0.8	Sand Lovegrass (Mason)	0.2
	Purple Prairie Clover (Cuero)	0.6	Sand Dropseed (Borden County)	0.2
	Engelmann Daisy (Eldorado)	0.75	Partridge Pea (Comanche)	0.6
	Illinois Bundleflower	1.3	Little Bluestem (OK Select)	0.8
	Awnless Bushsunflower (Plateau)	0.2	Englemann Daisy (Eldorado)	0.75
			Purple Prairie Clover	0.3
10 (Tyler)	Green Sprangletop	0.3	Green Sprangletop	0.3
Feb. 1–May 15	Bermudagrass	1.8	Bermudagrass	1.8
	Bahiagrass (Pensacola)	9.0	Bahiagrass (Pensacola)	9.0
	Sideoats Grama (Haskell)	2.7	Weeping Lovegrass (Ermelo)	0.5
	Illinois Bundleflower	1.0	Sand Lovegrass	0.5
			Lance-Leaf Coreopsis	1.0
11 (Lufkin)	Green Sprangletop	0.3	Green Sprangletop	0.3
Feb. 1–May 15	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

Table 1 (continued)
Permanent Rural Seed Mix

District and Planting Dates	Clay Soils Species and Rates (lb. PLS/acre)		Sandy Soils Species and Rates (lb. PLS/acre)	
24 (El Paso) Feb. 1–May 15	Green Sprangletop (Van Horn)	1.0	Green Sprangletop (Van Horn)	1.0
	Sideoats Grama (South Texas)	1.0	Hooded Windmillgrass (Mariah)	0.2
	Blue Grama (Hachita)	0.4	Blue Grama (Hachita)	0.4
	Galleta Grass (Viva)	0.6	Hairy Grama (Chaparral)	0.4
	Shortspike Windmillgrass (Welder)	0.2	Sand Lovegrass (Mason)	0.2
	Pink Pappusgrass (Maverick)	0.6	Sand Dropseed (Borden County)	0.2
	Alkali Sacaton (Saltalk)	0.2	Indian Ricegrass (Rim Rock)	1.6
	Plains Bristlegrass (Catarina Blend)	0.2	Sand Bluestem (Cottle County)	1.2
	False Rhodes Grass (Kinney)	0.1	Little Bluestem (Pastura)	0.8
	Whiplash Pappusgrass (Webb)	0.6	Purple Prairie Clover (Cuero)	0.3
	Arizona Cottontop (La Salle)	0.2		
25 (Childress) Feb. 1–May 15	Green Sprangletop	0.3	Green Sprangletop	0.3
	Sideoats Grama (El Reno)	2.7	Weeping Lovegrass (Ermelo)	1.2
	Blue Grama (Hachita)	0.9	Sand Dropseed (Borden Co.)	0.5
	Western Wheatgrass	2.1	Sand Lovegrass	0.8
	Galleta	1.6	Purple Prairie Clover	0.5
	Illinois Bundleflower	1.0		

Table 2
Permanent Urban Seed Mix

District and Planting Dates	Clay Soils Species and Rates (lb. PLS/acre)		Sandy Soils Species and Rates (lb. PLS/acre)	
1 (Paris) Feb. 1–May 15	Green Sprangletop	0.3	Green Sprangletop	0.3
	Bermudagrass	2.4	Bermudagrass	5.4
	Sideoats Grama (Haskell)	4.5		
2 (Ft. Worth) Feb. 1–May 15	Green Sprangletop	0.3	Green Sprangletop	0.3
	Sideoats Grama (El Reno)	3.6	Sideoats Grama (El Reno)	3.6
	Bermudagrass	2.4	Bermudagrass	2.1
	Buffalograss (Texoka)	1.6	Sand Dropseed (Borden Co.)	0.3
3 (Wichita Falls) Feb. 1–May 15	Green Sprangletop	0.3	Green Sprangletop	0.3
	Sideoats Grama (El Reno)	4.5	Sideoats Grama (El Reno)	3.6
	Bermudagrass	1.8	Bermudagrass	1.8
	Buffalograss (Texoka)	1.6	Sand Dropseed (Borden Co.)	0.4
4 (Amarillo) Feb. 15–May 15	Green Sprangletop	0.3	Green Sprangletop	0.3
	Sideoats Grama (El Reno)	3.6	Sideoats Grama (El Reno)	2.7
	Blue Grama (Hachita)	1.2	Blue Grama (Hachita)	0.9
	Buffalograss (Texoka)	1.6	Sand Dropseed (Borden Co.)	0.4
			Buffalograss (Texoka)	1.6
5 (Lubbock) Feb. 15–May 15	Green Sprangletop	0.3	Green Sprangletop	0.3
	Sideoats Grama (El Reno)	3.6	Sideoats Grama (El Reno)	2.7
	Blue Grama (Hachita)	1.2	Blue Grama (Hachita)	0.9
	Buffalograss (Texoka)	1.6	Sand Dropseed (Borden Co.)	0.4
			Buffalograss (Texoka)	1.6
6 (Odessa) Feb. 1–May 15	Green Sprangletop	0.3	Green Sprangletop	0.3
	Sideoats Grama (Haskell)	3.6	Sideoats Grama (Haskell)	2.7
	Blue Grama (Hachita)	1.2	Sand Dropseed (Borden Co.)	0.4
	Buffalograss (Texoka)	1.6	Blue Grama (Hachita)	0.9
			Buffalograss (Texoka)	1.6
7 (San Angelo) Feb. 1–May 1	Green Sprangletop	0.3	Green Sprangletop	0.3
	Sideoats Grama (Haskell)	7.2	Sideoats Grama (Haskell)	3.2
	Buffalograss (Texoka)	1.6	Sand Dropseed (Borden Co.)	0.3
			Blue Grama (Hachita)	0.9
			Buffalograss (Texoka)	1.6
8 (Abilene) Feb. 1–May 15	Green Sprangletop	0.3	Green Sprangletop	0.3
	Sideoats Grama (Haskell)	3.6	Sand Dropseed (Borden Co.)	0.3
	Blue Grama (Hachita)	1.2	Sideoats Grama (Haskell)	3.6
	Buffalograss (Texoka)	1.6	Blue Grama (Hachita)	0.8
			Buffalograss (Texoka)	1.6

Table 2 (continued)
Permanent Urban Seed Mix

District and Planting Dates	Clay Soils Species and Rates (lb. PLS/acre)		Sandy Soils Species and Rates (lb. PLS/acre)	
9 (Waco)	Green Sprangletop	0.3	Green Sprangletop	0.3
Feb. 1–May 15	Bermudagrass	1.8	Buffalograss (Texoka)	1.6
	Buffalograss (Texoka)	1.6	Bermudagrass	3.6
	Sideoats Grama (Haskell)	4.5	Sand Dropseed (Borden Co.)	0.4
10 (Tyler)	Green Sprangletop	0.3	Green Sprangletop	0.3
Feb. 1–May 15	Bermudagrass	2.4	Bermudagrass	5.4
	Sideoats Grama (Haskell)	4.5		
11 (Lufkin)	Green Sprangletop	0.3	Green Sprangletop	0.3
Feb. 1–May 15	Bermudagrass	2.4	Bermudagrass	5.4
	Sideoats Grama (Haskell)	4.5		
12 (Houston)	Green Sprangletop	0.3	Green Sprangletop	0.3
Jan. 15–May 15	Sideoats Grama (Haskell)	4.5	Bermudagrass	5.4
	Bermudagrass	2.4		
13 (Yoakum)	Green Sprangletop	0.3	Green Sprangletop	0.3
Jan. 15–May 15	Sideoats Grama (South Texas)	4.5	Bermudagrass	5.4
	Bermudagrass	2.4		
14 (Austin)	Green Sprangletop	0.3	Green Sprangletop	0.3
Feb. 1–May 15	Bermudagrass	2.4	Bermudagrass	4.8
	Sideoats Grama (South Texas)	3.6	Buffalograss (Texoka)	1.6
	Buffalograss (Texoka)	1.6		
15 (San Antonio)	Green Sprangletop	0.3	Green Sprangletop	0.3
Feb. 1–May 1	Sideoats Grama (South Texas)	3.6	Bermudagrass	4.8
	Bermudagrass	2.4	Buffalograss (Texoka)	1.6
	Buffalograss (Texoka)	1.6		
16 (Corpus Christi)	Green Sprangletop	0.3	Green Sprangletop	0.3
Jan. 1–May 1	Sideoats Grama (South Texas)	3.6	Bermudagrass	4.8
	Bermudagrass	2.4	Buffalograss (Texoka)	1.6
	Buffalograss (Texoka)	1.6		
17 (Bryan)	Green Sprangletop	0.3	Green Sprangletop	0.3
Feb. 1–May 15	Bermudagrass	2.4	Bermudagrass	5.4
	Sideoats Grama (Haskell)	4.5		
18 (Dallas)	Green Sprangletop	0.3	Green Sprangletop	0.3
Feb. 1–May 15	Sideoats Grama (El Reno)	3.6	Buffalograss (Texoka)	1.6
	Buffalograss (Texoka)	1.6	Bermudagrass	3.6
	Bermudagrass	2.4	Sand Dropseed (Borden Co.)	0.4
19 (Atlanta)	Green Sprangletop	0.3	Green Sprangletop	0.3
Feb. 1–May 15	Bermudagrass	2.4	Bermudagrass	5.4
	Sideoats Grama (Haskell)	4.5		
20 (Beaumont)	Green Sprangletop	0.3	Green Sprangletop	0.3
Jan. 15–May 15	Bermudagrass	2.4	Bermudagrass	5.4
	Sideoats Grama (Haskell)	4.5		
21 (Pharr)	Green Sprangletop	0.3	Green Sprangletop	0.3
Jan. 15–May 15	Sideoats Grama (South Texas)	3.6	Buffalograss (Texoka)	1.6
	Buffalograss (Texoka)	1.6	Bermudagrass	3.6
	Bermudagrass	2.4	Sand Dropseed (Borden Co.)	0.4
22 (Laredo)	Green Sprangletop	0.3	Green Sprangletop	0.3
Jan. 15–May 1	Sideoats Grama (South Texas)	4.5	Buffalograss (Texoka)	1.6
	Buffalograss (Texoka)	1.6	Bermudagrass	3.6
	Bermudagrass	1.8	Sand Dropseed	0.4
23 (Brownwood)	Green Sprangletop	0.3	Green Sprangletop	0.3
Feb. 1–May 15	Sideoats Grama (Haskell)	3.6	Buffalograss (Texoka)	1.6
	Bermudagrass	1.2	Bermudagrass	3.6
	Blue Grama (Hachita)	0.9	Sand Dropseed (Borden Co.)	0.4
24 (El Paso)	Green Sprangletop	0.3	Green Sprangletop	0.3
Feb. 1–May 15	Sideoats Grama (South Texas)	3.6	Buffalograss (Texoka)	1.6
	Blue Grama (Hachita)	1.2	Sand Dropseed (Borden Co.)	0.4
	Buffalograss (Texoka)	1.6	Blue Grama (Hachita)	1.8
25 (Childress)	Green Sprangletop	0.3	Green Sprangletop	0.3
Feb. 1–May 15	Sideoats Grama (El Reno)	3.6	Sand Dropseed (Borden Co.)	0.4
	Blue Grama (Hachita)	1.2	Buffalograss (Texoka)	1.6
	Buffalograss (Texoka)	1.6	Bermudagrass	1.8

Table 3
Temporary Cool Season Seeding

Districts	Dates	Seed Mix and Rates (lb. PLS/acre)	
Paris (1), Amarillo (4), Lubbock (5), Dallas (18)	September 1–November 30	Tall Fescue Western Wheatgrass Wheat (Red, Winter)	4.5 5.6 34
Odessa (6), San Angelo (7), El Paso (24)	September 1–November 30	Western Wheatgrass Wheat (Red, Winter)	8.4 50
Waco (9), Tyler (10), Lufkin (11), Austin (14), San Antonio (15), Bryan (17), Atlanta (19)	September 1–November 30	Tall Fescue Oats Wheat	4.5 24 34
Houston (12), Yoakum (13), Corpus Christi (16), Beaumont (20), Pharr (21), Laredo (22)	September 1–November 30	Oats	72
Ft. Worth (2), Wichita Falls (3), Abilene (8), Brownwood (23), Childress (25)	September 1–November 30	Tall Fescue Western Wheatgrass Cereal Rye	4.5 5.6 34

Table 4
Temporary Warm Season Seeding

Districts	Dates	Seed Mix and Rates (lb. PLS/acre)	
All	May 1–August 31	Foxtail Millet	34

2.2. **Fertilizer**. Use fertilizer in conformance with Article 166.2., "Materials."

2.3. **Vegetative Watering**. Use water that is clean and free of industrial wastes and other substances harmful to the growth of vegetation.

2.4. **Mulch**.

2.4.1. **Straw or Hay Mulch**. Use straw or hay mulch in conformance with Section 162.2.5., "Mulch."

2.4.2. **Cellulose Fiber Mulch**. Use only cellulose fiber mulches that are on the Approved Products List, *Erosion Control Approved Products*. (http://www.txdot.gov/business/resources/erosion-control.html) Submit one full set of manufacturer's literature for the selected material. Keep mulch dry until applied. Do not use molded or rotted material.

2.5. **Tacking Methods**. Use a tacking agent applied in accordance with the manufacturer's recommendations or a crimping method on all straw or hay mulch operations. Use tacking agents as approved or as specified on the plans.

3. CONSTRUCTION

Cultivate the area to a depth of 4 in. before placing the seed unless otherwise directed. Use approved equipment to vertically track the seedbed as shown on the plans or as directed. Cultivate the seedbed to a depth of 4 in. or mow the area before placement of the permanent seed when performing permanent seeding after an established temporary seeding. Plant the seed specified and mulch, if required, after the area has been completed to lines and grades as shown on the plans.

3.1. **Broadcast Seeding**. Distribute the seed or seed mixture uniformly over the areas shown on the plans using hand or mechanical distribution or hydro-seeding on top of the soil unless otherwise directed. Apply the mixture to the area to be seeded within 30 min. of placement of components in the equipment when seed and water are to be distributed as a slurry during hydro-seeding. Roll the planted area with a light roller or other suitable equipment. Roll sloped areas along the contour of the slopes.

3.2. **Straw or Hay Mulch Seeding**. Plant seed according to Section 164.3.1., "Broadcast Seeding." Apply straw or hay mulch uniformly over the seeded area immediately after planting the seed or seed mixture. Apply straw mulch at 2 to 2.5 tons per acre. Apply hay mulch at 1.5 to 2 tons per acre. Use a tacking method over the mulched area.

3.3. **Cellulose Fiber Mulch Seeding**. Plant seed in accordance with Section 164.3.1., "Broadcast Seeding." Apply cellulose fiber mulch uniformly over the seeded area immediately after planting the seed or seed mixture at the following rates.

- Sandy soils with slopes of 3:1 or less—2,500 lb. per acre.
- Sandy soils with slopes greater than 3:1—3,000 lb. per acre.
- Clay soils with slopes of 3:1 or less—2,000 lb. per acre.
- Clay soils with slopes greater than 3:1—2,300 lb. per acre.

Cellulose fiber mulch rates are based on dry weight of mulch per acre. Mix cellulose fiber mulch and water to make a slurry and apply uniformly over the seeded area using suitable equipment.

3.4. **Drill Seeding**. Plant seed or seed mixture uniformly over the area shown on the plans at a depth of 1/4 to 1/3 in. using a pasture or rangeland type drill unless otherwise directed. Plant seed along the contour of the slopes.

3.5. **Straw or Hay Mulching**. Apply straw or hay mulch uniformly over the area as shown on the plans. Apply straw mulch at 2 to 2.5 tons per acre. Apply hay mulch at 1.5 to 2 tons per acre. Use a tacking method over the mulched area.

Apply fertilizer in conformance with Article 166.3., "Construction." Seed and fertilizer may be distributed simultaneously during "Broadcast Seeding" operations, provided each component is applied at the specified rate. Apply half of the required fertilizer during the temporary seeding operation and the other half during the permanent seeding operation when temporary and permanent seeding are both specified for the same area.

Water the seeded areas at the rates and frequencies as shown on the plans or as directed.

4. MEASUREMENT

This Item will be measured by the square yard or by the acre.

5. PAYMENT

The work performed and the materials furnished in accordance with this Item and measured as provided under "Measurement" will be paid for at the unit price bid for "Broadcast Seeding (Perm)" of the rural or urban seed mixture and sandy or clay soil specified, "Broadcast Seeding (Temp)" of warm or cool season specified, "Straw or Hay Mulch Seeding (Perm)" of the rural or urban seed mixture and sandy or clay soil specified, "Straw or Hay Mulch Seeding (Temp)" of warm or cool season specified, "Cellulose Fiber Mulch Seeding (Perm)" of the rural or urban seed mixture and sandy or clay soil specified, "Cellulose Fiber Mulch Seeding (Temp)" of warm or cool season specified, "Drill Seeding (Perm)" of the rural or urban seed mixture and sandy or clay soil specified, "Drill Seeding (Temp)" of warm or cool season specified, and "Straw or Hay Mulching." This price is full compensation for furnishing materials, including water for hydro-seeding and hydro-mulching operations, mowing, labor, equipment, tools, supplies, and incidentals. Fertilizer will not be paid for directly but will be subsidiary to this Item. Water for irrigating the seeded area, when specified, will be paid for under Item 168, "Vegetative Watering."

APPENDIX IIIF-E

PERMITTED LANDFILL CONDITION
HYDROLOGIC CALCULATIONS

Includes pages IIIF-E-1 through IIIF-E-117

STATE OF TEXAS
CHARLES R. MARSH
105073
LICENSED
PROFESSIONAL ENGINEER
02/09/2023

CONTENTS

STATE OF TEXAS
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PROFESSIONAL ENGINEER
02/09/2023

HYPOTHETICAL STORM DATA

Hypothetical Storm Data

Precipitation data taken from NOAA Atlas 14 rainfall data.

Time	5 min	15 min	60 min	2 hr	3 hr	6 hr	12 hr	24 hr
25-Year Event	0.82	1.63	2.96	3.77	4.28	5.19	6.15	7.17

NOAA Atlas 14 - Precipitation-Frequency Atlas of the United States, Volume 11, Version 2.0: Texas (*U.S. Department of Commerce, National Oceanic and Atmospheric Administration, and National Weather Service, 2018*) was used to identify precipitation values for storm durations ranging from 5 minutes to 24 hours.

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. U.S. Army Corps of Engineers, Hydrologic Engineering Center, *HEC-HMS Hydrologic Modeling System 4.9,* January 2022.
2. United States Department of Agriculture, National Resource Conservation Service, Web Soil Survey for Johnson County, Texas (http://websoilsurvey.nrcs.usda.gov).
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-E-5).

Solution: Based on the soil survey information found in Ref. 2, hydrologic group B,C, and D soils predominate the soils within the permit boundary drainage area (see pages IIIF-E-5 through IIIF-E-8). Hydrologic group D was selected to represent the onsite soils.

The curve number for the offsite drainage areas around the site, large non-landfill drainage basins within the permit boundary, and drainage channels (O1, O2, O3, O4, S1, S2, CH1, CH2**,** CH3, CH4, and CH5) was calculated using the table on Page IIIF-E-11, assuming pasture land in fair conditions. The majority of the area is undeveloped and assume compare to the off-site and on-site subasins near the site.

Use: CN = 84

The final cover system was assumed to be in place and the erosion layer will control precipitation loss. A curve number that is corrected for the surface slope of the erosion layer may be computed first using the chart on page IIIF-E-11 to select an un-adjusted curve numb Calculate the adjusted curve number using equation 34 from Ref. 3 (see page IIIF-E-10).

$$CN_{II} = 100 - (100 - CN_{II\,o}) * (L^{*\,2} / S^{*}) \wedge (CN_{II\,o}^{-0.81})$$

Use: $CN_{II\,o} = 84$, $L^* = (500/500)$, $S^* = (.05/.04)$ for top dome surfaces
Use: $CN_{II\,o} = 84$, $L^* = (120/500)$, $S^* = (.33/.04)$ for side slopes

Calculate: CN = 84 for top dome surfaces
Calculate: CN = 86 for side slopes

- Use curve number calculated for side slopes for the entire final cover area, inculding top dome areas, conservatively.

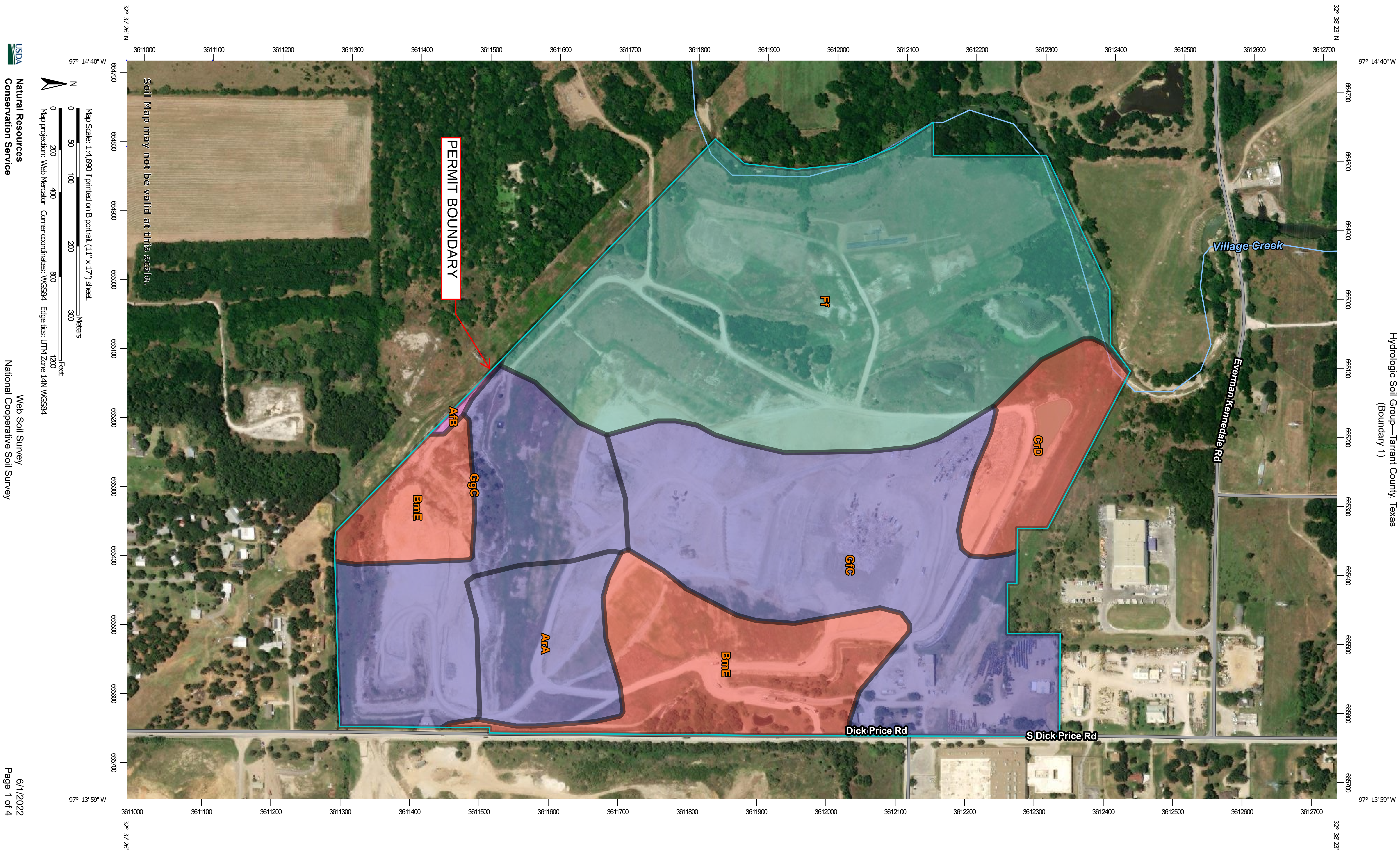
The pond areas are assumed to collect all precipitation for their areas:

Use: CN = 99

The initial abstraction is:

Use: I = 0.0"

- All drainage areas were modeled to assume no inital abstractions.



Hydrologic Soil Group—Tarrant County, Texas
(Boundary 1)
Soil Map may not be valid at this scale.
PERMIT BOUNDARY
Ff
GfC
GgC
BmE
ArA
AfB
GrD
Village Creek
Everman Kennedale Rd
Dick Price Rd
S Dick Price Rd
Map Scale: 1:4,890 if printed on B portrait (11" x 17") sheet.
Meters
0 50 100 200 300
Feet
0 200 400 800 1200
Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 14N WGS84
USDA
Natural Resources
Conservation Service
Web Soil Survey
National Cooperative Soil Survey
6/1/2022
Page 1 of 4
97° 14' 40" W
97° 13' 59" W
32° 37' 26" N
32° 38' 23" N

MAP LEGEND

Area of Interest (AOI)

Area of Interest (AOI)

Soils

Soil Rating Polygons

- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Soil Rating Lines

- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Soil Rating Points

- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Water Features

- Streams and Canals

Transportation

- Rails
- Interstate Highways
- US Routes
- Major Roads
- Local Roads

Background

- Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:20,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Tarrant County, Texas
Survey Area Data: Version 19, Sep 10, 2021

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Jan 18, 2020—Nov 15, 2020

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Hydrologic Soil Group

Map unit symbol	Map unit name	Rating	Acres in AOI	Percent of AOI
AfB	Arents, frequently flooded	A	0.3	0.2%
ArA	Arents, loamy	B	11.2	6.1%
BmE	Birome-Aubrey-Rayex complex, 5 to 15 percent slopes	D	26.2	14.2%
CrD	Crosstell fine sandy loam, 3 to 8 percent slopes	D	10.3	5.6%
Ff	Frio clay loam, 0 to 1 percent slopes, frequently flooded	C	69.7	37.8%
GfC	Gasil fine sandy loam, 3 to 8 percent slopes	B	42.3	22.9%
GgC	Gasil sandy clay loam, graded, 1 to 5 percent slopes	B	24.3	13.2%
Totals for Area of Interest			**184.4**	**100.0%**

Description

Hydrologic soil groups are based on estimates of runoff potential. Soils are assigned to one of four groups 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 rate of water transmission.

Group B. Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately 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 a 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 that 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 and the second is for undrained areas. Only the soils that in their natural condition are in group D are assigned to dual classes.

Rating Options

Aggregation Method: Dominant Condition

Component Percent Cutoff: None Specified

Tie-break Rule: Higher

where

CN_{II_o} = AMC-II curve number for mild slope (unadjusted for slope)

C_0 = regression constant for a given level of vegetation

C_1 = regression constant for a given level of vegetation

C_2 = regression constant for a given level of vegetation

IR = infiltration correlation parameter for given soil type

The relationship between CN_{II_o}, the vegetative cover and default soil texture is shown graphically in Figure 8. Table 7 gives values of C_0, C_1 *and* C_2 for the five types of vegetative cover built into the HELP program.

4.2.3 Adjustment of Curve Number for Surface Slope

A regression equation was developed to adjust the AMC-II curve number for surface slope conditions. The regression was developed based on kinematic wave theory where

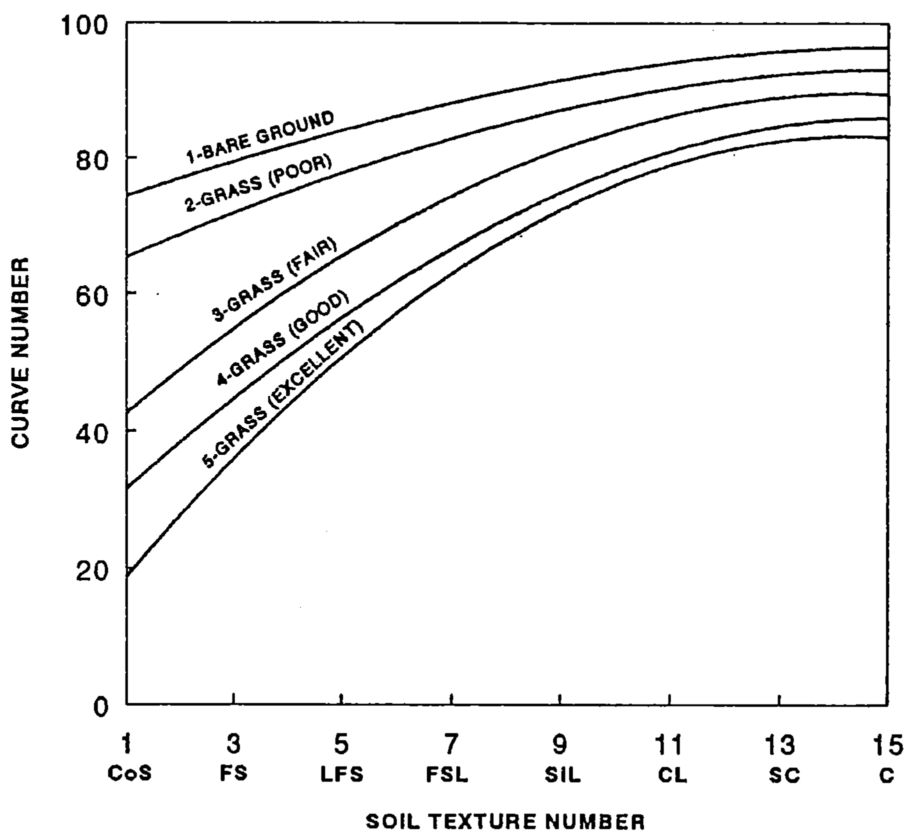


Figure 8. Relation between SCS Curve Number and Default Soil Texture Number for Various Levels of Vegetation

loam, and clayey loam as specified by saturated hydraulic conductivity, capillary drive, porosity, and maximum relative saturation, Two levels of vegetation were described--a good stand of grass (bluegrass sod) and a poor stand of grass (clipped range). Slopes of 0.04,0.10,0.20,0.35, and 0.50 ft/ft and slope lengths of 50, 100, 250, and 500 ft were used. Rainfalls of 1.1 inches, l-hour duration and 2nd quartile Huff distribution and of 3.8 inches, 6-hour duration and balanced distribution were modeled.

The resulting regression equation used for adjusting the AMC-II curve number computed for default soils and vegetation placed at mild slopes, CN_{II_o}, is:

$$CN_{II} = 100 - (100 - CN_{II_o}) \cdot \left(\frac{L^{*2}}{S^*} \right)^{CN_{II_o}^{-0.81}} \tag{34}$$

where

L^* = standardized dimensionless length, (L/500 ft)

S^* = standardized dimensionless slope, (S/0.04)

This same equation is used to adjust user-specified AMC-II curve numbers for surface slope conditions by substituting the user value for CN_{II_o} in Equation 34.

4.2.4 Adjustment of Curve Number for Frozen Soil

When the HELP program predicts frozen conditions to exist, the value of CN_{II} is increased, resulting in a higher calculated runoff. Knisel et al. (1985) found that this type of curve number adjustment in the CREAMS model resulted in improved predictions of annual runoff for several test watersheds. If the CN_{II} for unfrozen soil is less than or equal to 80, the CN_{II} for frozen soil conditions is set at 95. When the unfrozen soil CN_{II} is greater than 80, the CN_{II} is reset to be 98 on days when the program has determined the soil to be frozen. This adjustment results in an increase in CN_I and consequently a decrease in S_{mx} and S' (Equations 19, 26, and 30).

From Equations 19 and 21, it is apparent that as S' approaches zero, Q approaches P. In other words, as S' decreases, the calculated runoff becomes closer to being equal to the net rainfall which is most often, when frozen soil conditions exist, predominantly snowmelt. This will result in a decrease in infiltration under frozen soil conditions, which has been observed in numerous studies.

4.2.5 Summary of Daily Runoff Computation

The HELP model determines daily runoff by the following procedure:

TABLE 5.3 Values of SCS Curve Number for Rural Areas

Source: [McCuen, 1982]

Land Use Description	Hydrologic Soil Group 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

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HYDROGRAPH DEVELOPMENT INFORMATION

HYDROGRAPH DEVELOPMENT INFORMATION

Landfill Areas

Direct runoff methods, (i.e., kinematic wave) have been used for the majority of the landfill final cover areas. The kinematic wave method has been used to model the 5 percent topslope areas and 33 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 0.05 to 0.33 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-E-23, the swale spacing on 3H: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 0.0050 to 0.0953 (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 2.5:1 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. Espey "10-Minute" method has been used to estimate Snyder parameters. Snyder parameter estimations are provided on pages IIIF-E-18 through IIIF-E-23.

As discussed in Section 2 of Appendix IIIF, hydrographs for the areas outside of the permit boundary (O1, O2, O3, and O4), and larger areas inside the permit boundary (S1, S2, and S3) were developed using the Snyder unit hydrograph method. The percent imperviousness ranges from 2 percent to 25 percent, for the majority of the non-landfill on-site and off-site areas, which represents the majority of the watershed as undeveloped. Pond areas are assumed to be 100 percent impervious, and areas with significant channel surface or paved surfaces were assigned higher percentages of impervious area, as shown on IIIF-E-19.

Drainage Areas

The drainage areas used for this analysis are shown on Sheets IIIF-E-25 and IIIF-E-26. The routing scheme for the post-development condition is shown in the HEC-HMS output file presented on pages IIIF-E-28 through IIIF-E-65.

DISTRIBUTED RUNOFF METHOD
KINEMATIC WAVE EXAMPLE

Drainage area "DA2" is used in this example (refer to Sheet IIIF-E-17 for location of drainage area).

Watershed Specific Parameters:

A =	11.70	acres	Watershed Area (acres)
A =	0.0183	sq-miles	Watershed Area (sq-miles)
CN=	86		SCS Curve Number (see sheet IIIF-E-4 for more information)

Kinematic Wave parameter for overland flow:

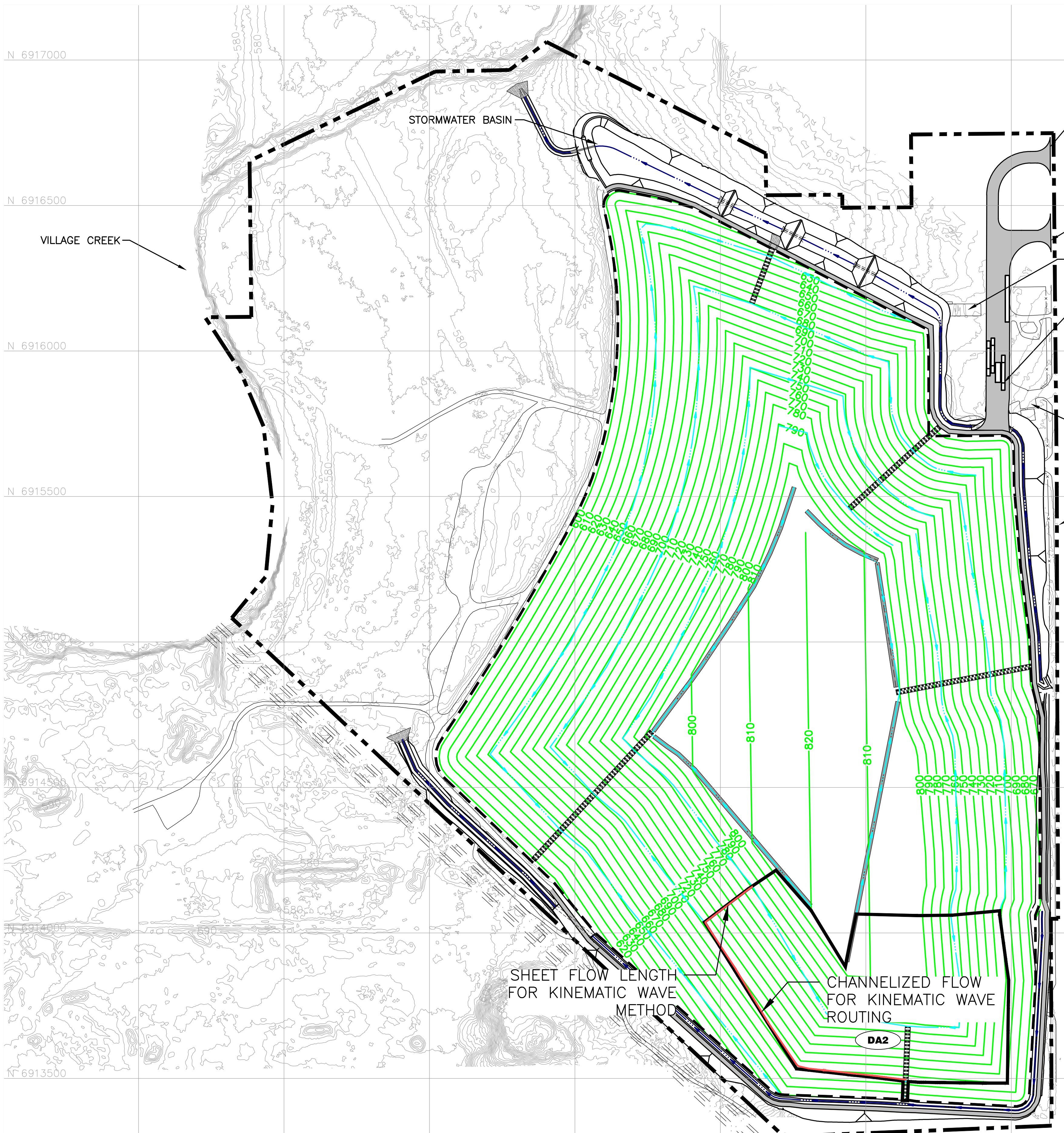
L=	205	ft	Typical overland flow (ft)
S=	0.33	ft/ft	Landfill slope (ft/ft)
N=	0.30		Manning's Coefficient

Percentage of the drainage area represented by this element is 100 percent

Kinematic Wave routing data for the swale:

L=	965	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.



N
0 200 400
SCALE IN FEET

LEGEND

- PERMIT BOUNDARY
- PERMITTED LIMIT OF WASTE
- 700 — PERMITTED FINAL COVER CONTOUR (SEE NOTE 2)
- N 6915000 — STATE PLANE COORDINATE
- 580 — EXISTING CONTOUR
- DRAINAGE CHUTE
- PERMITTED DRAINAGE SWALE
- PERMITTED PERIMETER CHANNEL
- DRAINAGE AREA BOUNDARY
- DA2 — DRAINAGE AREA DESIGNATION
- TOP DECK DRAINAGE TERRACE

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS PROVIDED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN 02-17-2022.
2. PERMITTED FINAL COVER GRADES ARE DEVELOPED FROM THE LANDFILL COMPLETION PLAN BY GEOSYNTEC CONSULTANTS, INC., DATED DECEMBER 2020.

STATE OF TEXAS
CHARLES R. MARSH
105073
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PROFESSIONAL ENGINEER
02/09/2023

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DATE: 12/2022
FILE: 0771-356-11
CAD: FIG IIIF-E-17 KINEMATIC WAVE.DWG

DRAWN BY: RAA
DESIGN BY: JBM
REVIEWED BY: CRM

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

PREPARED FOR
TEXAS REGIONAL LANDFILL COMPANY, LP

REVISIONS

NO.	DATE	DESCRIPTION

KINEMATIC WAVE PARAMETERS

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

WWW.WCGRP.COM

DRAWING IIIF-E-17

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O:\0771\356\EXPANSION 2022\PART III\IIIF\IIIF-E\IIIF-E-17 KINEMATIC WAVE PARAMETERS.dwg, jwilson, 1:2

ESPEY 10-MINUTE METHOD PARAMETERS

Snyder's Hydrograph Coefficients (Espey's 10 Minute Method)

Permitted Condition

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]
O1	5.11	935	0.0310	15	0.04	0.82	16.0	13.5	0.23	0.0080	1970.2	0.69
O2	4.56	590	0.0322	15	0.04	0.82	14.3	11.8	0.20	0.0071	2239.0	0.69
O3	17.39	680	0.0838	20	0.04	0.80	10.6	8.1	0.14	0.0272	2916.5	0.62
O4	6.25	530	0.0642	20	0.04	0.80	10.7	8.2	0.14	0.0098	3007.5	0.64
S1	11.05	1,225	0.0343	25	0.04	0.79	14.3	11.8	0.20	0.0173	2156.9	0.66
S2	63.06	2,440	0.0082	2	0.04	0.87	43.9	41.4	0.69	0.0985	605.4	0.65
S3	1.44	250	0.0560	2	0.04	0.87	16.1	13.6	0.23	0.0023	2062.7	0.73

[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

Snyder Unit Hydrograph uses lag time (T_{lag}) and peaking coefficient accounting for flood wave and watershed storage conditions.

Drainage area "S1" in the existing permitted condition is used in this example.

Estimated Watershed specific parameters

A =	11.05	acres	watershed area
L =	1225	feet	maximun flow length with this watershed
S =	0.0343	feet/feet	watershed slope
I =	25	percent (%)	watershed imperviousness
n =	0.04		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

Φ = for 25 percent impervious cover and n = 0.04

$\Phi = 0.79$

$T_r = 3.1(1225)(0.0343^{-0.25})(25^{-0.18})(0.79)$

$T_r = 14.3$ min

Calculate T_{lag}: watershed lag time

$T_{lag} = Tr - (\Delta t/2)$ — Δt is calculation interval, and 5 minutes is used in the HEC-HMS modeling in this project

$T_{lag} = 11.8$ minutes

$T_{lag} = 0.20$ hours

$A = A/640$

$A = 0.0173$ square miles

Calculate q_p: peak discharge of unit hydrograph per unit area (cfs/sq. mi).

$q_p = 31600(A^{-0.04})(T_r^{-1.07})$

$q_p = 31600(0.0173^{-0.04})(11.8^{-1.07})$

$q_p = 2156.9$ cfs/sq. mi

Calculate Peaking coefficient C_p:

$C_p = 49.375(A^{-0.04})(T_r^{-1.07})(T_{lag})$

$C_p = 49.375(0.0173^{-0.04})(14.3^{-1.07})(0.20)$

$C_p = 0.66$

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.

$$C_p = \frac{q_p \times T_L}{640} \qquad 6.30$$

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:

$$T_r = 3.10\, L^{0.23} S^{-0.25} I^{-0.18} \Phi^{1.57} \qquad 6.31$$

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.

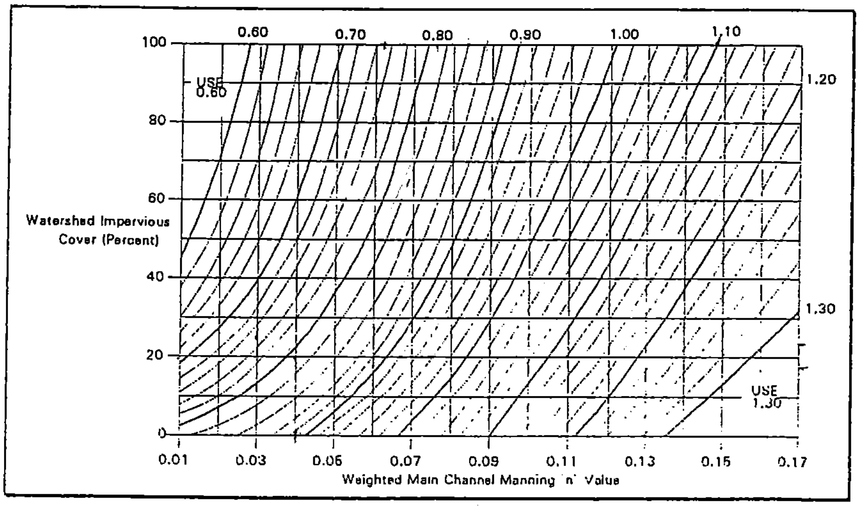

FIGURE 6.12 Determination of Conveyance Efficiency Coefficient Φ

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

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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.

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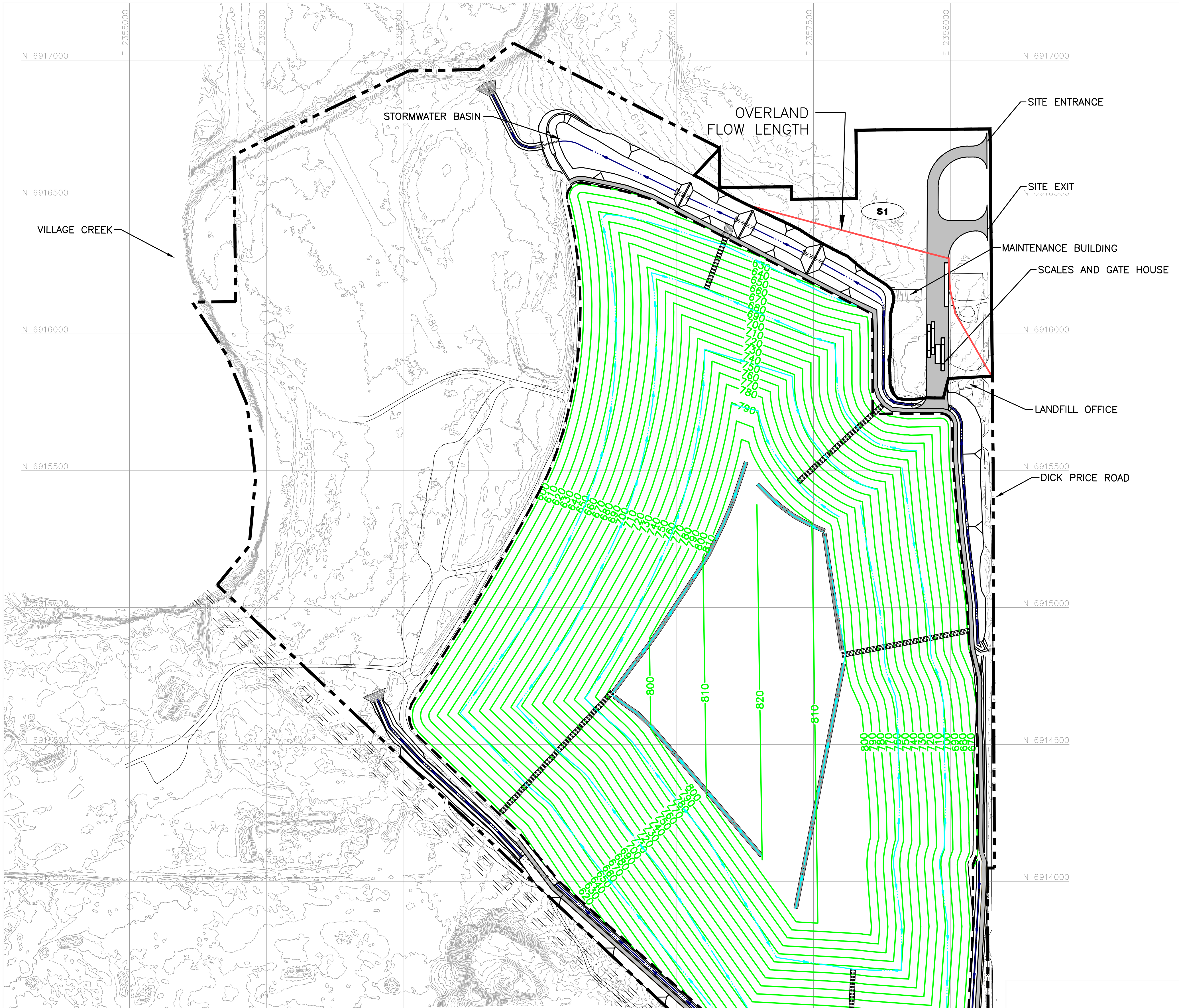

0 200 400
SCALE IN FEET

LEGEND

	PERMIT BOUNDARY
	PERMITTED LIMIT OF WASTE
700	PERMITTED FINAL COVER CONTOUR
N 6915000	STATE PLANE COORDINATE
580	EXISTING CONTOUR
	DRAINAGE CHUTE
	PERMITTED DRAINAGE SWALE
	PERMITTED PERIMETER CHANNEL
	DRAINAGE AREA BOUNDARY
S1	DRAINAGE AREA DESIGNATION
	TOP DECK DRAINAGE TERRACE

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS PROVIDED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN 02–17–2022.
2. PERMITTED FINAL COVER GRADES ARE DEVELOPED FROM THE LANDFILL COMPLETION PLAN BY GEOSYNTEC CONSULTANTS, INC., DATED DECEMBER 2020.

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02/09/2023

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DATE: 12/2022
FILE: 0771–356–11
CAD: FIG IIIF–E–23 EPSEY METHOD.DWG

DRAWN BY: JDW
DESIGN BY: JBM
REVIEWED BY: CRM

Weaver Consultants Group
TBPE REGISTRATION NO. F–3727

PREPARED FOR
TEXAS REGIONAL LANDFILL COMPANY, LP

REVISIONS

NO.	DATE	DESCRIPTION

EPSEY "10–MINUTE" METHOD PARAMETERS

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

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DRAWING IIIF–E–23

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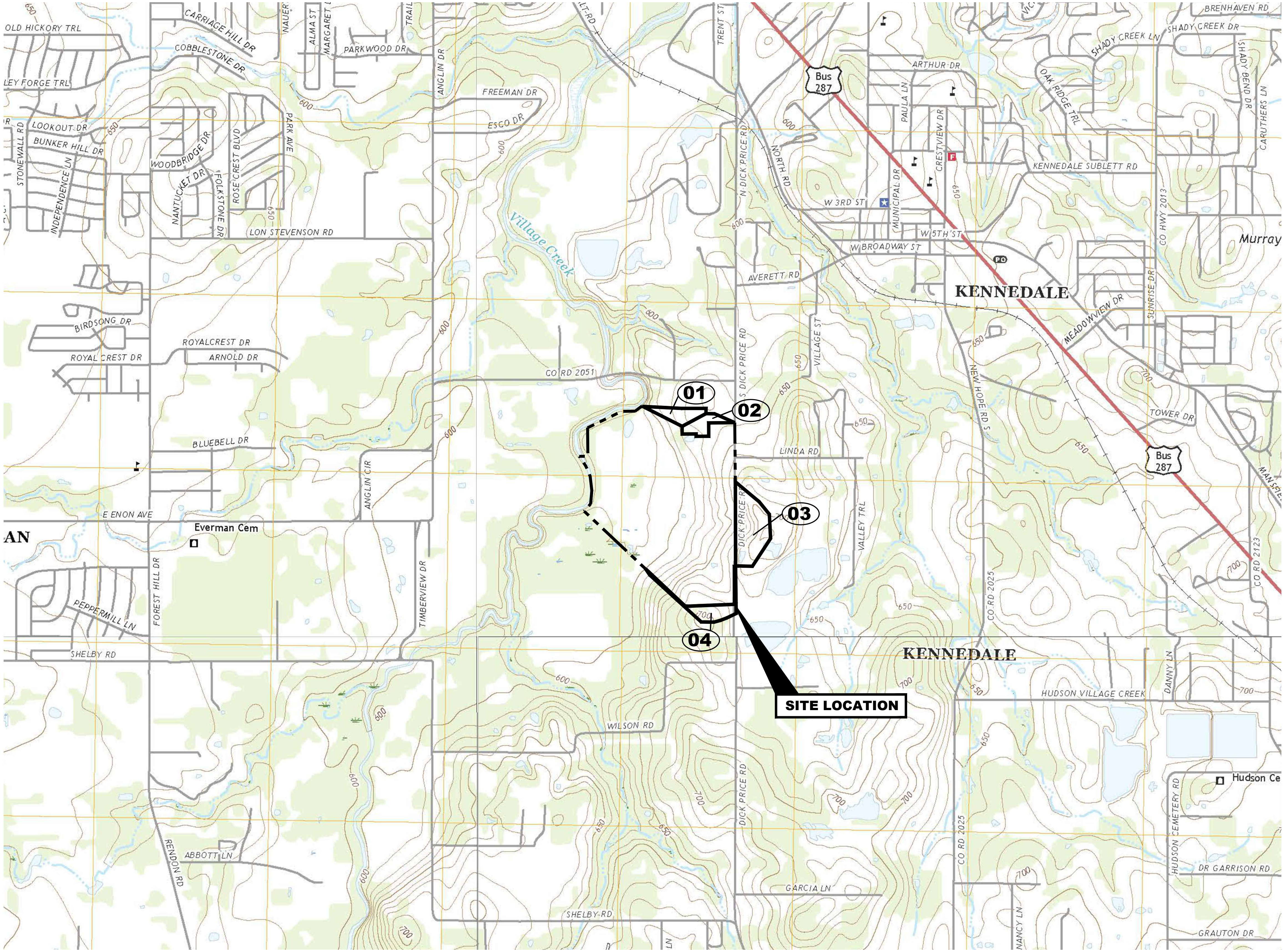
PERMITTED LANDFILL HEC-1 ANALYSIS
DRAINAGE AREAS



N

0 1000 2000
SCALE IN FEET

LEGEND

PERMIT BOUNDARY
DRAINAGE AREA BOUNDARY
02 DRAINAGE AREA LABEL

ROAD CLASSIFICATION

Expressway
Secondary Hwy
Ramp
Local Connector
Local Road
4WD
Interstate Route
US Route
State Route

DRAINAGE AREA NO.	AREA (ACRES)
01	5.11
02	4.56
03	17.39
04	6.25
TOTAL	33.32

NOTES:

1. ADAPTED FROM USGS 7.5 MINUTE QUADRANGLE TOPOGRAPHIC MAP (FORT WORTH, KENNEDALE, BURLESON AND MANSFIELD, TEXAS, 2019).
2. DRAINAGE AREA DELINEATION WITHIN THE PERMIT BOUNDARY IS INCLUDED ON DRAWING IIIF–E–26.

FORT WORTH, TX 2019
KENNEDALE, TX 2019
BURLESON, TX 2019
MANSFIELD, TX 2019

STATE OF TEXAS
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02/09/2023

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FILE: 0771–356–11
CAD: IIIF–E–25–OFFSITE DRAINAGE.DWG

DRAWN BY: RAA
DESIGN BY: BPY
REVIEWED BY: CRM

PREPARED FOR
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REVISIONS

NO.	DATE	DESCRIPTION

Weaver Consultants Group
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MAJOR PERMIT AMENDMENT
POST–DEVELOPMENT OFFSITE DRAINAGE AREAS

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

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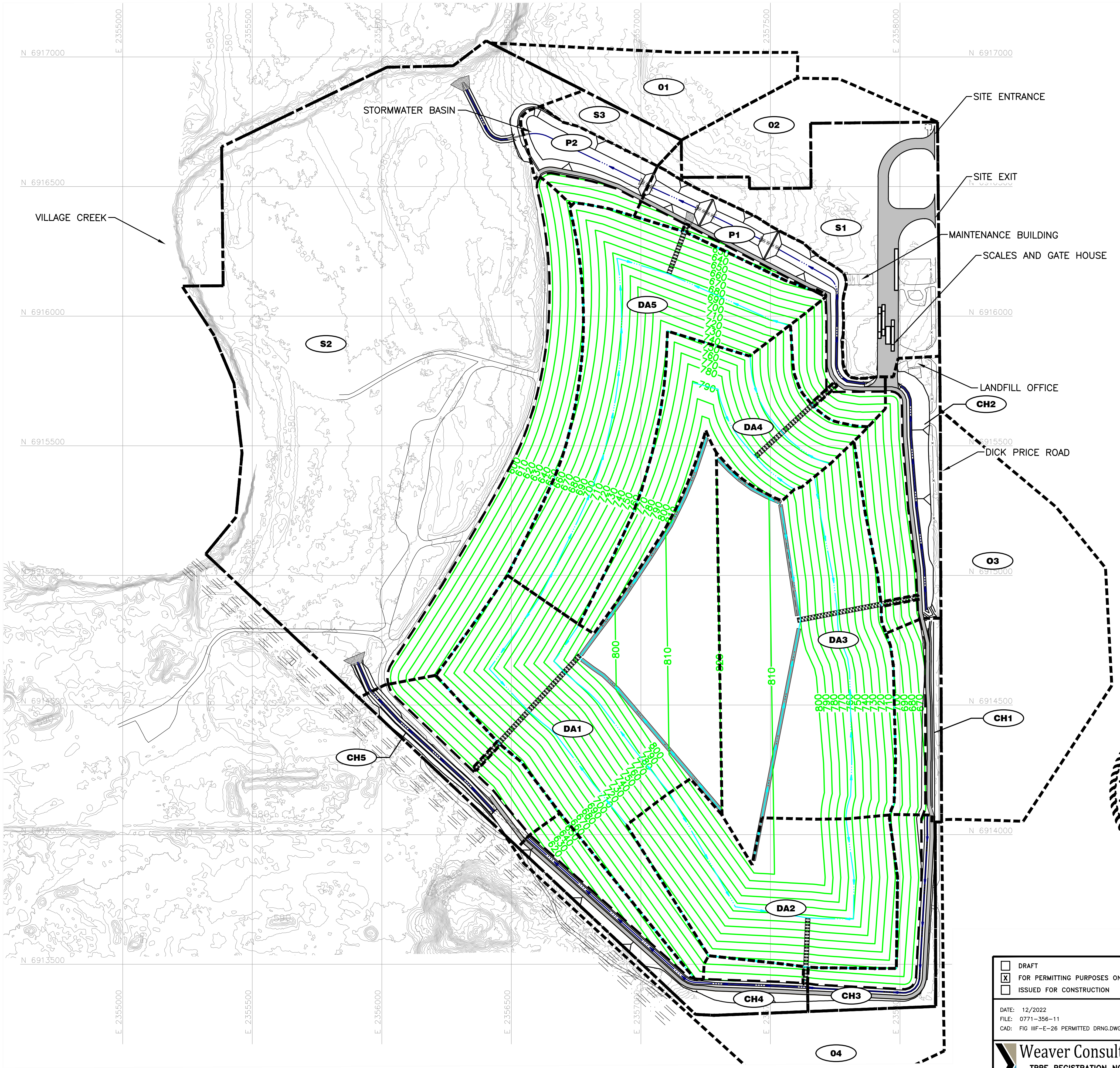

0 200 400
SCALE IN FEET

LEGEND

- PERMIT BOUNDARY
- PERMITTED LIMIT OF WASTE
- 700 PERMITTED FINAL COVER CONTOUR
- N 6915000 STATE PLANE COORDINATE
- 580 EXISTING CONTOUR
- DRAINAGE CHUTE
- PERMITTED DRAINAGE SWALE
- PERMITTED PERIMETER CHANNEL
- DRAINAGE DIVIDE
- O3 DRAINAGE AREA DESIGNATION
- TOP DECK DRAINAGE TERRACE

NOTES:

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2. PERMITTED FINAL COVER GRADES ARE DEVELOPED FROM THE LANDFILL COMPLETION PLAN BY GEOSYNTEC CONSULTANTS, INC., DATED DECEMBER 2020.

DRAINAGE AREA NO.	AREA (ACRES)	DRAINAGE AREA NO.	AREA (ACRES)
DA1	21.60	P1	5.53
DA2	11.70	P2	2.82
DA3	19.60	CH1	3.16
DA4	9.15	CH2	6.07
DA5	15.90	CH3	3.87
S1	11.05	CH4	5.00
S2	63.06	CH5	4.40
S3	1.44		
O1	5.11		
O2	4.56		
O3	17.39		
O4	6.25		

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DRAWN BY: JDW
DESIGN BY: JBM
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NO.	DATE	DESCRIPTION

MAJOR PERMIT AMENDMENT
PERMITTED LANDFILL DRAINAGE

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

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DRAWING IIIF-E-26

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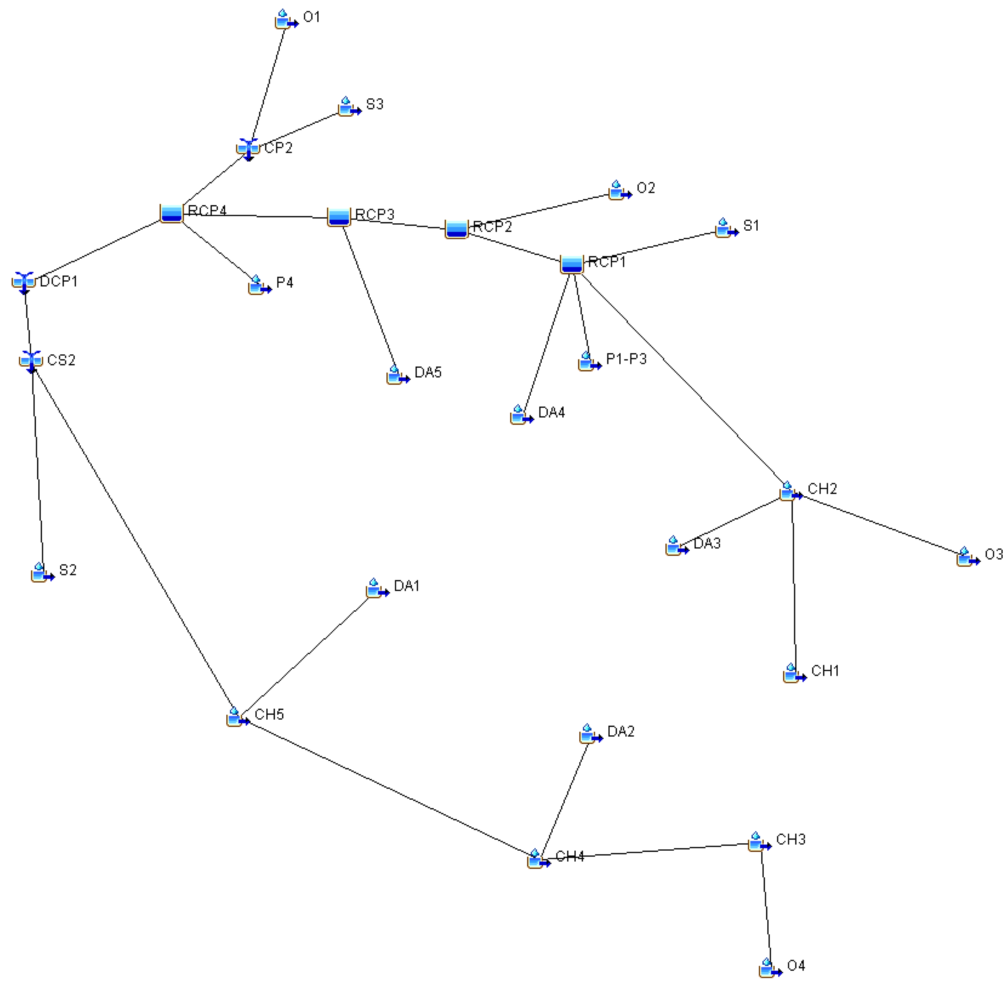
HEC-HMS OUTPUT – PERMITTED LANDFILL
25-YEAR, 24-HOUR STORM EVENT


O1
S3
CP2
RCP4
RCP3
O2
RCP2
S1
RCP1
DCP1
P4
CS2
DA5
P1-P3
DA4
CH2
DA3
O3
S2
DA1
CH1
CH5
DA2
CH3
CH4
O4

Project: Existing_Conditions_Model
Simulation Run: 25-Year Storm
Simulation Start: 29 December 2020, 01:00
Simulation End: 31 December 2020, 23:00

HMS Version: 4.9
Executed: 25 September 2022, 18:17

Global Parameter Summary - Subbasin

	Area (MIē)
Element Name	**Area (MIē)**
O1	0.01
S3	0
Da3	0.03
O3	0.03
Ch1	0
Ch2	0.01
S1	0.02
Da4	0.01
P1 - P3	0.01
O2	0.01
Da5	0.02
S2	0.1
Da2	0.02
O4	0.01
Ch3	0.01
Ch4	0.01
Da1	0.03
Ch5	0.01
P4	0

Downstream

Element Name	Downstream
O1	Cp2
S3	Cp2
Da3	Ch2
O3	Ch2
Ch1	Ch2
Ch2	Rcp1
S1	Rcp1
Da4	Rcp1
P1 - P3	Rcp1
O2	Rcp2
Da5	Rcp3
S2	Cs2
Da2	Ch4
O4	Ch3
Ch3	Ch4
Ch4	Ch5
Da1	Ch5
Ch5	Cs2
P4	Rcp4

Loss Rate: Scs

Element Name	Percent Impervious Area	Curve Number
O1	0	84
S3	0	84
O3	0	84
S1	0	84
P1 - P3	0	99
O2	0	84
S2	0	84
O4	0	84
P4	0	99

Transform: Snyder

Element Name	Snyder Method	Snyder Tp	Snyder Cp
O1	Standard	0.23	0.69
S3	Standard	0.23	0.73
O3	Standard	0.14	0.62
S1	Standard	0.2	0.66
O2	Standard	0.2	0.69
S2	Standard	0.69	0.65
O4	Standard	0.14	0.64

Transform: Kinematic Wave

Element Name	Transform
Da3	Kinematic Wave
Ch1	Kinematic Wave
Ch2	Kinematic Wave
Da4	Kinematic Wave
Da5	Kinematic Wave
Da2	Kinematic Wave
Ch3	Kinematic Wave
Ch4	Kinematic Wave
Da1	Kinematic Wave
Ch5	Kinematic Wave

Transform: Scs

Element Name	Lag	Unitgraph Type
P1 - P3	0.1	Standard
P4	0.1	Standard

Global Results Summary

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (IN)
O1	0.01	21.48	29Dec2020, 13:20	5.3
S3	0	6.12	29Dec2020, 13:15	5.3
Da3	0.03	115.51	29Dec2020, 13:10	4.81
O3	0.03	83.69	29Dec2020, 13:15	5.3
Ch1	0	18.28	29Dec2020, 13:10	5.05
Ch2	0.07	242.18	29Dec2020, 13:10	5.06
S1	0.02	48.16	29Dec2020, 13:15	5.3
Da4	0.01	54.5	29Dec2020, 13:10	5.25
P1 - P3	0.01	36	29Dec2020, 13:10	7.05
Rcp1	0.11	340.9	29Dec2020, 13:15	5.26
O2	0.01	20.32	29Dec2020, 13:15	5.3

Rcp2	0.12	329.67	29Dec2020, 13:20	5.26
Da5	0.02	89.07	29Dec2020, 13:10	4.41
Rcp3	0.14	375.55	29Dec2020, 13:20	5.11
Cp2	0.01	27.57	29Dec2020, 13:20	5.3
S2	0.1	140.92	29Dec2020, 13:45	5.3
Da2	0.02	70.12	29Dec2020, 13:10	5.21
O4	0.01	30.55	29Dec2020, 13:15	5.3
Ch3	0.02	47.18	29Dec2020, 13:10	5.16
Ch4	0.04	143.14	29Dec2020, 13:10	5.15
Da1	0.03	129.86	29Dec2020, 13:10	5.13
Ch5	0.08	294.02	29Dec2020, 13:10	5.14
Cs2	0.18	350.07	29Dec2020, 13:10	5.23
P4	0	18.42	29Dec2020, 13:10	7.05
Rcp4	0.16	330.63	29Dec2020, 13:30	5.18
Dcp1	0.34	533.14	29Dec2020, 13:30	5.21

Subbasin: O1

Area (MIē) : 0.01
Downstream : Cp2

Loss Rate: Scs

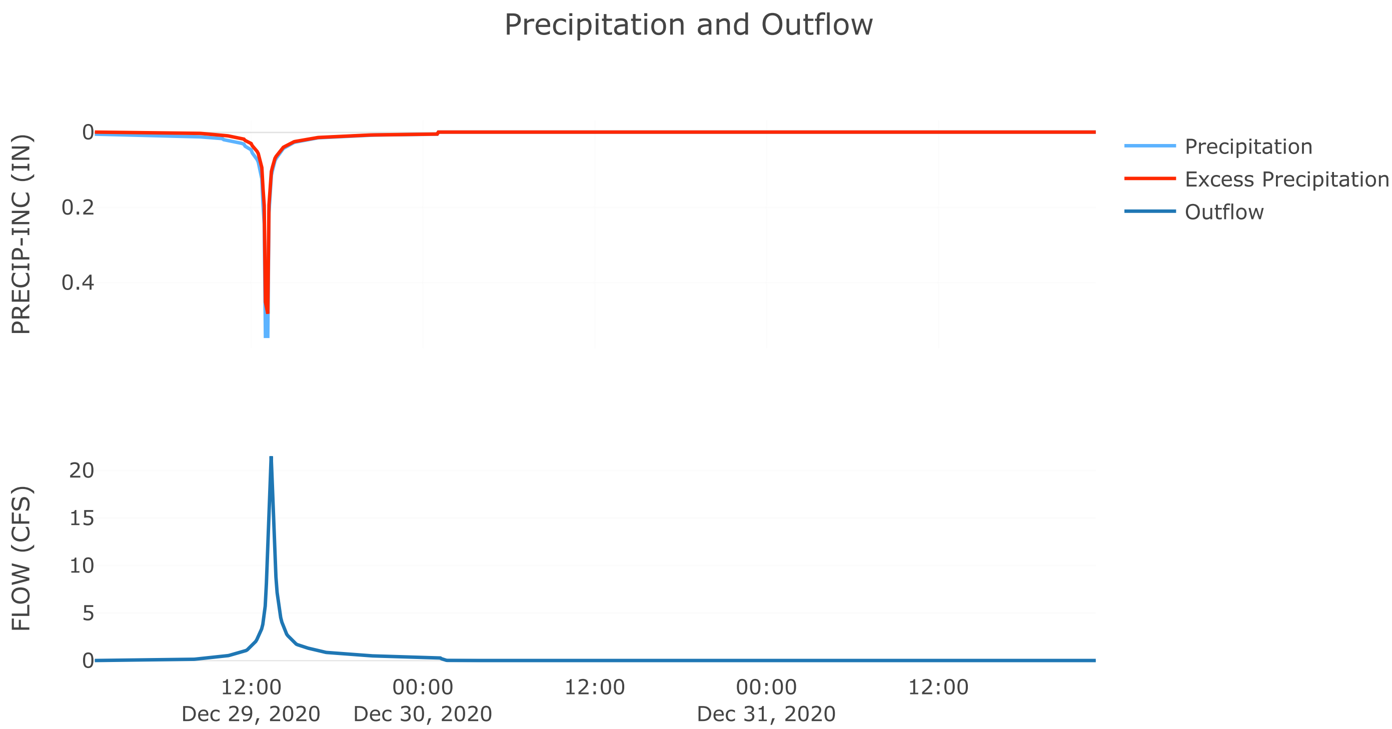
Percent Impervious Area	0
Curve Number	84

Transform: Snyder

Snyder Method	Standard
Snyder Tp	0.23
Snyder Cp	0.69

Results: O1

Peak Discharge (CFS)	21.48
Time of Peak Discharge	29Dec2020, 13:20
Volume (IN)	5.3
Precipitation Volume (AC - FT)	3.06
Loss Volume (AC - FT)	0.8
Excess Volume (AC - FT)	2.26
Direct Runoff Volume (AC - FT)	2.26
Baseflow Volume (AC - FT)	0


Precipitation and Outflow
PRECIP-INC (IN)
0
0.2
0.4
Precipitation
Excess Precipitation
Outflow
FLOW (CFS)
20
15
10
5
0
12:00
Dec 29, 2020
00:00
Dec 30, 2020
12:00
00:00
Dec 31, 2020
12:00

Subbasin: S3

Area (MIē) : 0
Downstream : Cp2

Loss Rate: Scs

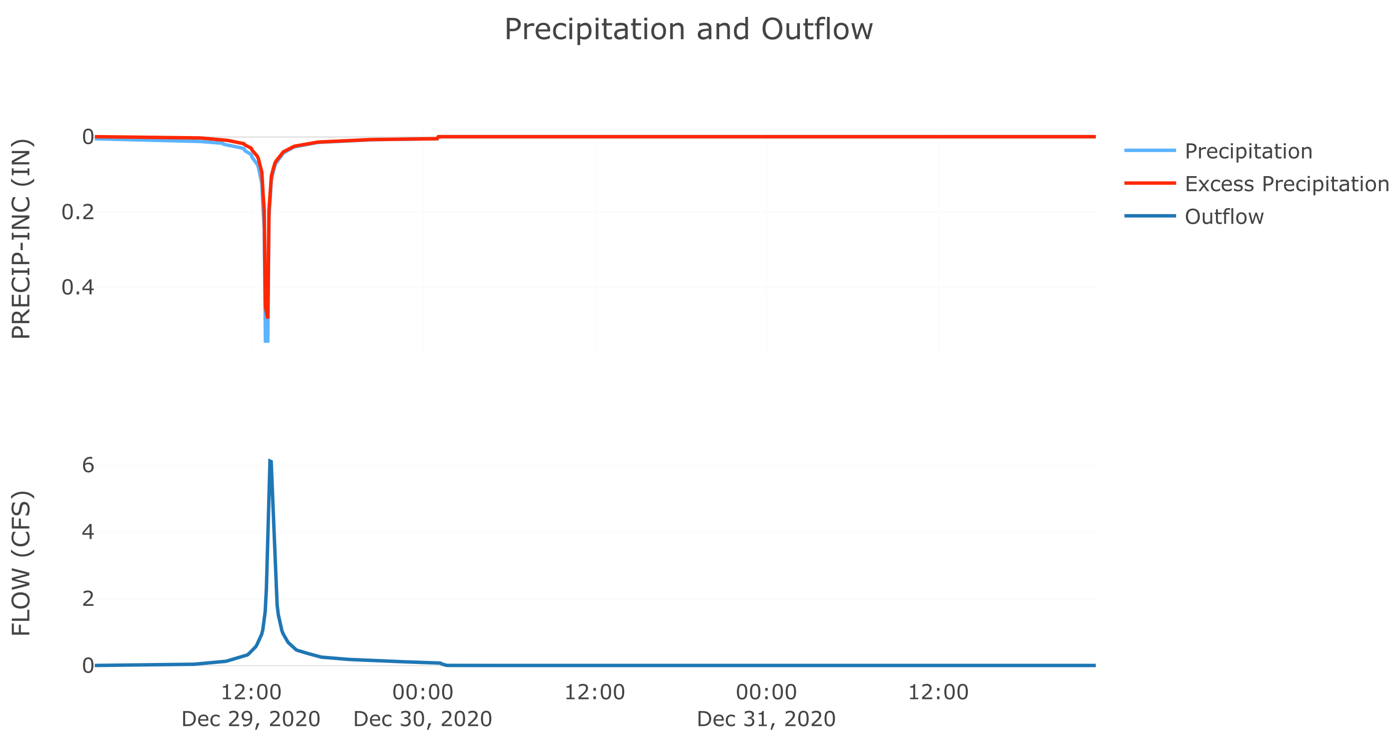
Percent Impervious Area	0
Curve Number	84

Transform: Snyder

Snyder Method	Standard
Snyder Tp	0.23
Snyder Cp	0.73

Results: S3

Peak Discharge (CFS)	6.12
Time of Peak Discharge	29Dec2020, 13:15
Volume (IN)	5.3
Precipitation Volume (AC - FT)	0.84
Loss Volume (AC - FT)	0.22
Excess Volume (AC - FT)	0.62
Direct Runoff Volume (AC - FT)	0.62
Baseflow Volume (AC - FT)	0

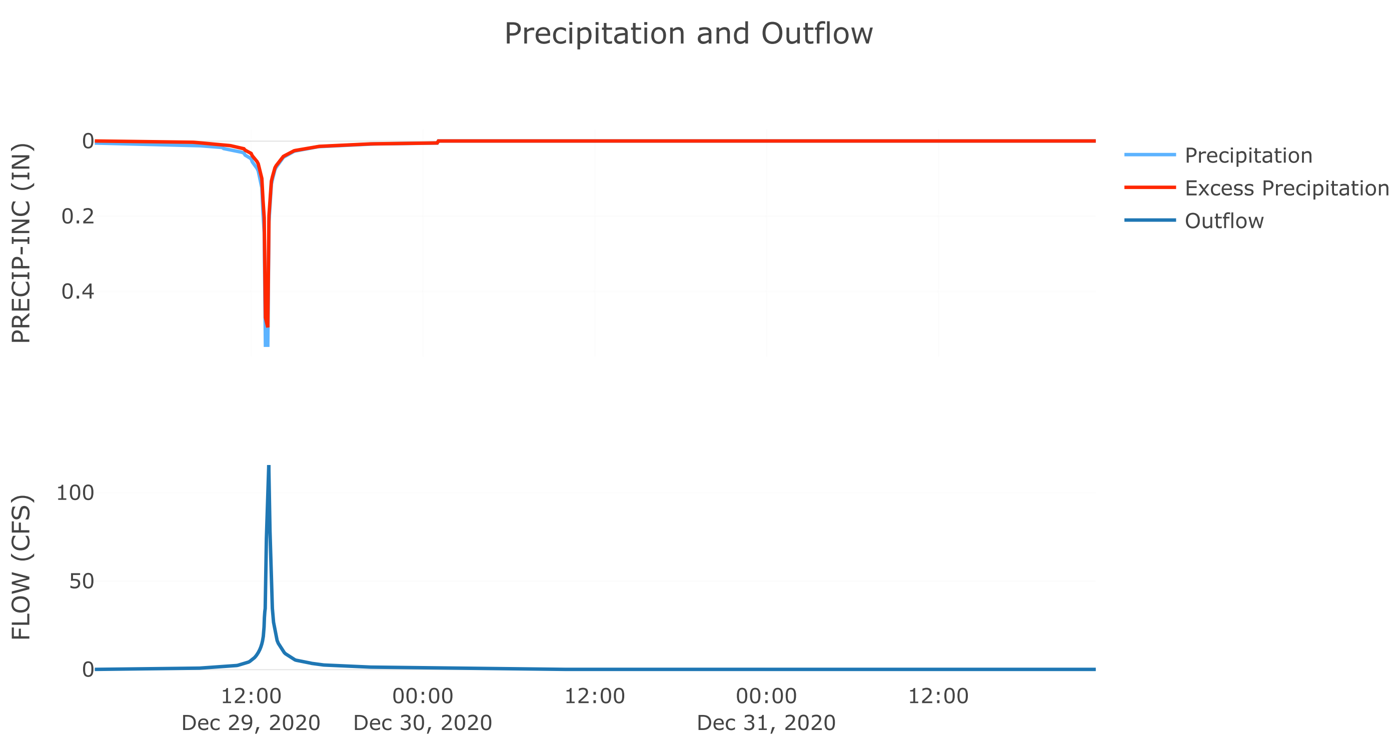

Precipitation and Outflow
PRECIP-INC (IN)
0
0.2
0.4
Precipitation
Excess Precipitation
Outflow
FLOW (CFS)
6
4
2
0
12:00
Dec 29, 2020
00:00
Dec 30, 2020
12:00
00:00
Dec 31, 2020
12:00

Subbasin: DA3

Area (MIē) : 0.03
Downstream : Ch2
Transform : Kinematic Wave

Results: DA3

Peak Discharge (CFS)	115.51
Time of Peak Discharge	29Dec2020, 13:10
Volume (IN)	4.81
Precipitation Volume (AC - FT)	11.7
Loss Volume (AC - FT)	2.68
Excess Volume (AC - FT)	9.02
Direct Runoff Volume (AC - FT)	7.85
Baseflow Volume (AC - FT)	0



Subbasin: O3

Area (MIē) : 0.03
Downstream : Ch2

Loss Rate: Scs

Percent Impervious Area	0
Curve Number	84

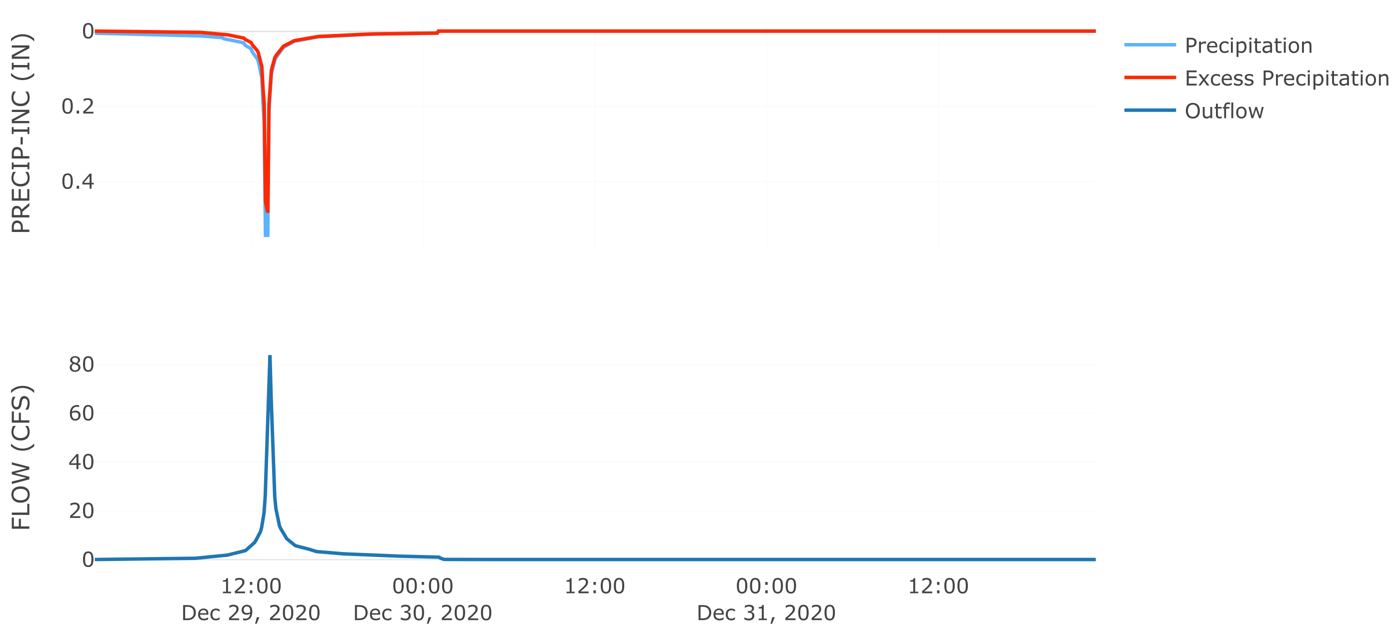
Transform: Snyder

Snyder Method	Standard
Snyder Tp	0.14
Snyder Cp	0.62

Results: O3

Peak Discharge (CFS)	83.69
Time of Peak Discharge	29Dec2020, 13:15
Volume (IN)	5.3
Precipitation Volume (AC - FT)	10.4
Loss Volume (AC - FT)	2.71
Excess Volume (AC - FT)	7.69
Direct Runoff Volume (AC - FT)	7.69
Baseflow Volume (AC - FT)	0

Precipitation and Outflow

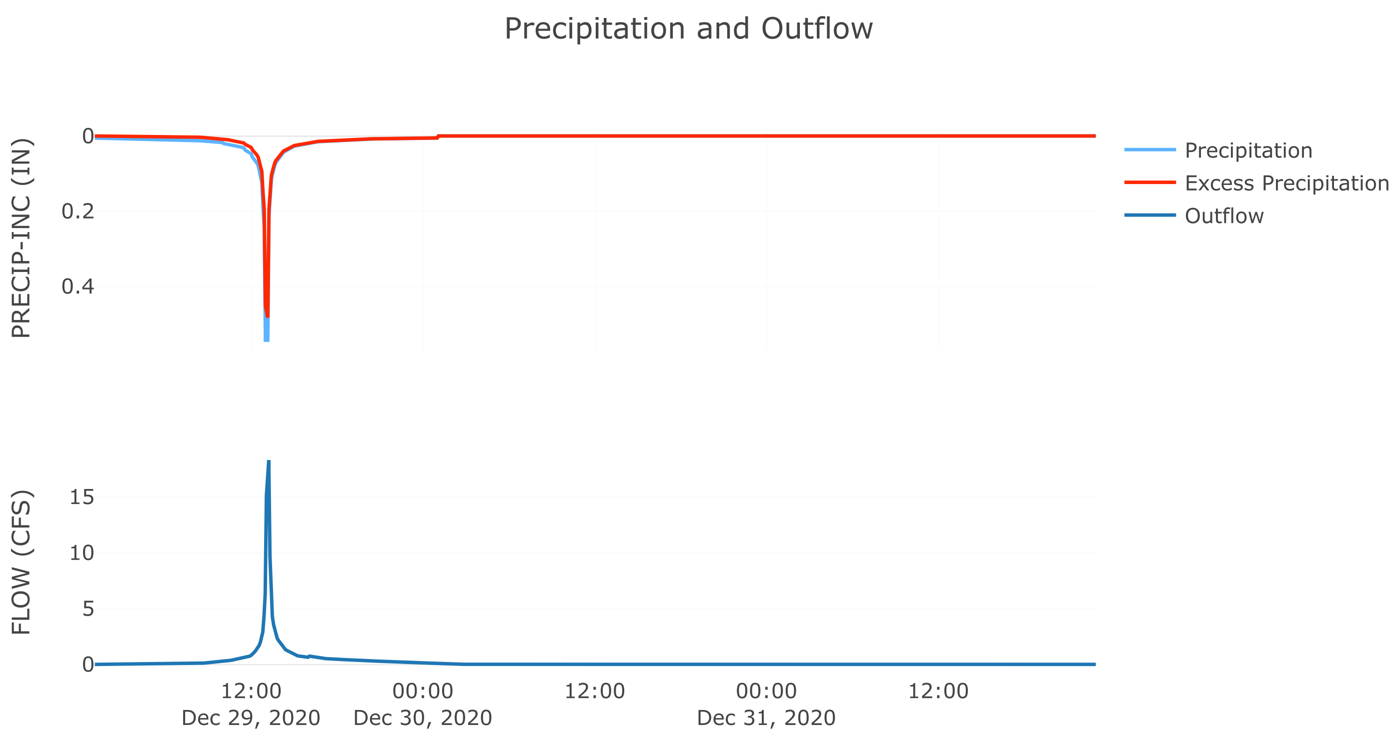


Subbasin: CH1

Area (MI2) : 0
Downstream : Ch2
Transform : Kinematic Wave

Results: CH1

Peak Discharge (CFS)	18.28
Time of Peak Discharge	29Dec2020, 13:10
Volume (IN)	5.05
Precipitation Volume (AC - FT)	1.87
Loss Volume (AC - FT)	0.49
Excess Volume (AC - FT)	1.39
Direct Runoff Volume (AC - FT)	1.32
Baseflow Volume (AC - FT)	0

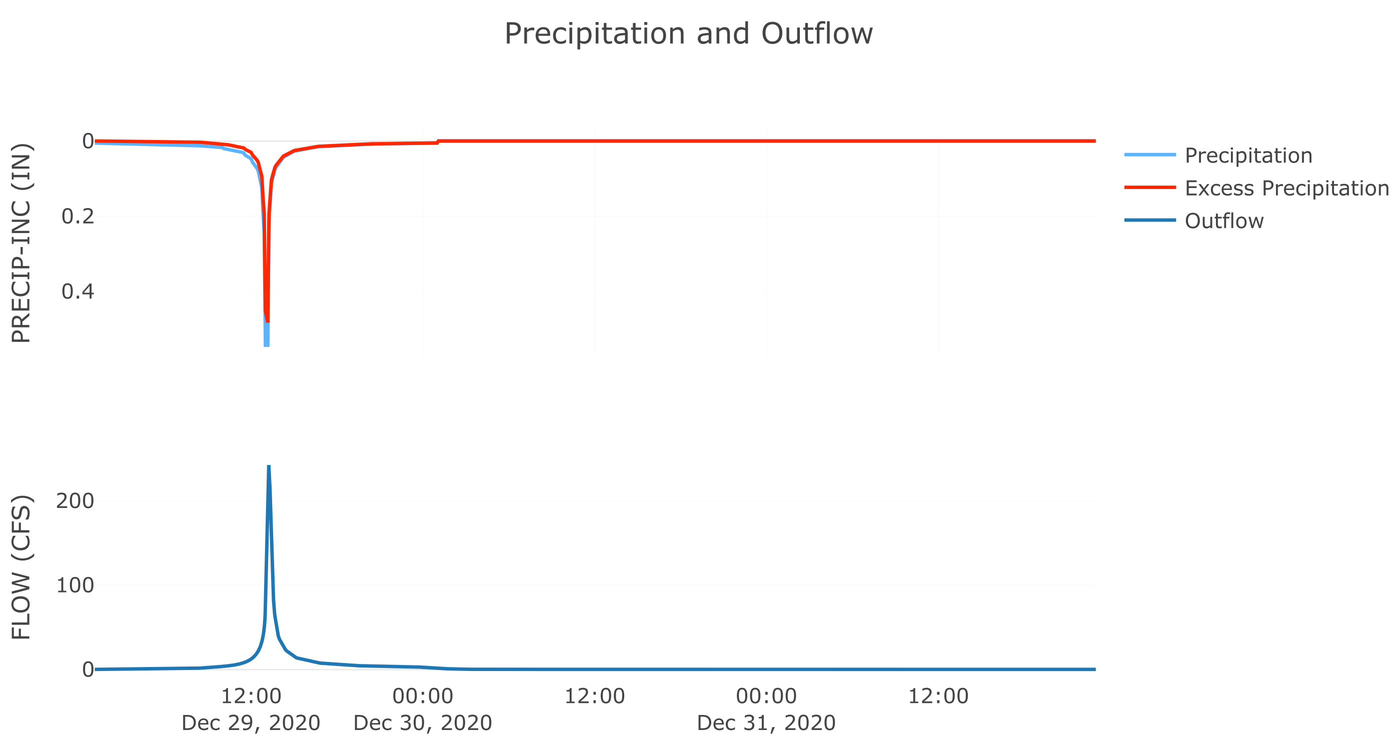


Subbasin: CH2

Area (MIē) : 0.01
Downstream : Rcp1
Transform : Kinematic Wave

Results: CH2

Peak Discharge (CFS)	242.18
Time of Peak Discharge	29Dec2020, 13:10
Volume (IN)	5.06
Precipitation Volume (AC - FT)	27.61
Loss Volume (AC - FT)	7.19
Excess Volume (AC - FT)	20.41
Direct Runoff Volume (AC - FT)	19.5
Baseflow Volume (AC - FT)	0



Subbasin: S1

Area (MI²) : 0.02
Downstream : Rcp1

Loss Rate: Scs

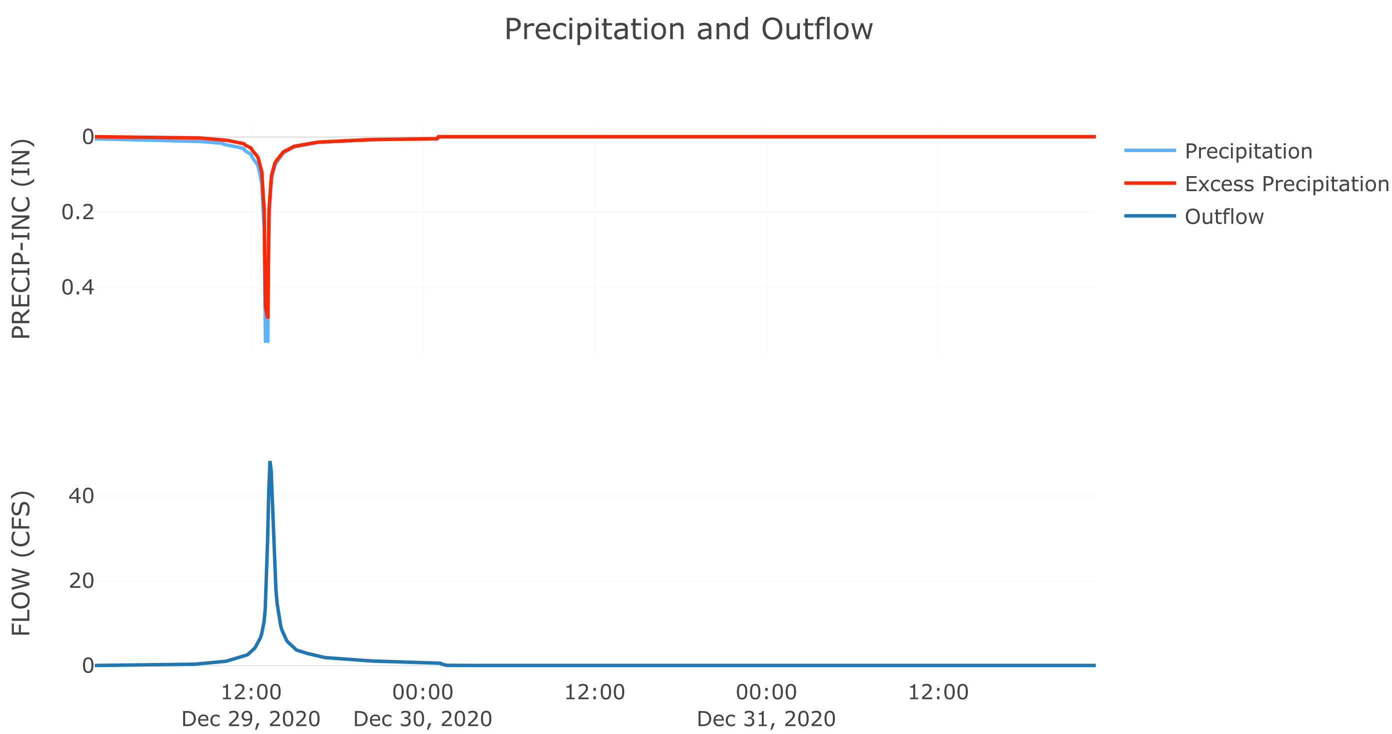
Percent Impervious Area	0
Curve Number	84

Transform: Snyder

Snyder Method	Standard
Snyder Tp	0.2
Snyder Cp	0.66

Results: S1

Peak Discharge (CFS)	48.16
Time of Peak Discharge	29Dec2020, 13:15
Volume (IN)	5.3
Precipitation Volume (AC - FT)	6.62
Loss Volume (AC - FT)	1.72
Excess Volume (AC - FT)	4.89
Direct Runoff Volume (AC - FT)	4.89
Baseflow Volume (AC - FT)	0



Precipitation and Outflow
PRECIP-INC (IN)
0
0.2
0.4
Precipitation
Excess Precipitation
Outflow
FLOW (CFS)
40
20
0
12:00
Dec 29, 2020
00:00
Dec 30, 2020
12:00
00:00
Dec 31, 2020
12:00

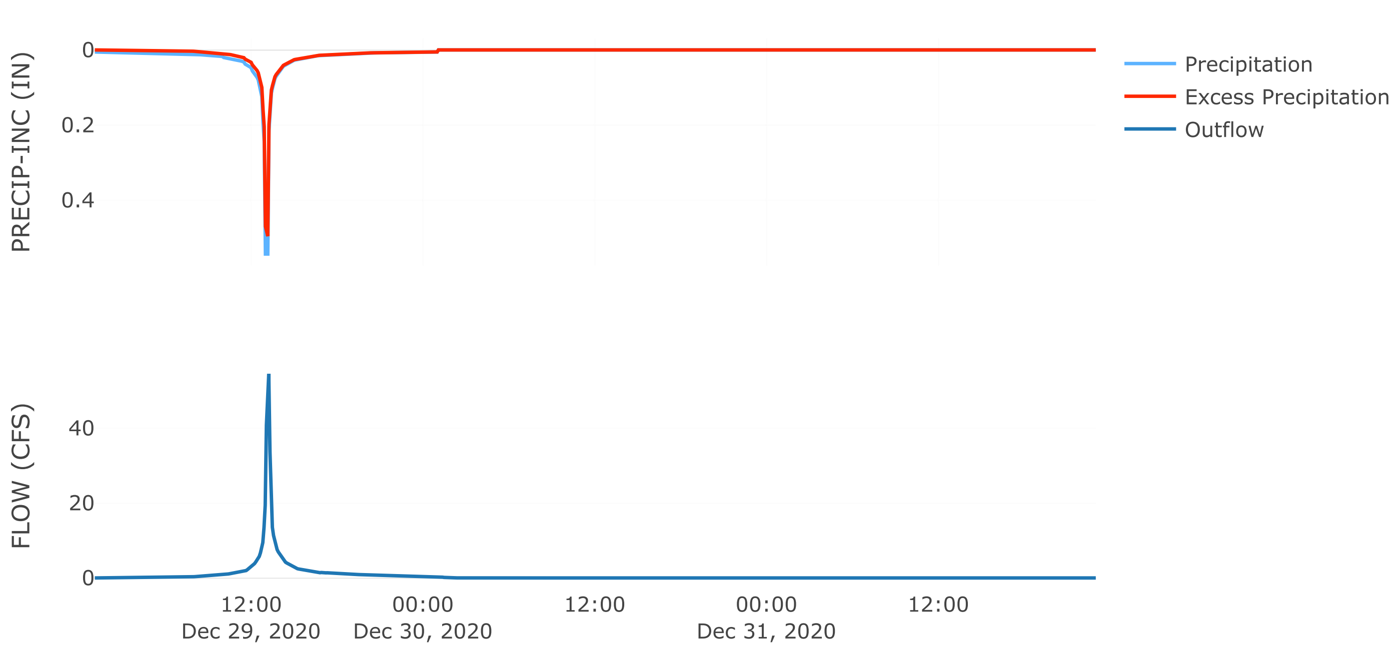
Subbasin: DA4

Area (MIē) : 0.01
Downstream : Rcp1
Transform : Kinematic Wave

Results: DA4

Peak Discharge (CFS)	54.5
Time of Peak Discharge	29Dec2020, 13:10
Volume (IN)	5.25
Precipitation Volume (AC - FT)	5.47
Loss Volume (AC - FT)	1.25
Excess Volume (AC - FT)	4.22
Direct Runoff Volume (AC - FT)	4.01
Baseflow Volume (AC - FT)	0

Precipitation and Outflow



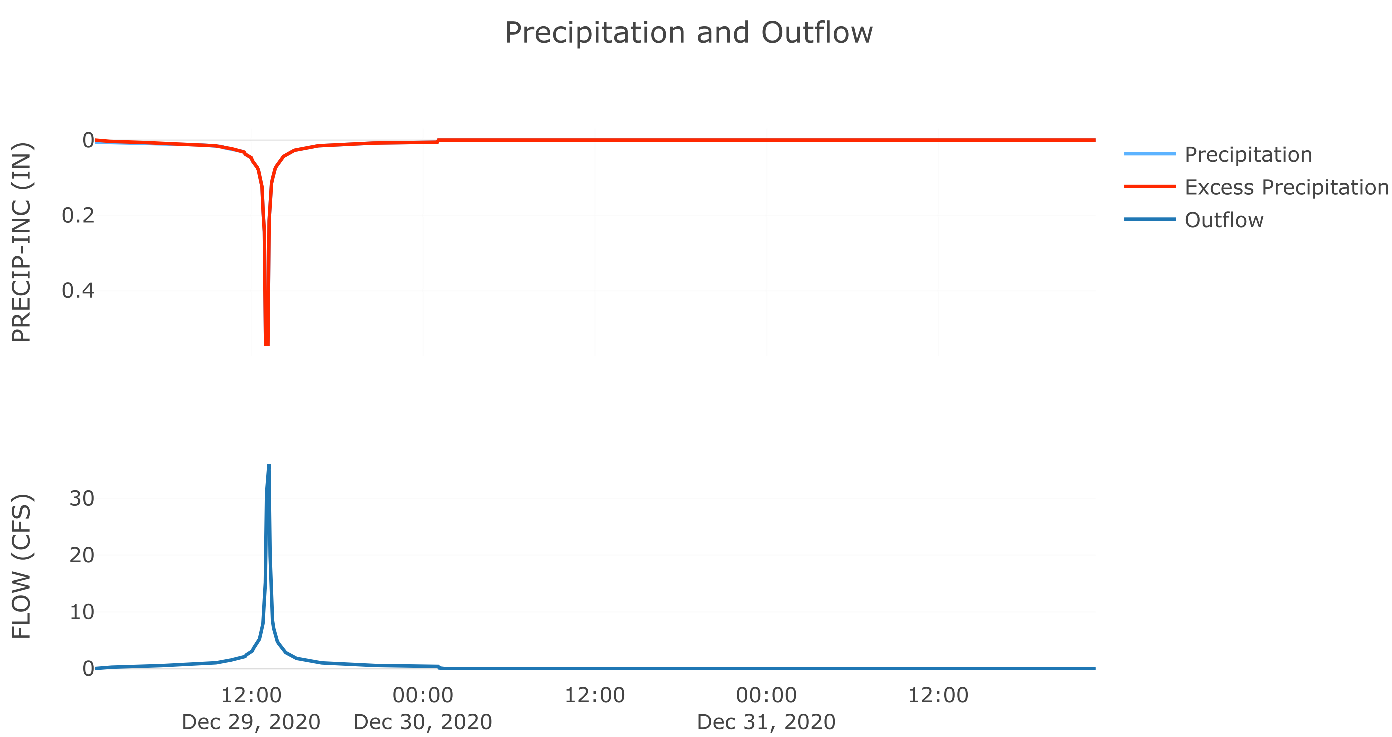
Subbasin: P1-P3

Area (MIē) : 0.01
Downstream : Rcp1

Loss Rate: Scs	
Percent Impervious Area	0
Curve Number	99

Transform: Scs	
Lag	0.1
Unitgraph Type	Standard

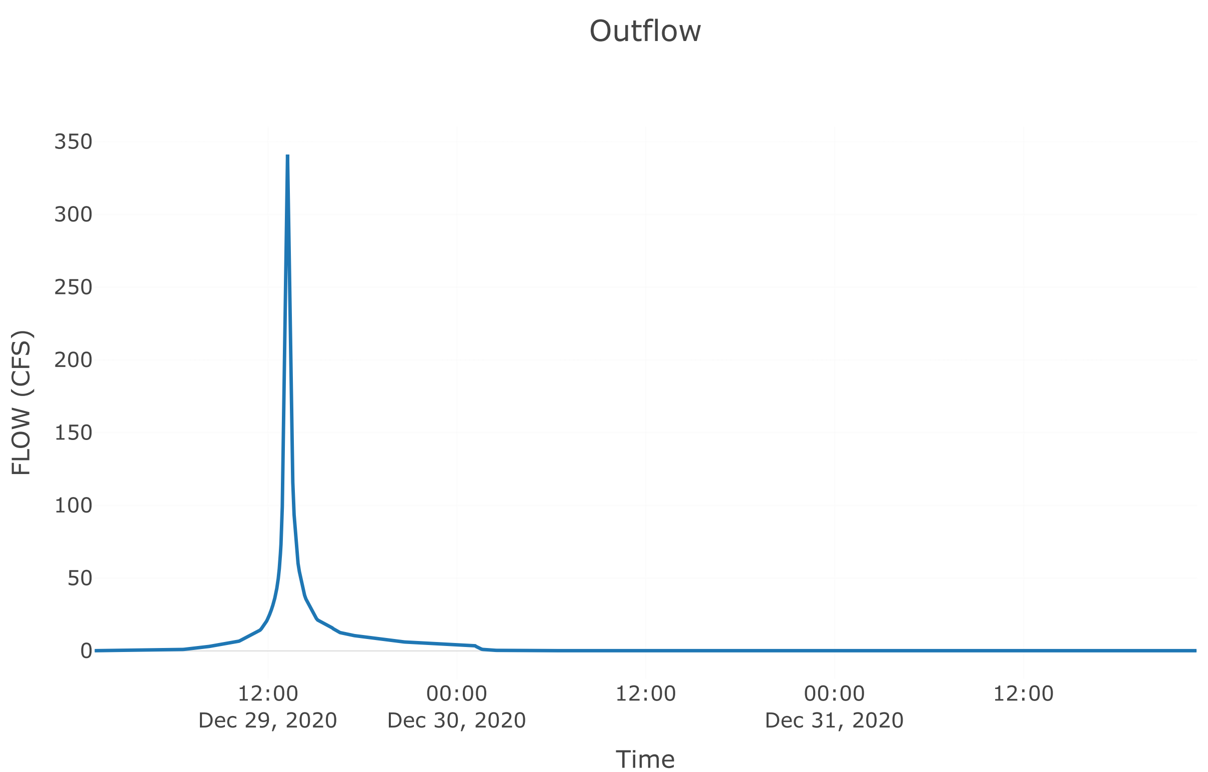
Results: P1-P3	
Peak Discharge (CFS)	36
Time of Peak Discharge	29Dec2020, 13:10
Volume (IN)	7.05
Precipitation Volume (AC - FT)	3.29
Loss Volume (AC - FT)	0.05
Excess Volume (AC - FT)	3.23
Direct Runoff Volume (AC - FT)	3.23
Baseflow Volume (AC - FT)	0



Reservoir: RCP1

Downstream : Rcp2

Results: RCP1	
Peak Discharge (CFS)	340.9
Time of Peak Discharge	29Dec2020, 13:15
Volume (IN)	5.26
Peak Inflow (CFS)	373.94
Time of Peak Inflow	29Dec2020, 13:10
Inflow Volume (AC - FT)	31.63
Maximum Storage (AC - FT)	0.65
Peak Elevation (FT)	620.57
Discharge Volume (AC - FT)	31.55



Subbasin: O2

Area (MIē) : 0.01
Downstream : Rcp2

Loss Rate: Scs

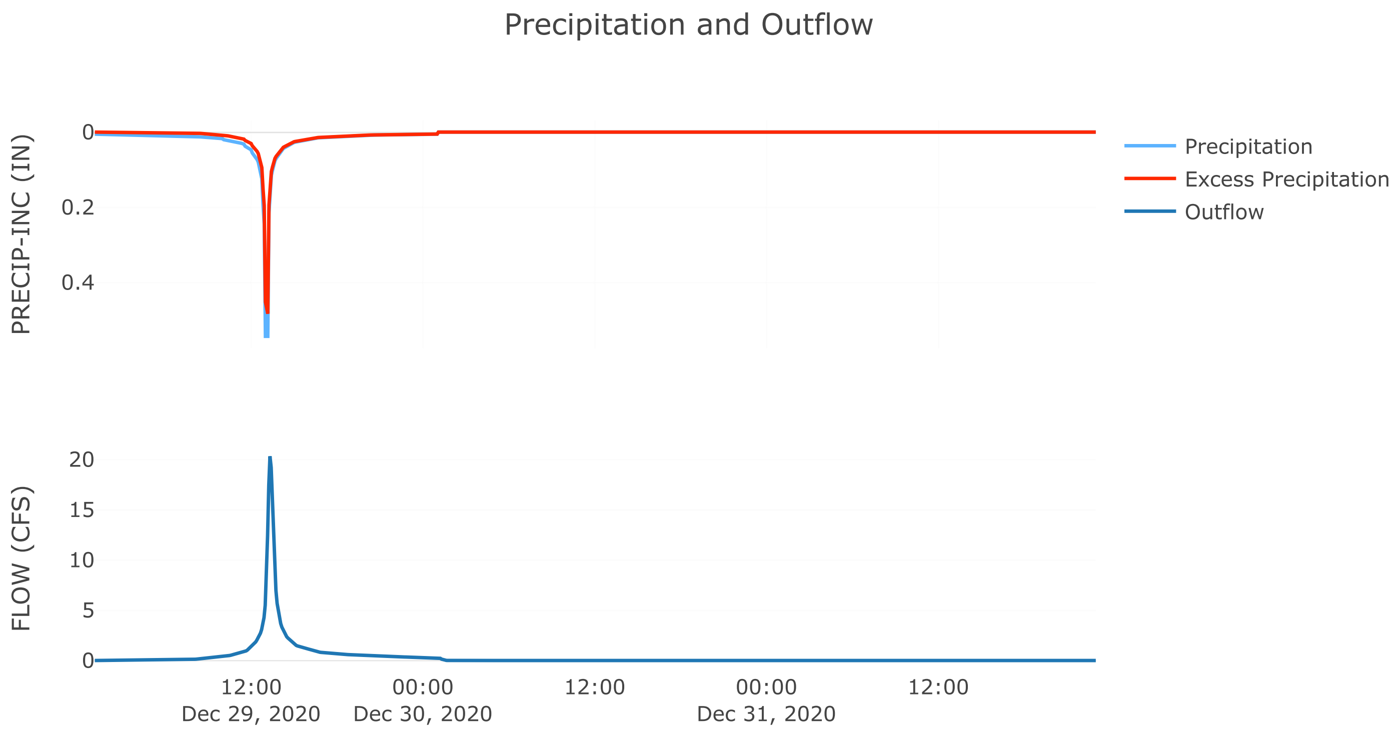
Percent Impervious Area	0
Curve Number	84

Transform: Snyder

Snyder Method	Standard
Snyder Tp	0.2
Snyder Cp	0.69

Results: O2

Peak Discharge (CFS)	20.32
Time of Peak Discharge	29Dec2020, 13:15
Volume (IN)	5.3
Precipitation Volume (AC - FT)	2.72
Loss Volume (AC - FT)	0.71
Excess Volume (AC - FT)	2.01
Direct Runoff Volume (AC - FT)	2.01
Baseflow Volume (AC - FT)	0

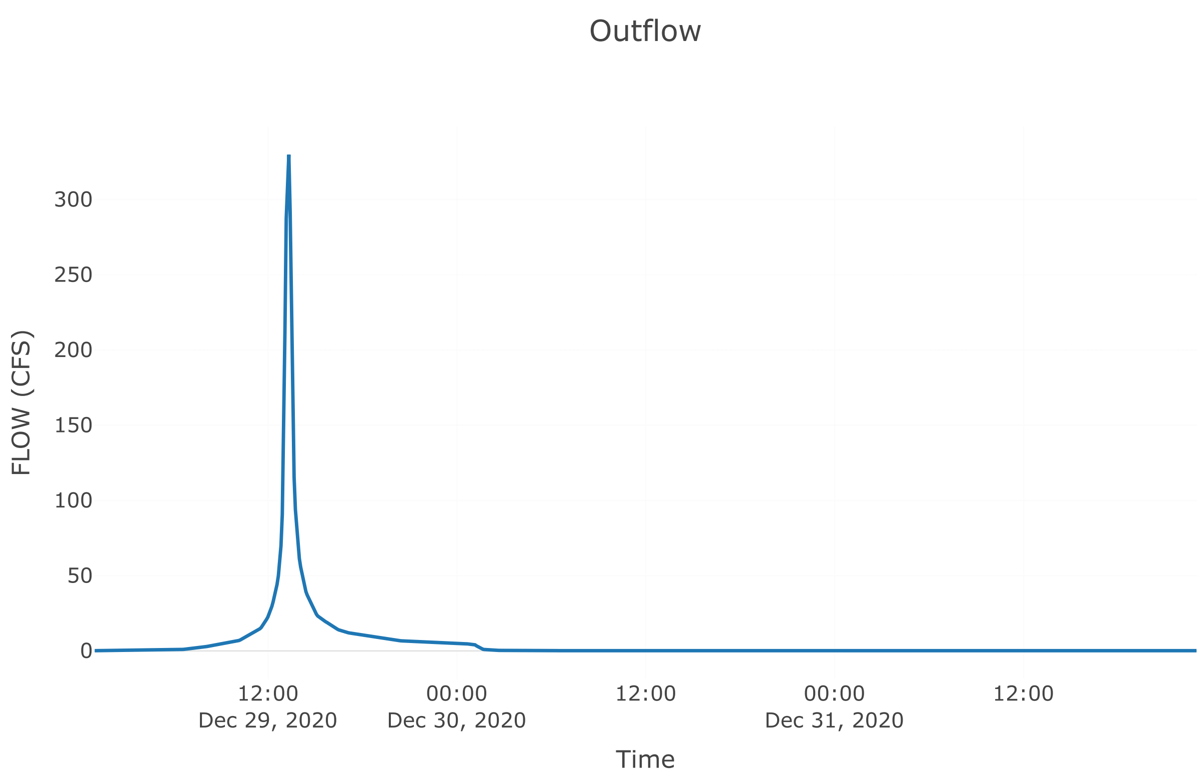


Precipitation and Outflow
PRECIP-INC (IN)
0
0.2
0.4
Precipitation
Excess Precipitation
Outflow
FLOW (CFS)
20
15
10
5
0
12:00
Dec 29, 2020
00:00
Dec 30, 2020
12:00
00:00
Dec 31, 2020
12:00

Reservoir: RCP2

Downstream : Rcp3

Results: RCP2	
Peak Discharge (CFS)	329.67
Time of Peak Discharge	29Dec2020, 13:20
Volume (IN)	5.26
Peak Inflow (CFS)	361.22
Time of Peak Inflow	29Dec2020, 13:15
Inflow Volume (AC - FT)	33.56
Maximum Storage (AC - FT)	1.18
Peak Elevation (FT)	610.27
Discharge Volume (AC - FT)	33.5

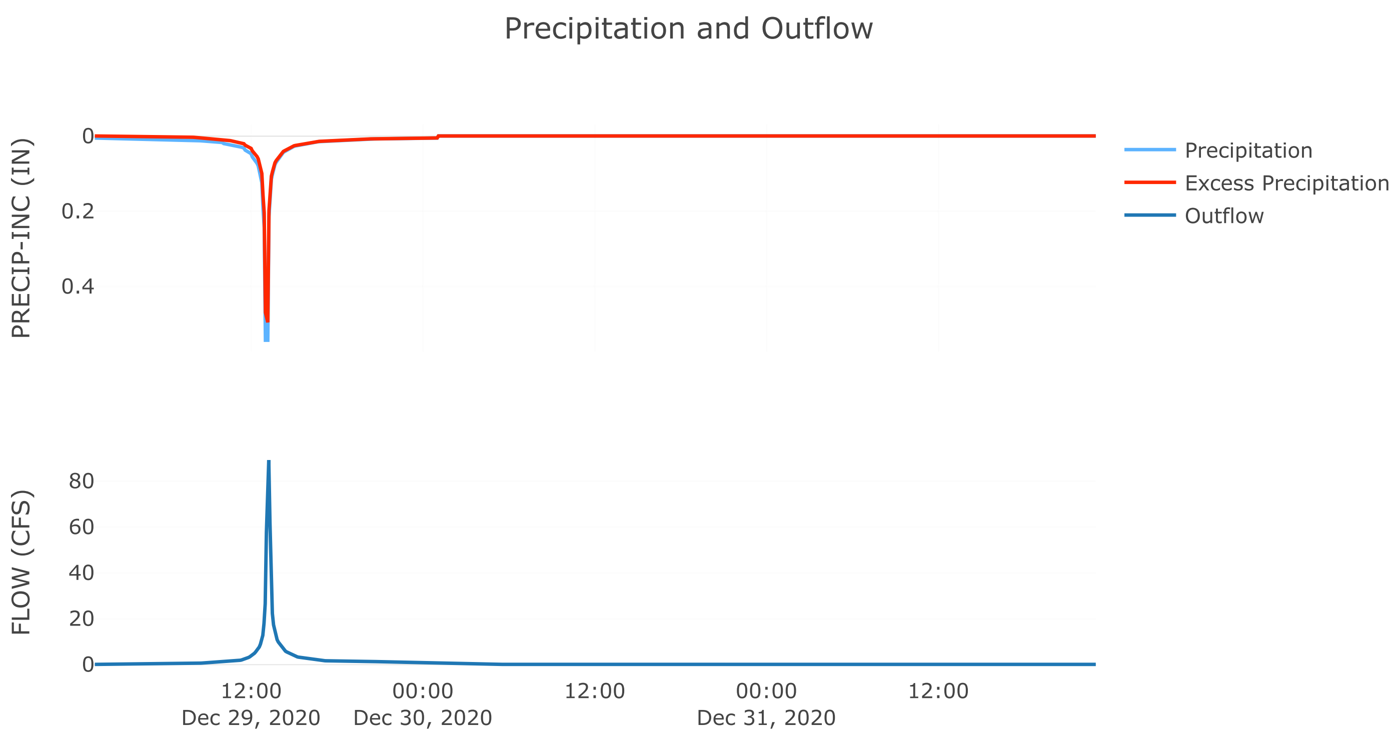


Subbasin: DA5

Area (MIē) : 0.02
Downstream : Rcp3
Transform : Kinematic Wave

Results: DA5

Peak Discharge (CFS)	89.07
Time of Peak Discharge	29Dec2020, 13:10
Volume (IN)	4.41
Precipitation Volume (AC - FT)	9.48
Loss Volume (AC - FT)	2.17
Excess Volume (AC - FT)	7.31
Direct Runoff Volume (AC - FT)	5.83
Baseflow Volume (AC - FT)	0

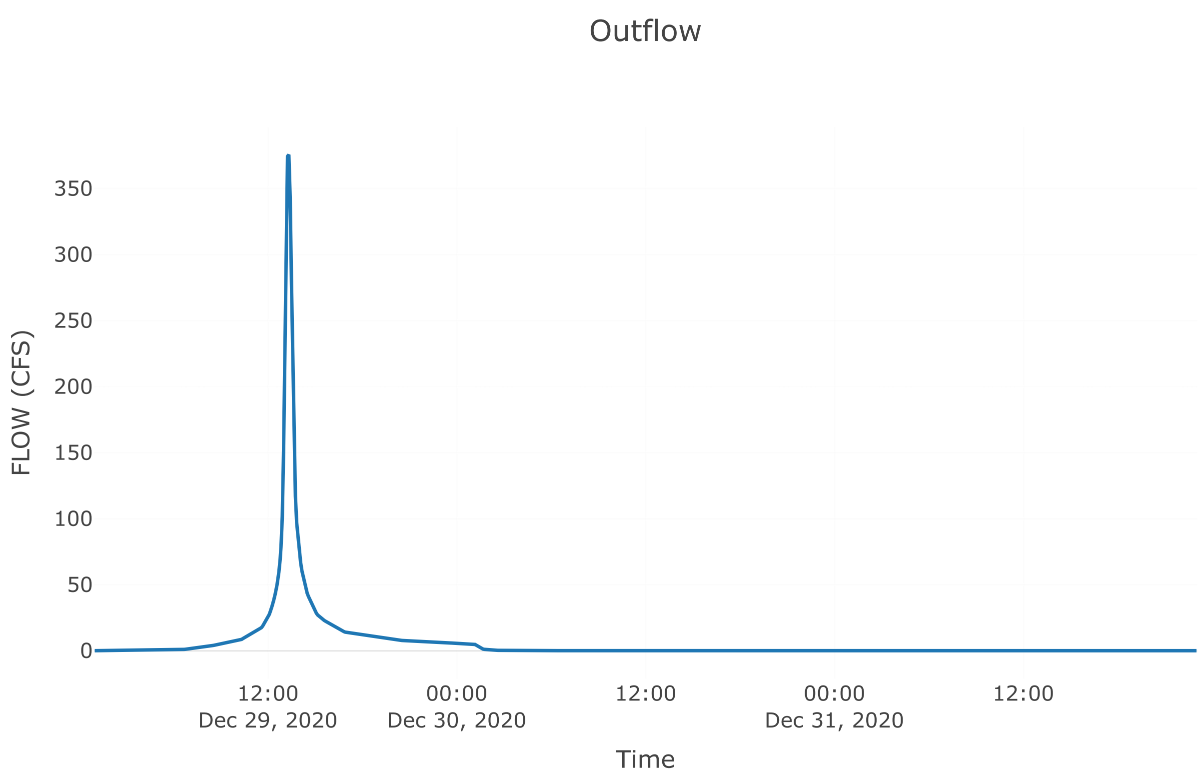


Reservoir: RCP3

Downstream : Rcp4

Results: RCP3

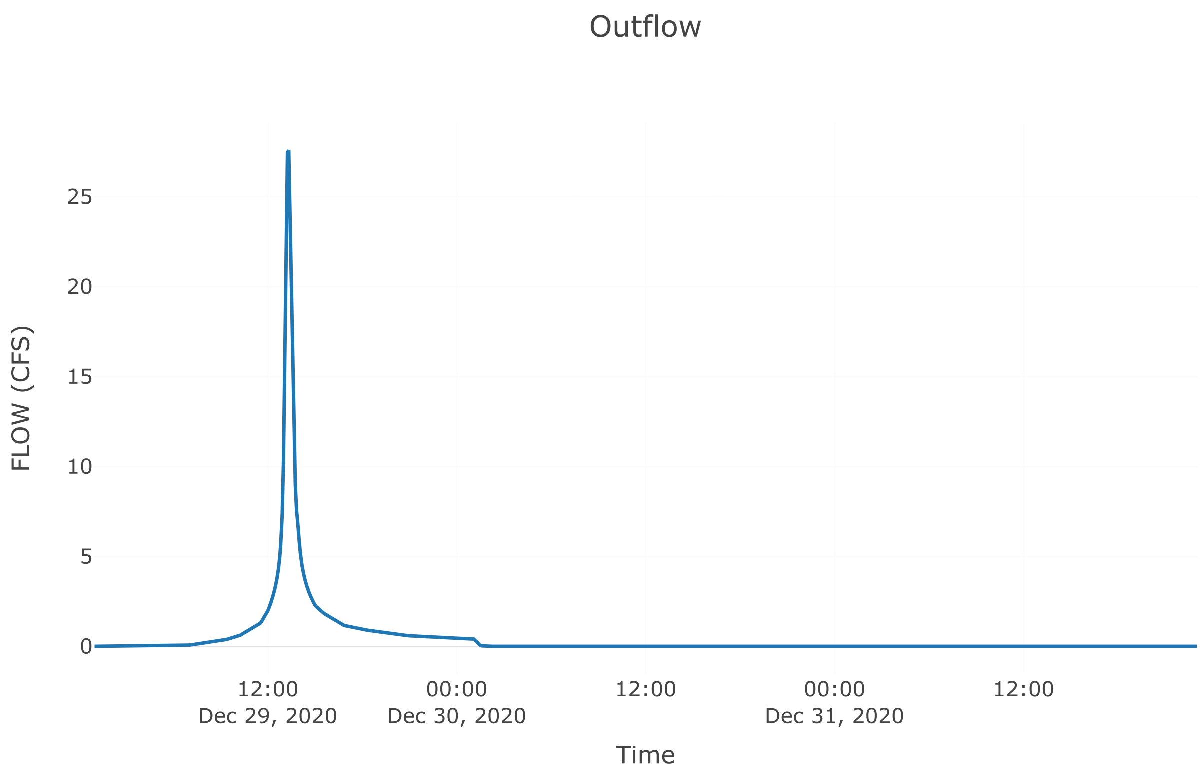
Peak Discharge (CFS)	375.55
Time of Peak Discharge	29Dec2020, 13:20
Volume (IN)	5.11
Peak Inflow (CFS)	387.27
Time of Peak Inflow	29Dec2020, 13:15
Inflow Volume (AC - FT)	39.33
Maximum Storage (AC - FT)	0.97
Peak Elevation (FT)	602.64
Discharge Volume (AC - FT)	39.29



Junction: CP2

Downstream : Rcp4

Results: CP2	
Peak Discharge (CFS)	27.57
Time of Peak Discharge	29Dec2020, 13:20
Volume (IN)	5.3



Subbasin: S2

Area (MIē) : 0.1
Downstream : Cs2

Loss Rate: Scs

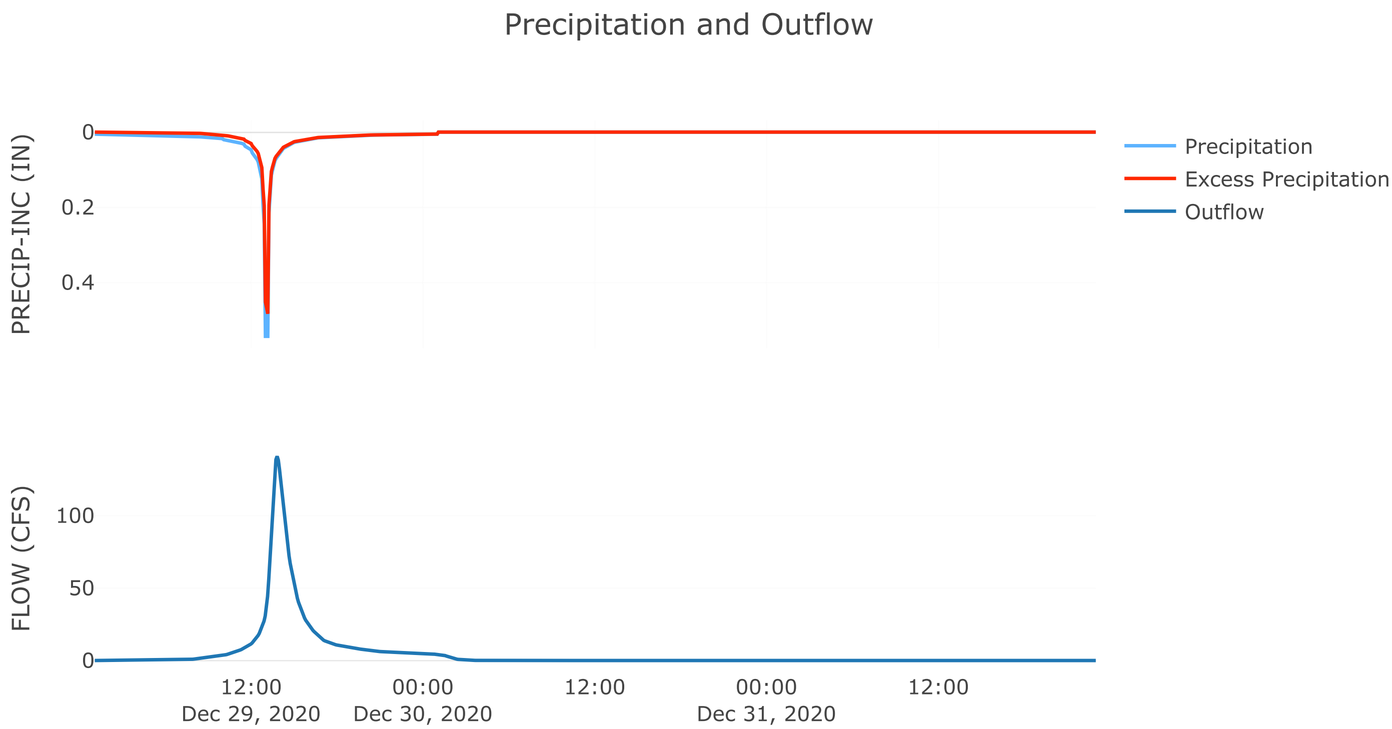
Percent Impervious Area	0
Curve Number	84

Transform: Snyder

Snyder Method	Standard
Snyder Tp	0.69
Snyder Cp	0.65

Results: S2

Peak Discharge (CFS)	140.92
Time of Peak Discharge	29Dec2020, 13:45
Volume (IN)	5.3
Precipitation Volume (AC - FT)	37.67
Loss Volume (AC - FT)	9.82
Excess Volume (AC - FT)	27.85
Direct Runoff Volume (AC - FT)	27.85
Baseflow Volume (AC - FT)	0



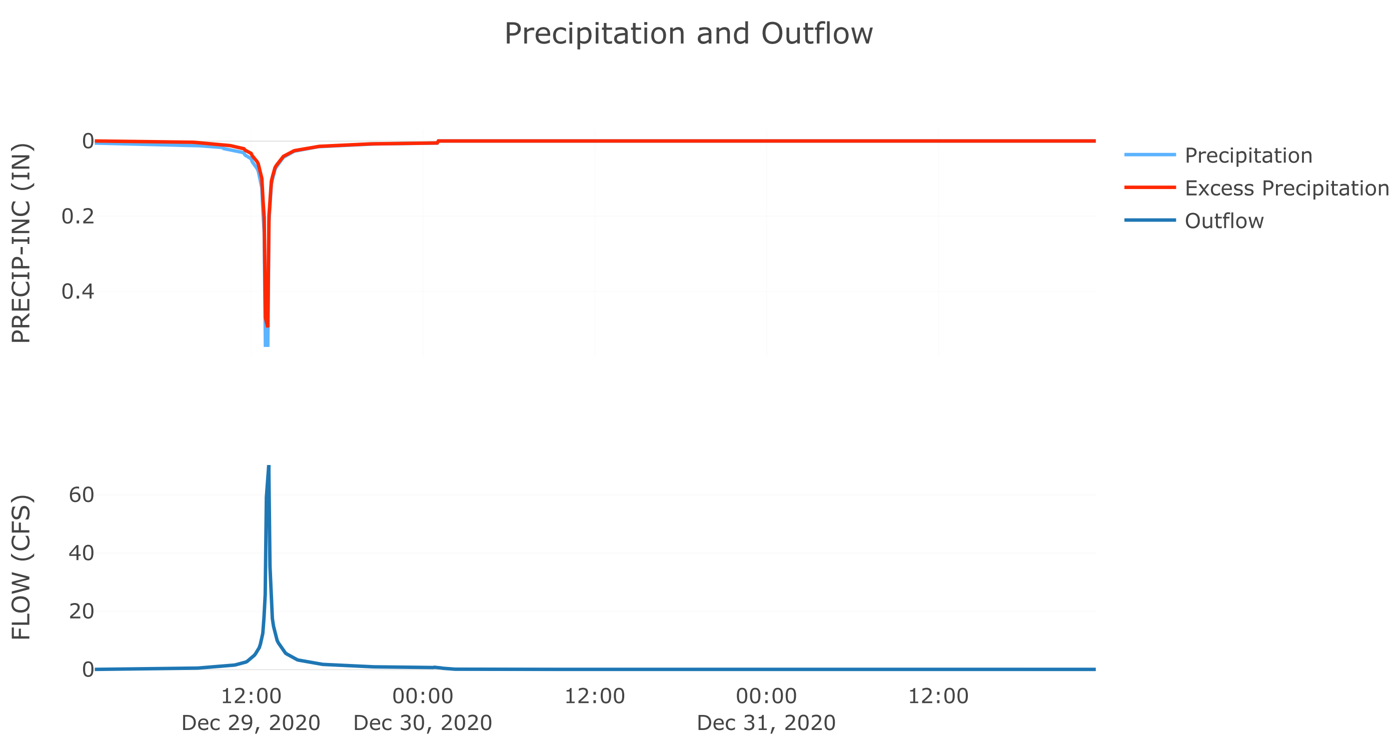
Precipitation and Outflow
PRECIP-INC (IN)
0
0.2
0.4
Precipitation
Excess Precipitation
Outflow
FLOW (CFS)
100
50
0
12:00
Dec 29, 2020
00:00
Dec 30, 2020
12:00
00:00
Dec 31, 2020
12:00

Subbasin: DA2

Area (MIē) : 0.02
Downstream : Ch4
Transform : Kinematic Wave

Results: DA2

Peak Discharge (CFS)	70.12
Time of Peak Discharge	29Dec2020, 13:10
Volume (IN)	5.21
Precipitation Volume (AC - FT)	7
Loss Volume (AC - FT)	1.6
Excess Volume (AC - FT)	5.4
Direct Runoff Volume (AC - FT)	5.08
Baseflow Volume (AC - FT)	0



Subbasin: O4

Area (MIē) : 0.01
Downstream : Ch3

Loss Rate: Scs

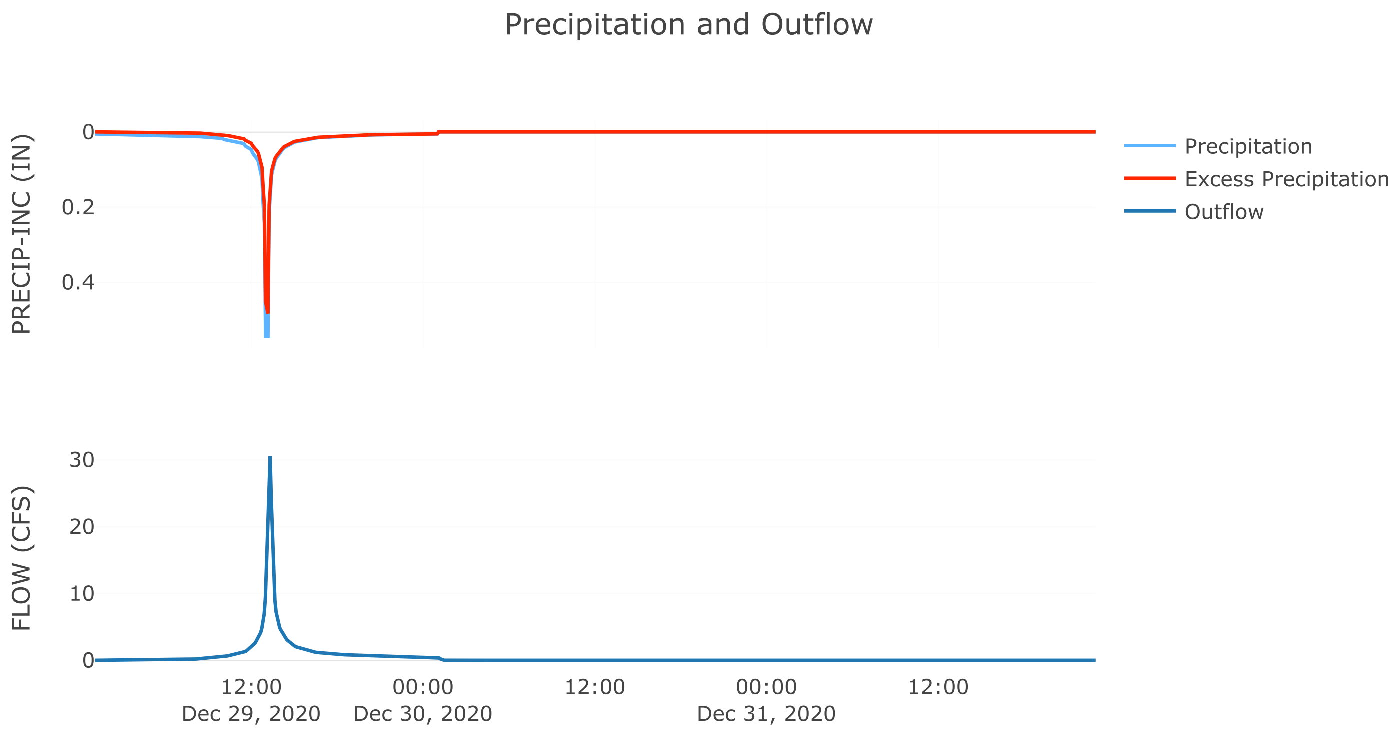
Percent Impervious Area	0
Curve Number	84

Transform: Snyder

Snyder Method	Standard
Snyder Tp	0.14
Snyder Cp	0.64

Results: O4

Peak Discharge (CFS)	30.55
Time of Peak Discharge	29Dec2020, 13:15
Volume (IN)	5.3
Precipitation Volume (AC - FT)	3.75
Loss Volume (AC - FT)	0.98
Excess Volume (AC - FT)	2.77
Direct Runoff Volume (AC - FT)	2.77
Baseflow Volume (AC - FT)	0

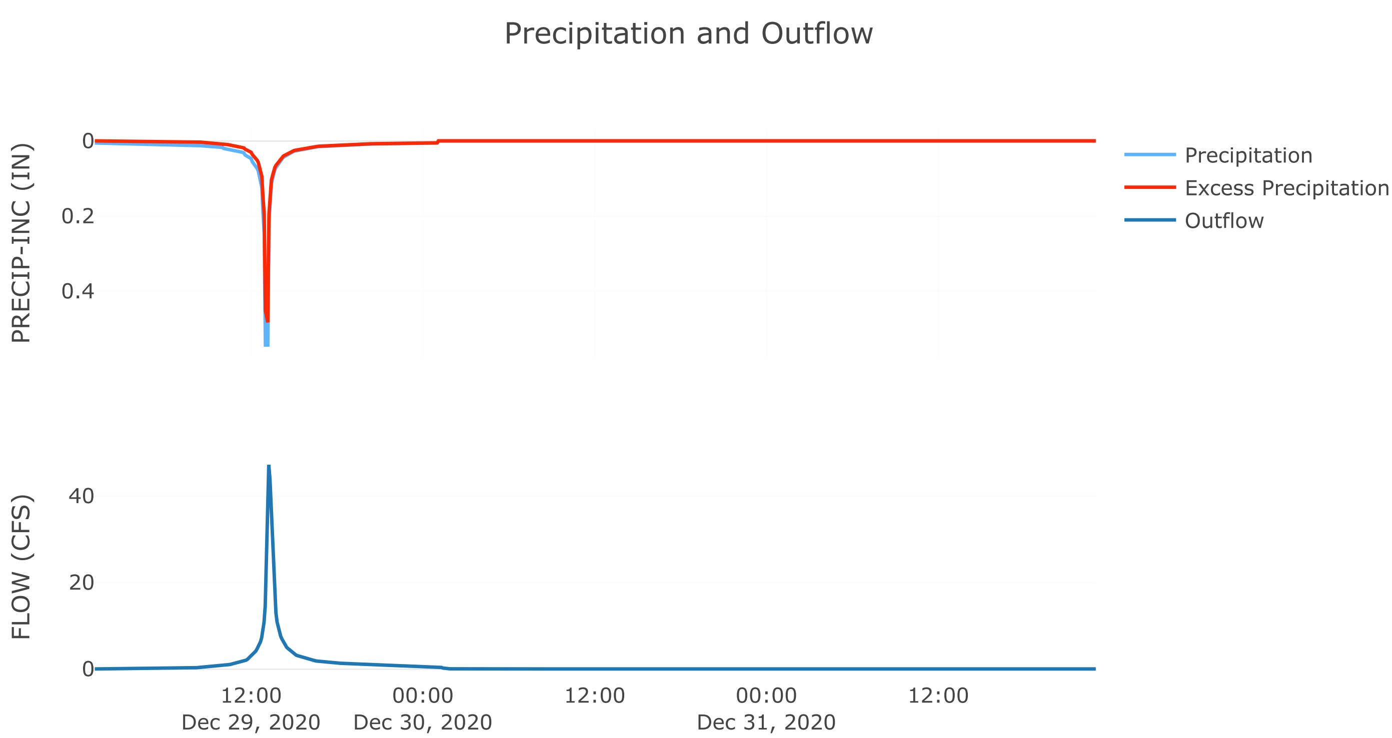

Precipitation and Outflow
PRECIP-INC (IN)
0
0.2
0.4
Precipitation
Excess Precipitation
Outflow
FLOW (CFS)
30
20
10
0
12:00
Dec 29, 2020
00:00
Dec 30, 2020
12:00
00:00
Dec 31, 2020
12:00

Subbasin: CH3

Area (MIē) : 0.01
Downstream : Ch4
Transform : Kinematic Wave

Results: CH3

Peak Discharge (CFS)	47.18
Time of Peak Discharge	29Dec2020, 13:10
Volume (IN)	5.16
Precipitation Volume (AC - FT)	6.08
Loss Volume (AC - FT)	1.58
Excess Volume (AC - FT)	4.5
Direct Runoff Volume (AC - FT)	4.38
Baseflow Volume (AC - FT)	0

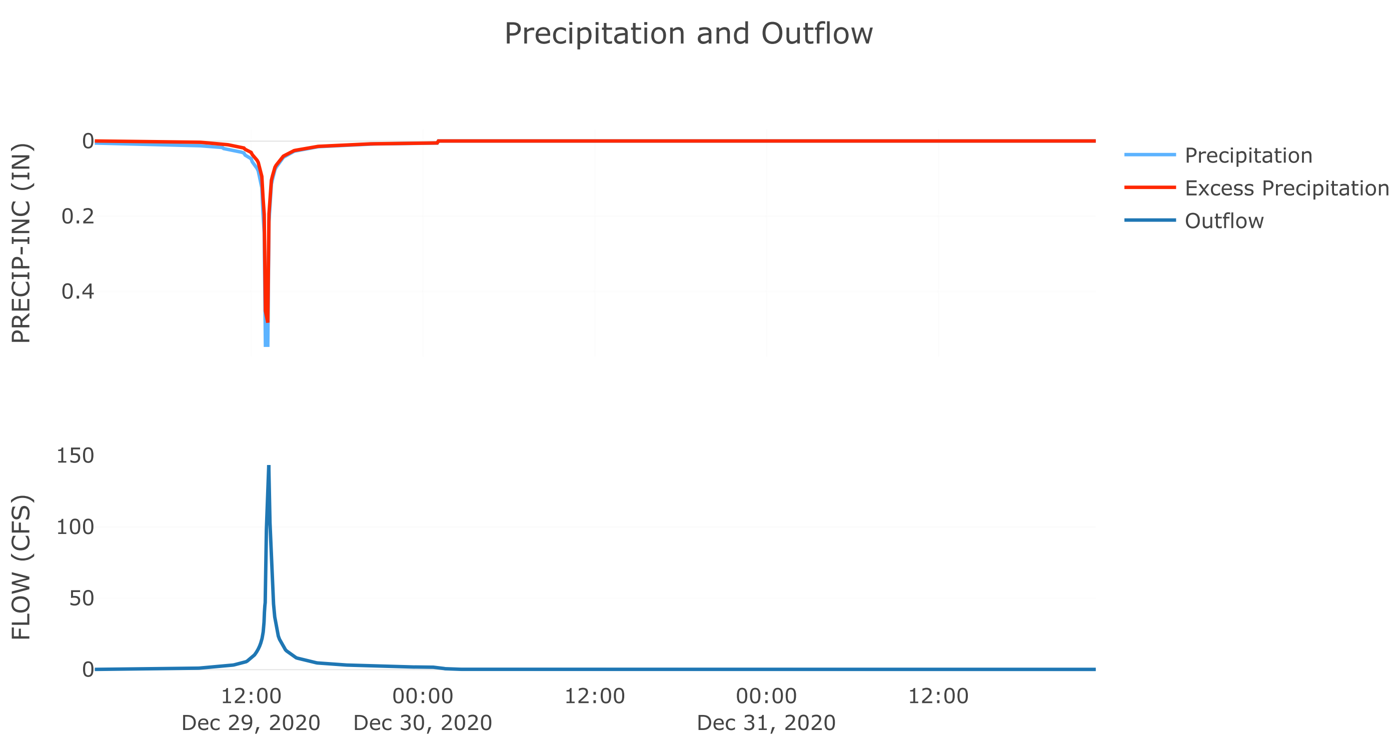


Subbasin: CH4

Area (MIē) : 0.01
Downstream : Ch5
Transform : Kinematic Wave

Results: CH4

Peak Discharge (CFS)	143.14
Time of Peak Discharge	29Dec2020, 13:10
Volume (IN)	5.15
Precipitation Volume (AC - FT)	16.06
Loss Volume (AC - FT)	4.19
Excess Volume (AC - FT)	11.88
Direct Runoff Volume (AC - FT)	11.54
Baseflow Volume (AC - FT)	0

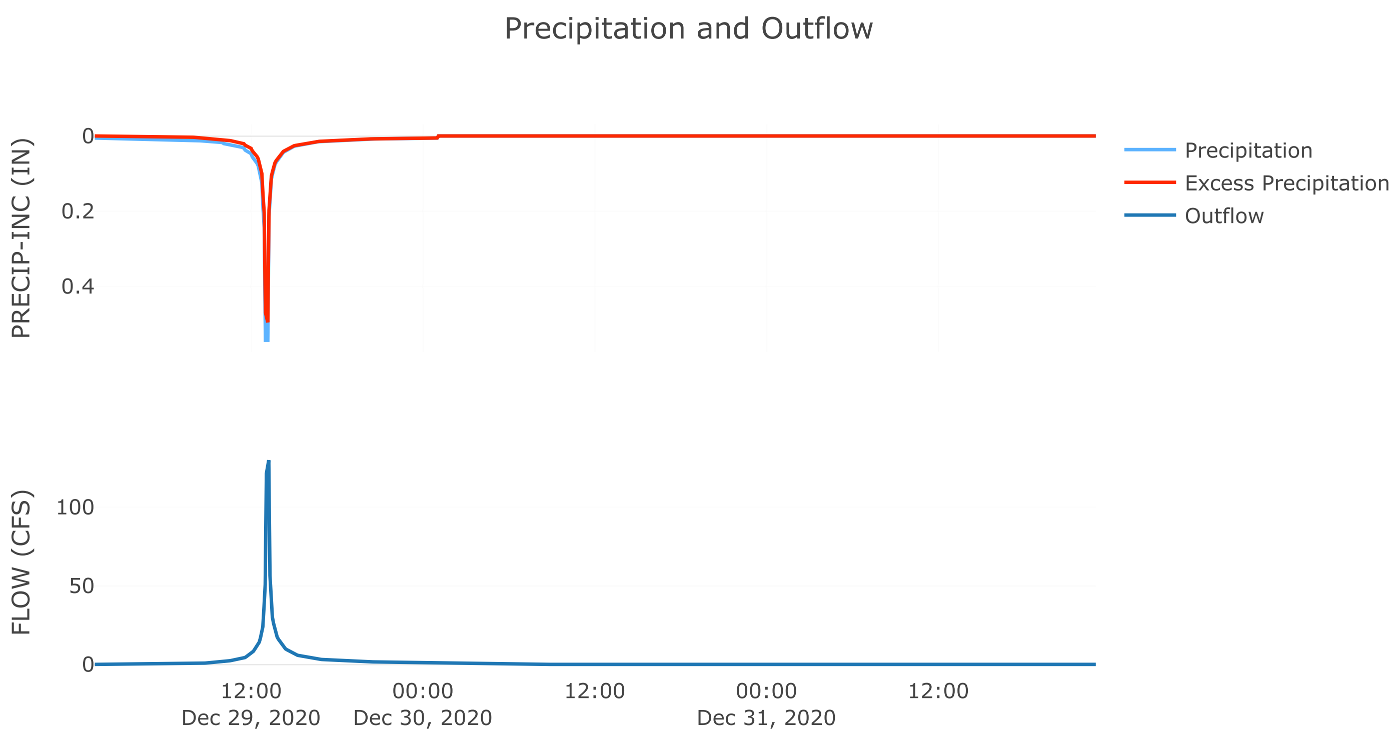


Subbasin: DA1

Area (MIē) : 0.03
Downstream : Ch5
Transform : Kinematic Wave

Results: DA1

Peak Discharge (CFS)	129.86
Time of Peak Discharge	29Dec2020, 13:10
Volume (IN)	5.13
Precipitation Volume (AC - FT)	12.93
Loss Volume (AC - FT)	2.96
Excess Volume (AC - FT)	9.97
Direct Runoff Volume (AC - FT)	9.25
Baseflow Volume (AC - FT)	0

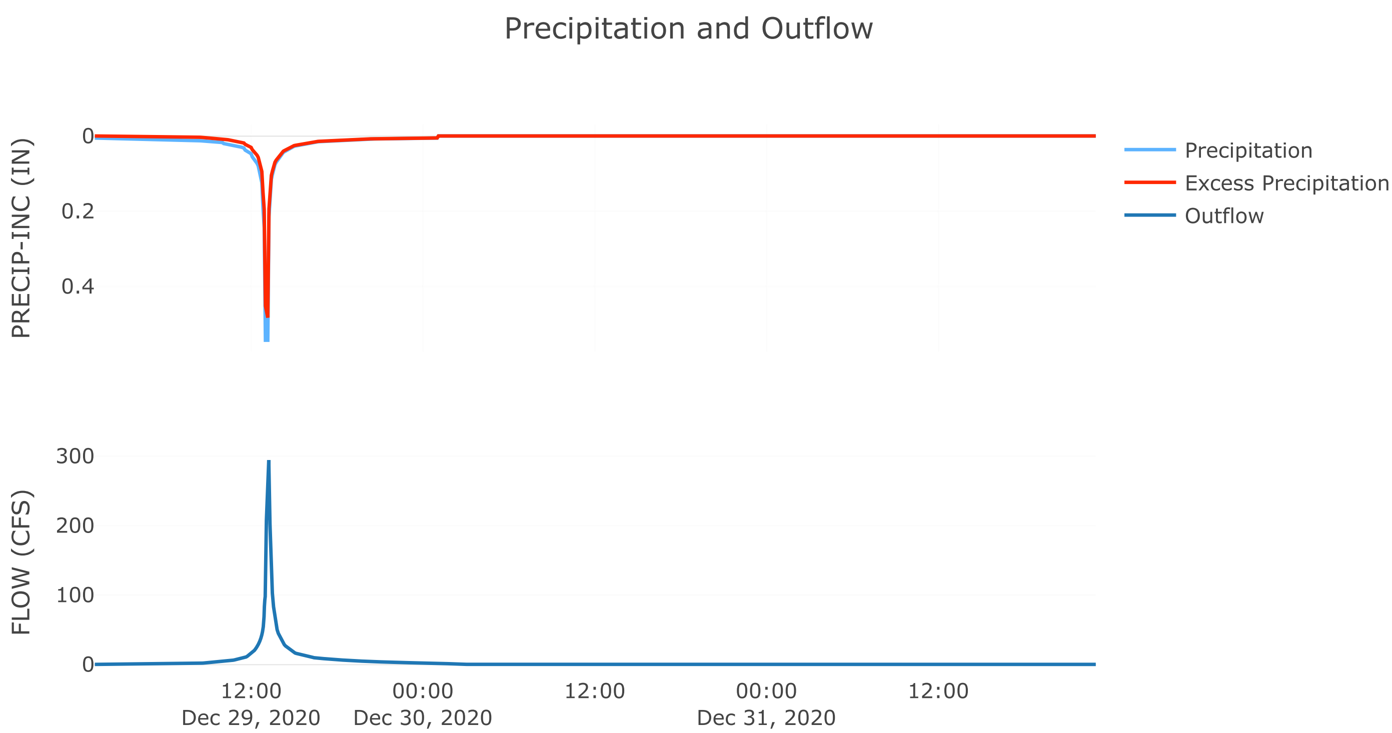


Subbasin: CH5

Area (MIē) : 0.01
Downstream : Cs2
Transform : Kinematic Wave

Results: CH5

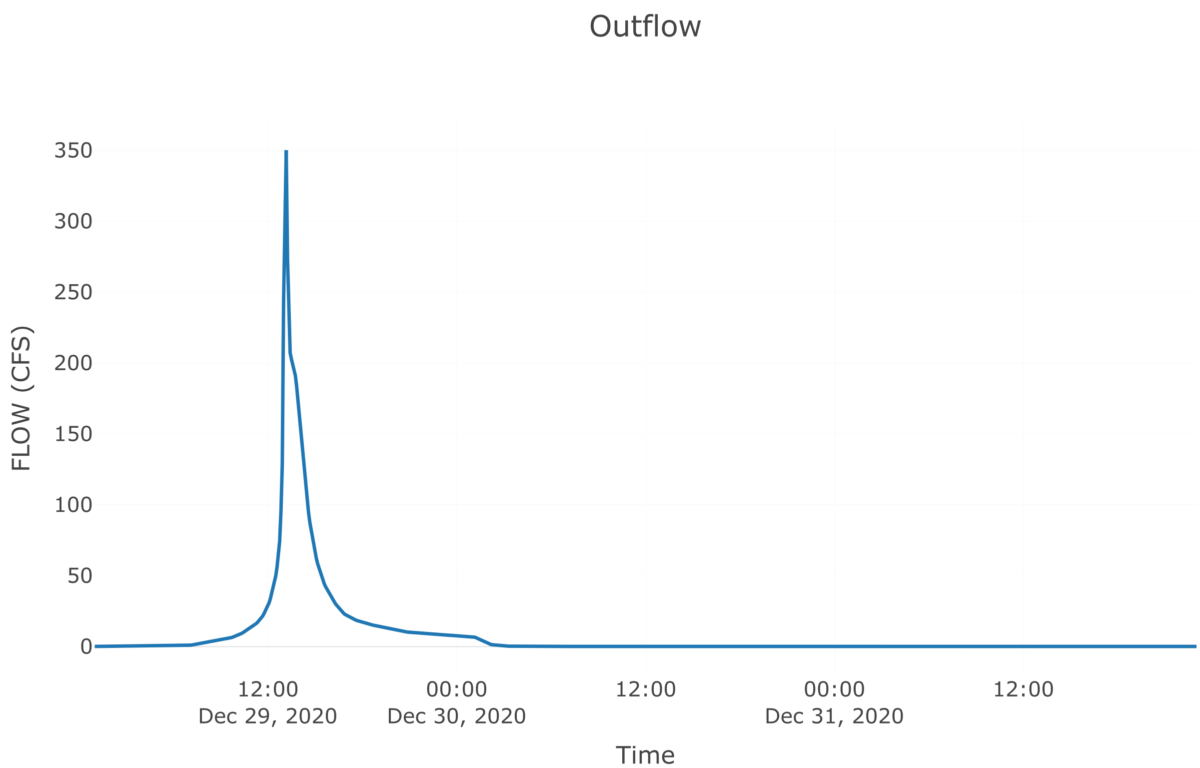
Peak Discharge (CFS)	294.02
Time of Peak Discharge	29Dec2020, 13:10
Volume (IN)	5.14
Precipitation Volume (AC - FT)	31.62
Loss Volume (AC - FT)	8.24
Excess Volume (AC - FT)	23.38
Direct Runoff Volume (AC - FT)	22.69
Baseflow Volume (AC - FT)	0



Junction: CS2

Downstream : Dcp1

Results: CS2	
Peak Discharge (CFS)	350.07
Time of Peak Discharge	29Dec2020, 13:10
Volume (IN)	5.23



Subbasin: P4

Area (MIē) : 0
Downstream : Rcp4

Loss Rate: Scs

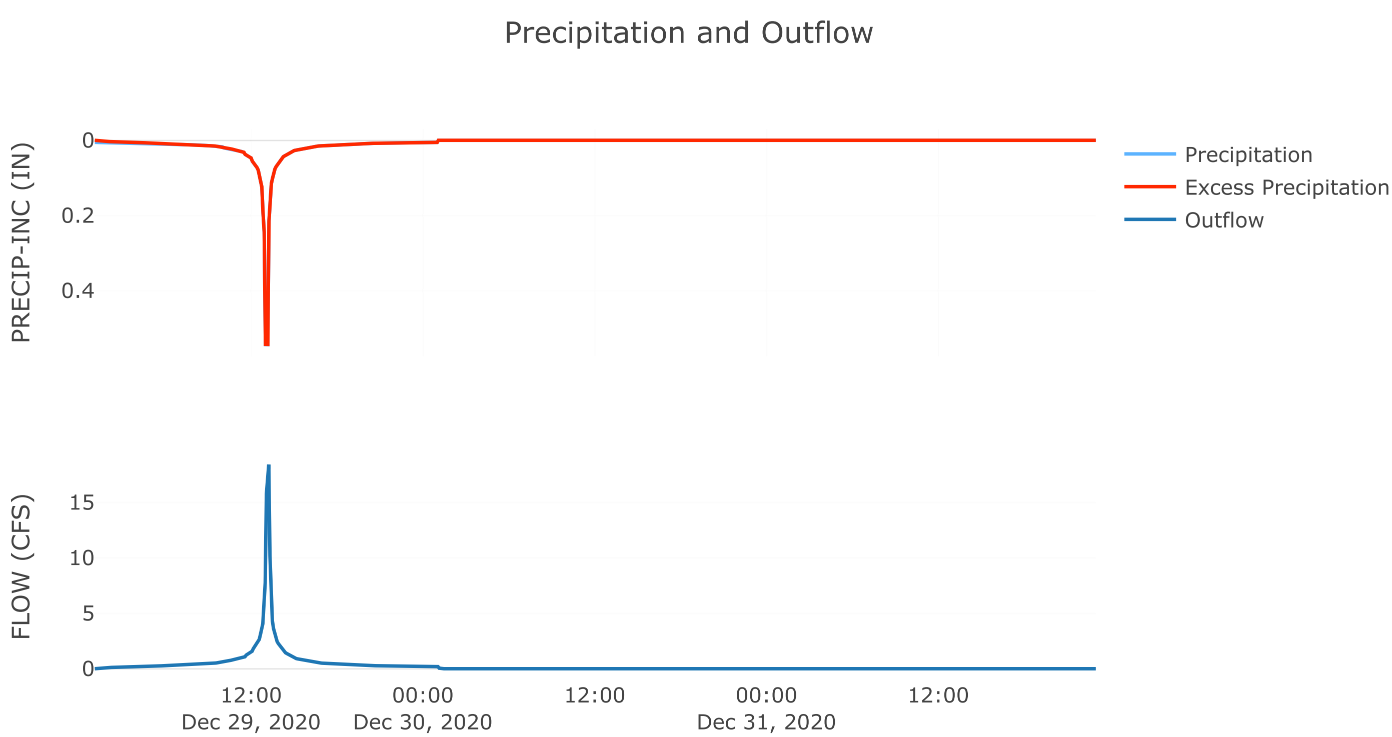
Percent Impervious Area	0
Curve Number	99

Transform: Scs

Lag	0.1
Unitgraph Type	Standard

Results: P4

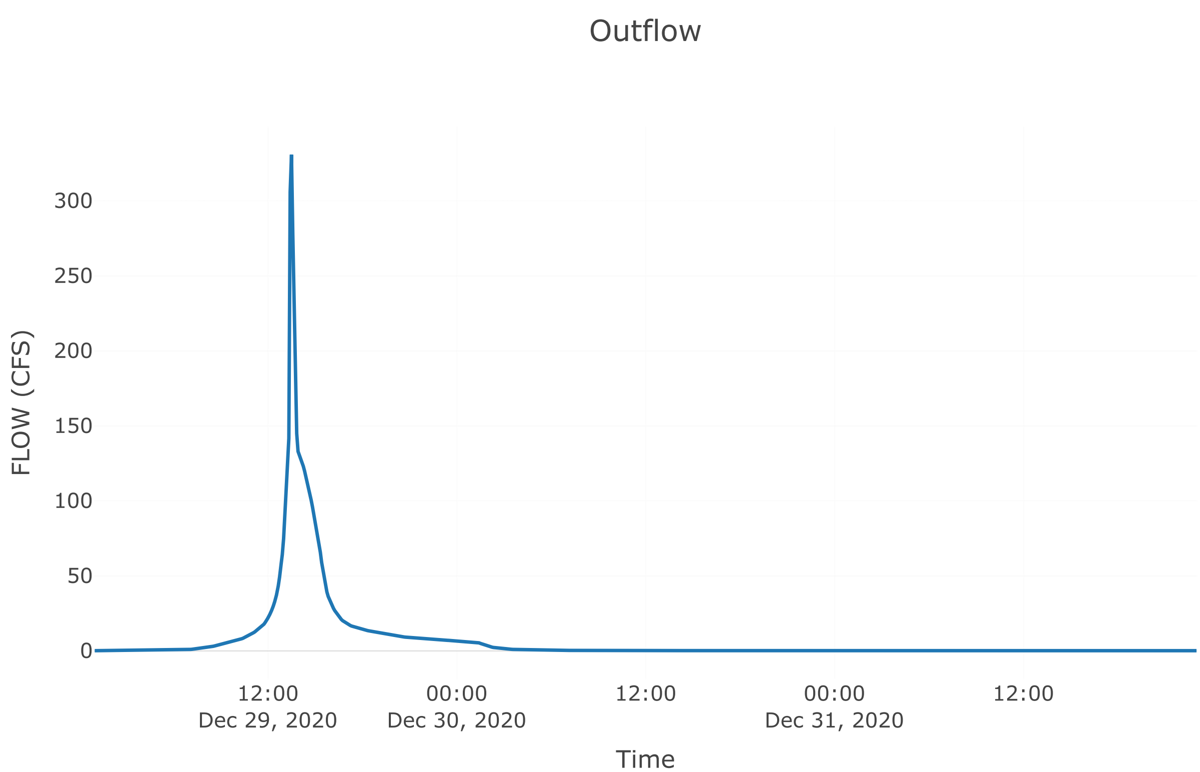
Peak Discharge (CFS)	18.42
Time of Peak Discharge	29Dec2020, 13:10
Volume (IN)	7.05
Precipitation Volume (AC - FT)	1.68
Loss Volume (AC - FT)	0.03
Excess Volume (AC - FT)	1.65
Direct Runoff Volume (AC - FT)	1.65
Baseflow Volume (AC - FT)	0



Reservoir: RCP4

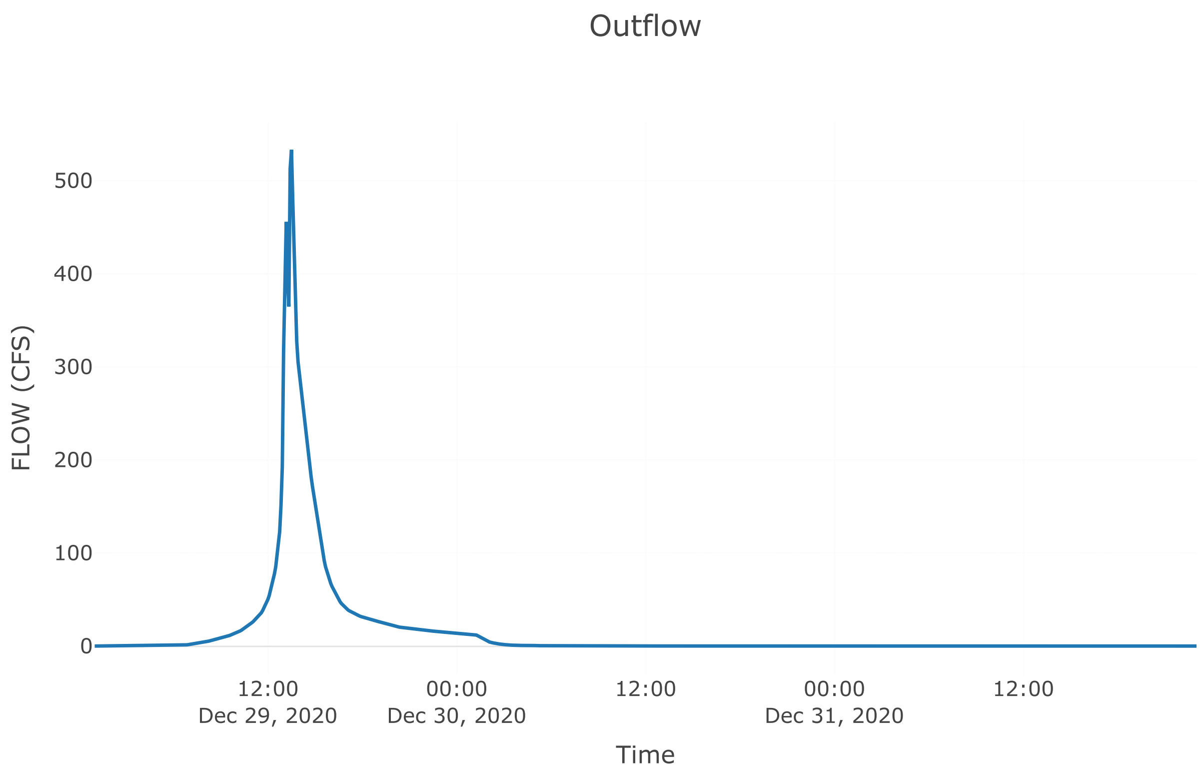
Downstream : Dcp1

Results: RCP4	
Peak Discharge (CFS)	330.63
Time of Peak Discharge	29Dec2020, 13:30
Volume (IN)	5.18
Peak Inflow (CFS)	411.93
Time of Peak Inflow	29Dec2020, 13:15
Inflow Volume (AC - FT)	43.83
Maximum Storage (AC - FT)	10.6
Peak Elevation (FT)	597.94
Discharge Volume (AC - FT)	43.91



Junction: DCP1

Results: DCP1	
Peak Discharge (CFS)	533.14
Time of Peak Discharge	29Dec2020, 13:30
Volume (IN)	5.21



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 on pages IIIF-E-28 through IIIF-E-65.
- Permitted landfill condition volume information is summarized on page IIIF-E-68.

FORT WORTH C&D LANDFILL
0771-356-11-35
EXCESS RAINFALL
VOLUME CALCULATIONS

Required: Determine the volume generated by the site and offsite areas using the excess rainfall calculated in the HEC-HMS analysis of the post-development site conditions.

Method: 1. Use the excessive rainfall data generated by the HEC-HMS analysis (see pages IIIF-E-27 through IIIF-E-65) to determine the volume produced by the site for the post-development conditions.

1. Existing Permit Condition

1. a. Total Flow to Village Creek of Fort Worth C&D northwest of permit boundary (DCP1)

Area No.	Area (sq mi)	Total Excess Rainfall (in)	Area (ac)	Volume (ac-ft)
DA1	**0.0338**	**5.53**	**21.60**	**10.0**
DA2	**0.0183**	**5.53**	**11.70**	**5.4**
DA3	**0.0306**	**5.53**	**19.60**	**9.0**
DA4	**0.0143**	**5.53**	**9.15**	**4.2**
DA5	**0.0248**	**5.53**	**15.90**	**7.3**
S1	**0.0173**	**5.30**	**11.05**	**4.9**
S2	**0.0985**	**5.30**	**63.06**	**27.9**
S3	**0.0022**	**5.30**	**1.44**	**0.6**
CH1	**0.0049**	**5.30**	**3.16**	**1.4**
CH2	**0.0095**	**5.30**	**6.07**	**2.7**
CH3	**0.0061**	**5.30**	**3.87**	**1.7**
CH4	**0.0078**	**5.30**	**5.00**	**2.2**
CH5	**0.0069**	**5.30**	**4.40**	**1.9**
P1-P3	**0.0086**	**7.05**	**5.53**	**3.2**
P4	**0.0044**	**7.05**	**2.82**	**1.7**
O1	**0.0080**	**5.30**	**5.11**	**2.3**
O2	**0.0071**	**5.30**	**4.56**	**2.0**
O3	**0.0272**	**5.30**	**17.39**	**7.7**
O4	**0.0098**	**5.30**	**6.25**	**2.8**

Total Volume of flow discharging from the Permit Boundary to Village Creek (refer to Figure 4.4 in the Drainage Report for the location) = 98.9 ac-ft

VELOCITY CALCULATIONS

FORT WORTH C&D LANDFILL
0771-356-11-35
VELOCITY CALCULATIONS

Required: Determine the flow velocities entering and exiting the permit boundary using HYDROCALC HYDRAULICS (Version 2.01, 1996-2010) for the flows calculated for the 25-year and 25- year storm event in the HEC-HMS analysis.

Method:

1. Use the flow data generated by the HEC-HMS analysis to determine velocity of runoff entering the landfill permit boundary.
2. Use the flow data generated by the HEC-HMS analysis to determine velocity of runoff exiting the landfill permit boundary.

1. Flow Velocity entering the landfill permit boundary

O1

- Flows were obtained from the HEC-HMS files included in this Appendix and are summarized below.

Q_{25} = 21.5 cfs

Storm Year	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)
25	21.5	0.0310	0.04	20.00	20.00	25.00	0.28	2.51

Note: Calculations were performed using the HYDROCALC HYDRAULICS for Windows program developed by Dodson and Associates (Version 2.01, 1996-2010)

O2

- Flows were obtained from the HEC-HMS files included in this Appendix and are summarized below.

Q_{25} = 20.3 cfs

Storm Year	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)
25	20.3	0.0322	0.04	13.00	26.00	25.00	0.27	2.50

Note: Calculations were performed using the HYDROCALC HYDRAULICS for Windows program developed by Dodson and Associates (Version 2.01, 1996-2010)

O3

- Flows were obtained from the HEC-HMS files included in this Appendix and are summarized below.

Q_{25} = 83.7 cfs

Storm Year	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)
25	83.7	0.0838	0.04	2.50	2.50	15.00	0.66	7.60

Note: Calculations were performed using the HYDROCALC HYDRAULICS for Windows program developed by Dodson and Associates (Version 2.01, 1996-2010)

O4

- Flows were obtained from the HEC-HMS files included in this Appendix and are summarized below.

Q_{25} = 30.5 cfs

Storm Year	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)
25	30.5	0.0642	0.04	4.00	4.00	2.00	0.89	6.16

Note: Calculations were performed using the HYDROCALC HYDRAULICS for Windows program developed by Dodson and Associates (Version 2.01, 1996-2010)

2.
Flow Velocity exiting the landfill permit boundary

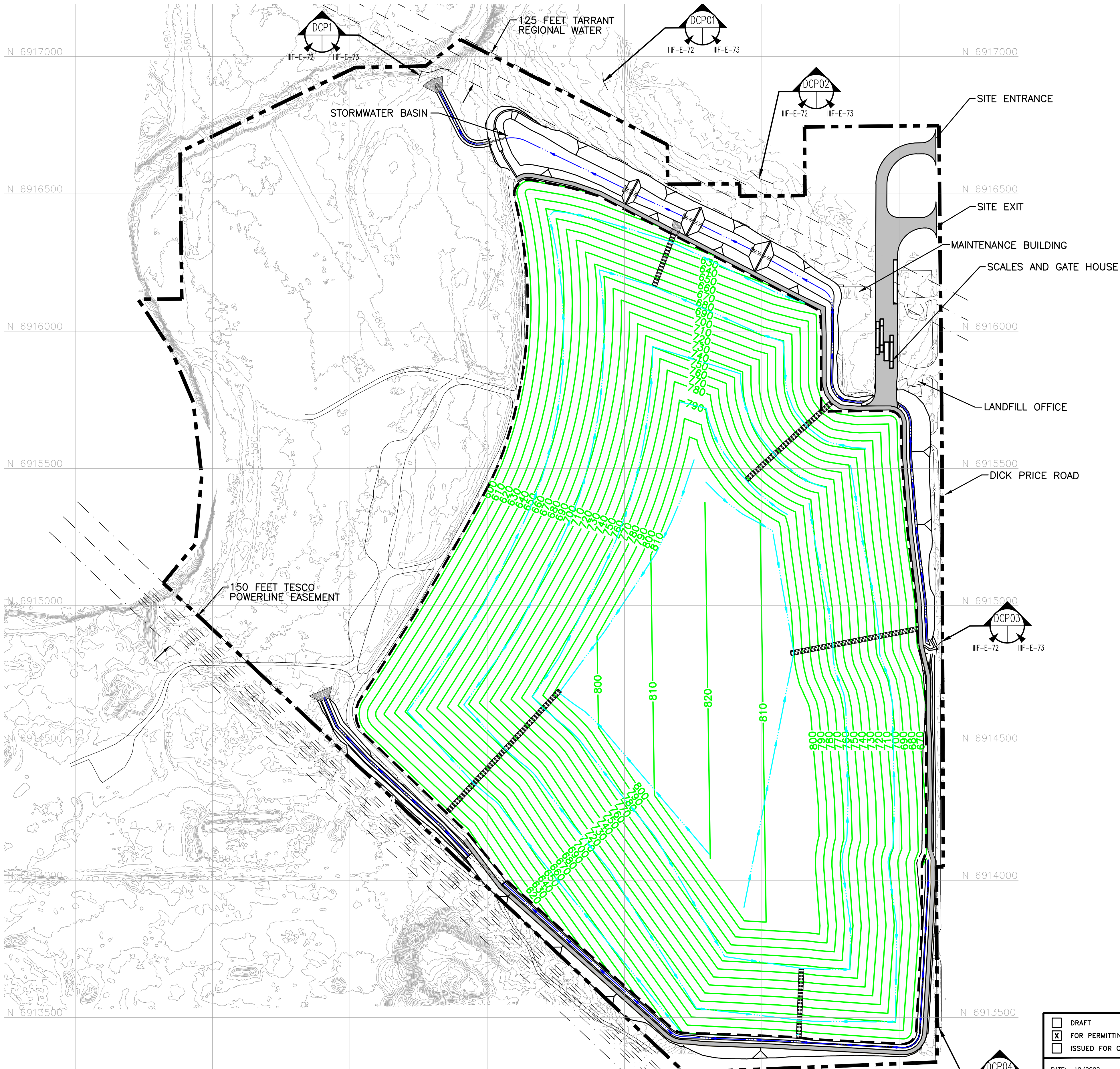
DCP1

- Flows were obtained from the HEC-HMS files included in this Appendix and are summarized below.

$Q_{25} =$ 533.1 cfs

Storm Year	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)
25	533.1	0.008	0.04	4.00	10.00	38.00	2.15	4.67

Note: Calculations were performed using the HYDROCALC HYDRAULICS for Windows program developed by Dodson and Associates (Version 2.01, 1996-2010).



0 200 400

SCALE IN FEET

LEGEND

- PERMIT BOUNDARY
- PERMITTED LIMIT OF WASTE
- 700 PERMITTED FINAL COVER CONTOUR (SEE NOTE 2)
- N 6915000 STATE PLANE COORDINATE
- 580 EXISTING CONTOUR
- DRAINAGE CHUTE
- PERMITTED DRAINAGE SWALE

NOTES:

1. EXISTING CONTOURS AND ELEVATIONS PROVIDED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN 02-17-2022.
2. PERMITTED FINAL COVER GRADES ARE DEVELOPED FROM THE LANDFILL COMPLETION PLAN BY GEOSYNTEC CONSULTANTS, INC., DATED DECEMBER 2020.
3. REFER TO APPENDIX IIIF-SURFACE WATER DRAINAGE PLAN FOR DRAINAGE DESIGN INFORMATION.
4. MAXIMUM FINAL COVER ELEVATION IS 860.0 FT-MSL. MAXIMUM TOP OF WASTE ELEVATION IS 857.5 FT-MSL.

STATE OF TEXAS
CHARLES R. MARSH
105073
LICENSED
PROFESSIONAL ENGINEER
02/09/2023

☐ DRAFT
☒ FOR PERMITTING PURPOSES ONLY
☐ ISSUED FOR CONSTRUCTION

DATE: 12/2022
FILE: 0771-356-11
CAD: IIIF-E-72 POST DEV DRAINAGE.DWG

DRAWN BY: RAA
DESIGN BY: JBM
REVIEWED BY: CRM

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

PREPARED FOR
TEXAS REGIONAL LANDFILL COMPANY, LP

REVISIONS

NO.	DATE	DESCRIPTION

MAJOR PERMIT AMENDMENT
PERMITTED LANDFILL DRAINAGE

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

WWW.WCGRP.COM

DRAWING IIIF-E-72

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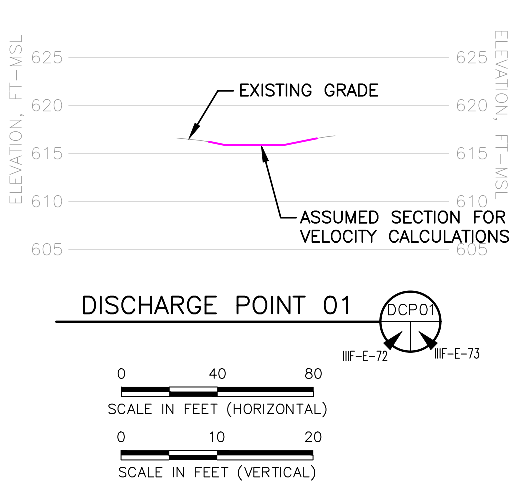

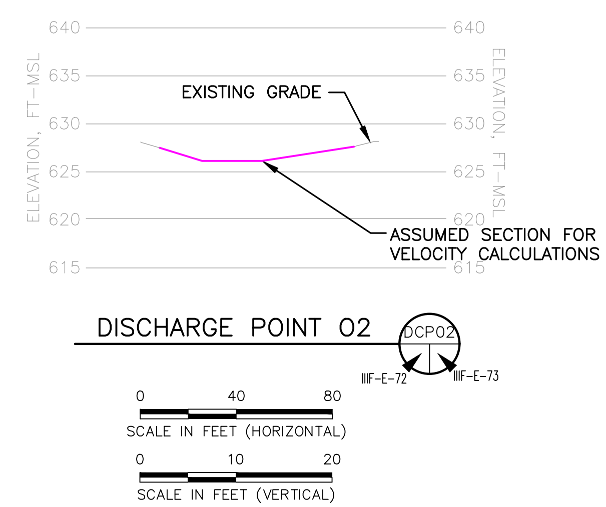

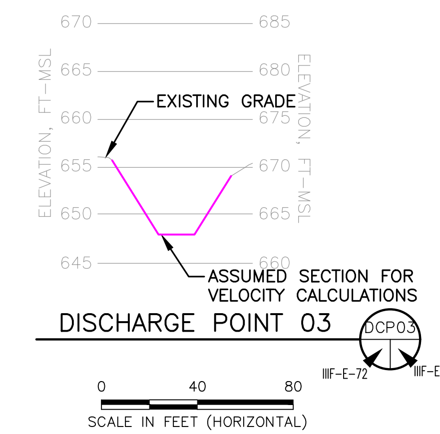

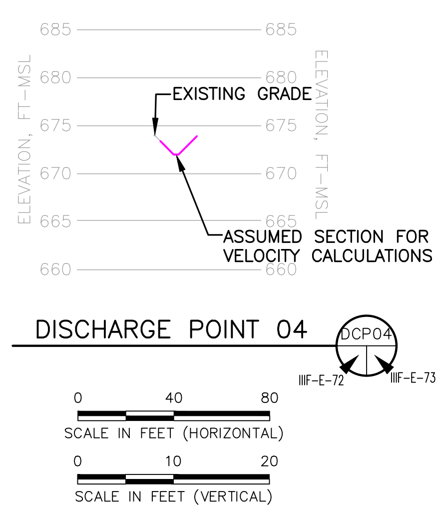

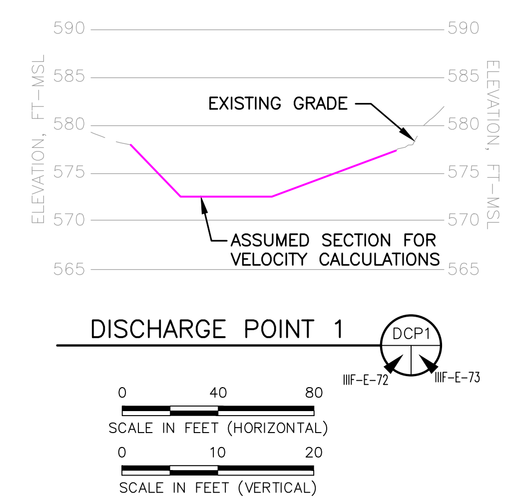

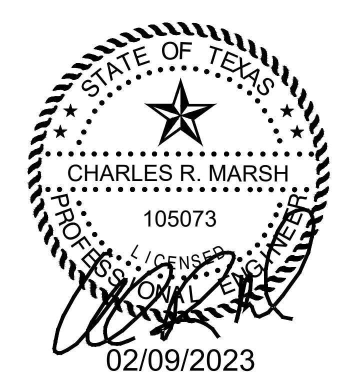

☐ DRAFT
☒ FOR PERMITTING PURPOSES ONLY
☐ ISSUED FOR CONSTRUCTION

DATE: 12/2022
FILE: 0771-356-11
CAD: IIIF-E-73-DISCHARGE POINT SEC.DWG

DRAWN BY: RAA
DESIGN BY: BPY
REVIEWED BY: CRM

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

PREPARED FOR
TEXAS REGIONAL LANDFILL COMPANY, LP

REVISIONS

NO.	DATE	DESCRIPTION

MAJOR PERMIT AMENDMENT
UPDATED PERMITTED DISCHARGE POINT VELOCITY CALCULATIONS

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

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DRAWING IIIF-E-73

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EXISTING PERMITTED DRAINAGE CALCULATION EXCERPTS

ATTACHMENT 2

DRAINAGE REPORT

Prepared for:
Texas Regional Landfill Company, LP

PERMIT AMENDMENT APPLICATION

PART III – SITE DEVELOPMENT PLAN
ATTACHMENT 2

FACILITY SURFACE WATER DRAINAGE REPORT

FORT WORTH C&D LANDFILL
PERMIT NO. MSW-1983D
FORT WORTH, TARRANT COUNTY, TEXAS

Prepared by:

Geosyntec consultants
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STATE OF TEXAS
SCOTT M. GRAVES
86557
LICENSED PROFESSIONAL ENGINEER
12/14/2020

FOR PERMIT PURPOSES ONLY

Submitted May 2020
Revised September 2020
Revised December 2020

TABLE OF CONTENTS

STATE OF TEXAS
SCOTT M. GRAVES
86557
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12/14/2020
FOR PERMIT PURPOSES ONLY
GEOSYNTEC CONSULTANTS, INC.
TEXAS ENG. FIRM
REGISTRATION NO. F-1182

ATTACHMENTS

STATE OF TEXAS
SCOTT M. GRAVES
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12/14/2020

FOR PERMIT PURPOSES ONLY

GEOSYNTEC CONSULTANTS, INC.
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REGISTRATION NO. F-1182

1. INTRODUCTION

1.1 Purpose

Pursuant to 30 TAC §330.63(c), this Facility Surface Water Drainage Report (Drainage Report) has been developed as part of the permit amendment application for the proposed lateral expansion of the Fort Worth C&D Landfill, Fort Worth, Texas (site). This Drainage Report has been prepared to demonstrate that the facility design complies with the requirements of 30 TAC §330.303, and to address the applicable requirements of 30 TAC Chapter 330, Subchapter G. This Drainage Report and the analyses and computations referenced herein were prepared in a manner consistent with guidance provided in the Texas Commission on Environmental Quality (TCEQ) Surface Water Drainage and Erosional Stability Guidelines for a Municipal Solid Waste Landfill, Regulatory Guidance (RG)-417 (TCEQ, 2018).

The Drainage Report includes a narrative description of the drainage conditions and features at the site under pre-development and post-development conditions, addresses flood control, and is accompanied by engineering design drawings and supporting hydrology and hydraulic structural design calculations for the site's drainage features. Specific objectives of this Drainage Report are to:

- present an overview of the project, site watershed setting, and information on the site in relation to the 100-year floodplain;
- describe the currently permitted (Permit MSW-1983C) site conditions and establish the pre-development drainage conditions;
- summarize the proposed post-development surface water management system design and describe the drainage features and components within the facility area;
- describe the post-development drainage conditions;
- describe the hydrologic method and design parameters applied to estimate peak flow rates and runoff volumes for both the pre-development and post-development drainage conditions;
- compare pre-development versus post-development discharges from the site and provide analyses and discussion to demonstrate that the existing pre-development drainage patterns will not be adversely altered as a result of the proposed landfill expansion;

- describe the hydraulic methods and design parameters applied to design the features and components of the surface water management system, and present the structural design of these facilities;
- present the erosion and sediment control measures, including requirements for surface water inspections and maintenance;
- address protection from 100-year frequency flooding; and
- present overall conclusions that summarize the results of the surface water drainage analysis and design.

1.2 Project Overview

The Fort Worth C&D Landfill is an existing Type IV Municipal Solid Waste (MSW) Facility located in unincorporated Tarrant County approximately 15 miles south of downtown Fort Worth. The facility is located approximately 2.4 miles south of interstate highway IH-20 and 5 miles east of IH-35W, on Dick Price Road. Location maps are presented elsewhere in the permit amendment application (e.g., Part II, Appendix IIA). The current-permitted facility (Permit No. MSW-1983C) has a permit boundary encompassing approximately 151.7 acres and the waste disposal footprint of the current permitted design is 77.7 acres).

A lateral expansion of the facility is proposed in this permit amendment application (MSW-1983D). As a part of this lateral expansion, the permit boundary is proposed to increase to approximately 184.35 acres and the waste disposal footprint is proposed to increase to 100.3 acres. The remaining acreage not designated for waste disposal will be utilized for buffer zones, entrance facilities (entrance/exit road, scales and scale house/office area), perimeter access roads, surface water drainage features, groundwater monitoring wells, and landfill gas monitoring and control systems.

A series of engineering drawings are presented in Attachment 2A of this Drainage Report to present the proposed surface water management system design and associated drainage features. Drawing 2-1 in Attachment 2A introduces the proposed facility drainage design, by presenting the "Facility Surface Water Management Plan" and shows the location of the landfill and identifies the associated drainage facilities and features.

1.3 Site Setting and Watershed Information

Regionally, the site is in southern Tarrant County, within the Lower West Fork Trinity watershed of the Trinity River Basin. The site is part of the Village Creek watershed (which has a watershed area of about 122,500 acres) and more specifically, the Village Creek-Lake Arlington sub-watershed (which has a watershed area of about 24,000 acres).

The United States Geological Survey (USGS) developed a system of Hydrologic Unit Codes (HUC) which are arranged or nested within each other from the largest geographic area (regions) to the smallest geographic area (cataloging units) to identify watersheds; these designations are used to identify the site setting. At the site, Village Creek (HUC-10 No. 1203010204) is located along the western side of the site. Village Creek is a tributary of the West Fork Trinity River and is in the Village Creek-Lake Arlington sub-watershed (HUC-12 No. 120301020403). Village Creek originates southwest of the site and flows in a general northeast direction, and flows into Lake Arlington approximately 2.7 miles north of the site. More specifically, at the site Village Creek flows in a northerly direction on the western side of the site. Village Creek receives surface water from and drains areas in southern portions of Tarrant County as well as portions of the City of Fort Worth, the City of Burleson, the City of Crowley, and the City of Joshua.

Clean (uncontaminated) surface water runoff from the existing facility is managed through drainage terraces, downchute channels, and perimeter channels which route surface water towards Village Creek. The proposed landfill expansion will have similar surface water management features routing surface water in the same general manner off the landfill and through perimeter drainage channels and ultimately, to discharge into Village Creek on the western portion of the site.

1.4 100-Year Floodplain Information

TCEQ rules for the siting of landfills include a location restriction in 30 TAC §330.547, which indicates that no solid waste disposal operations shall be permitted in Federal Emergency Management Agency (FEMA)-defined 100-year floodways; and that new municipal solid waste management units, existing municipal solid waste units, and lateral expansions that are located in 100-year floodplains must meet certain additional requirements. The lateral expansion will meet this location restriction and will not be located in a 100-year floodway, nor will the landfill unit be located in a 100-year floodplain. A demonstration of compliance with this location restriction is provided in Part II of the permit amendment application (see Part II Narrative Report, Section 10.1) as required by 30 TAC §330.61(m)(1). An overview of this information is presented below.

With respect to designated floodplains, the site and adjacent areas are part of FEMA Flood Insurance Rate Map (FIRM) Number 48439C0340K, Panel 340 (September 25, 2009). The published FEMA map has not yet been updated to reflect the presence of a compacted earthen levee that was constructed and now exists at the Fort Worth C&D Landfill to form the western perimeter berm of the landfill. The approximately 10-ft tall earthen levee was part of a FEMA-approved Conditional Letter of Map Revision (CLOMR) [Case No. 91-06-24R, approved November 6, 1991] to remove portions of the site from the 100-year floodplain, to allow landfill development to take place in those areas.

A map showing the resulting FEMA defined 100-year floodplain location in relation to the existing site is presented in Part II (see Drawing IIF-1, in Appendix IIF). Also, the FEMA map and additional backup documentation in Part II, Appendix IIF provide the 100-year flood profile elevations at the site. The conclusions of the floodplain evaluation are as follows:

- The facility's landfill disposal limits are and will remain outside the FEMA 100-year floodway, and the 100-year floodplain.

- The 100-year flood elevations in Village Creek as it crosses the site, using information on the published FEMA map, range from an elevation of about 593 ft above mean sea level (ft, MSL) next to southern portions of the site, to approximately 589 ft, MSL next to northern portions of the site.

- The existing levee on the western side of the landfill was constructed to a minimum elevation of 595 ft, MSL, while the 100-year flood elevations adjacent to this levee range from elevation 592 ft, MSL on the south end of the levee, to 589 ft, MSL on the north end of the levee; thus more than 3-ft of freeboard is provided between the 100-year flood elevation and the limit of waste elevation at the edges of the landfill [also note that adjacent landfill perimeter areas south and north of the levee have perimeter berm elevations greater than elevation 595 ft, MSL].

- The limit of fill construction of not just the landfill itself, but also the ancillary landfill-related earthen fill features proposed by this permit amendment application (e.g., the landfill perimeter berms, the surface water pond berms), are outside of the 100-year floodplain.

Additional information on protection of the facility from flooding (including addressing flood protection freeboard at the landfill perimeter) is discussed in Section 7 of this Report, after details of the proposed design and supporting analyses are presented.

2. DESCRIPTION OF THE PRE-DEVELOPMENT CONDITION

From review of United States Geological Survey (USGS) maps showing the topography of the natural conditions of the site prior to development/disturbance activities (e.g., Part II, Appendix IIA, Drawing IIA-2), the conditions before the landfill existed can be described as rolling on the east side of the site (with slopes generally less than 10%), transitioning into a flat river valley of Village Creek on the west side of the site. The pre-landfill natural ground elevations of the site ranged from approximately an elevation of 700 ft, MSL in the southern part of the site, to around an elevation of 580 ft, MSL at the downstream side of Village Creek in the western portion of the site. As mentioned previously, the existing landfill has been largely developed, which has changed the conditions within the current permit boundary to be those permitted rather than the pre-landfill natural conditions.

Therefore, the pre-development drainage areas encompass the current as-permitted (Permit No. MSW-1983C) final landfill conditions at the facility, as well as off-site drainage areas that contribute runoff to the site. This will allow a proper comparison to post-development conditions at the common points-of-interest (the outfalls where surface water exits the site to Village Creek), as discussed later in this report. Specifically, the pre-development conditions are defined as follows:

- within the permit boundary, the pre-development conditions are the current as-permitted (MSW-1983C) final landfill condition; and
- other off-site areas that contribute run-on to the site are delineated using the existing topography of those conditions.

The pre-development conditions and resulting drainage areas are delineated on Drawing 2-2, presented in Attachment 2A of this Drainage Report. Inspection of Drawing 2-2 shows that the overall site pre-development drainage area is 207.14 acres. Based on the current-permitted final landfill geometry and the delineation of existing conditions of non-landfill areas of the site, the pre-development model routes runoff to two discharge locations where flows leave the permit boundary: (i) a midpoint site outfall; and (ii) an overall site outfall.

Runoff from southern areas of the site are routed under pre-development conditions via perimeter drainage channels to a South Surface Water Pond. The midpoint site outfall is located on the west side of the site, where runoff from the South Surface Water Pond leaves the permit boundary at Village Creek. The pre-development drainage sub-area to the midpoint site outfall is 95.7 acres. The midpoint site outfall was selected for modeling and evaluation purposes because it represents a location where flow leaves the permit boundary, and thus is a relevant point to include in the model for evaluation of drainage patterns, as well as for comparison of pre-development vs. post-development flows.

The overall site outfall is located at the northwest portion of the site, where all flows generated by the 207.14 acre site drainage area leave the permit boundary in Village Creek. Note that runoff leaving the site from the midpoint site outfall flows northward in Village Creek and re-joins the permit boundary. Accordingly, the overall site outfall includes flow from the midpoint site outfall, as well as contributing flows from northern areas of the site. Near the overall site outfall at the extreme northwest corner of the permit boundary at Village Creek, there is a discharge structure – as currently permitted – composed of a riprap-lined drainage channel conveying flows leaving the North Surface Water Pond into Village Creek. The overall site outfall also includes overland flows from adjacent existing contributing areas that drain towards Village Creek along this part of the site. The overall site outfall was selected for modeling and evaluation purposes because it represents the eventual routing point of all flows generated by the site, where these flows leave the permit boundary in Village Creek. Therefore, it is a relevant point to include in the model for evaluation of drainage patterns, as well as for comparison of pre-development vs. post-development flows.

Drawing 2-2 also indicates the calculated peak flow rate and the volume of runoff discharged from the site due to a 24-year, 24-hour rainfall event under pre-development conditions. A description of the selected hydrologic method and design parameters is presented subsequently in this Drainage Report.

3. PROPOSED SURFACE WATER MANAGEMENT SYSTEM

3.1 General

This section summarizes the proposed surface water management system design and describes the drainage features and components within the landfill facility. The landfill facility will have above and below grade waste filling over lined areas. A series of drawings presenting the liner base (excavation) grades, the site configuration during phased development and waste filling, and the landfill completion plan, are presented in Part II of the permit amendment application (see Drawings IIA-18 through IIA-22). As described below, certain permanent components of the overall site surface water management system will be constructed at certain points during the development sequence, while other components will be installed as portions of the landfill reach final grade or at the time of closure.

As mentioned, specific to this Drainage Report, a series of engineering drawings are presented in Attachment 2A to present the surface water management system design and associated drainage features. Drawing 2-1 in Attachment 2A of this Drainage Report presents the final configuration of the landfill and the related surface water management system features. As shown, the landfill will have overall sideslopes inclined at 3 horizontal to 1 vertical (3H:1V) (i.e., 33%). At the crest of the final cover sideslopes, the final cover grades then continue up at a shallower top-deck grade of five percent (5%), up to a peak (ridgeline) elevation. In this Drainage Report, final cover slope areas with grades of 5% are designated as "top deck areas", and final cover slopes with overall grades of 3H:1V are designated as "sideslope areas".

3.2 Surface Water Management System Components

Various surface water management system components collect and convey surface water from the final cover system to the discharge points (i.e., outfalls) from the site, as described below. The sizing and hydraulic design of these features is described later in this Drainage Report, in Section 5 (which references detailed calculation packages presented as attachments included with this Drainage Report).

Drainage Terraces and Downchutes. Sideslope drainage terraces installed as "tack-on" berms on the final cover sideslope will intercept surface water runoff (i.e., sheet flow) along the up-gradient sideslope areas of the final cover, and convey runoff to downchute channels. Similar drainage terraces will be constructed at the crest of the landfill sideslope, or the base of the top deck of the final cover, to collect and convey sheet flow runoff from the 5% slope top deck surfaces to the downchute channels. Trapezoidal shaped downchute channels oriented essentially perpendicular to the landfill slopes (i.e., down-slope) will collect the runoff from the top deck and sideslopes and convey the surface water to the landfill perimeter. These downchute channels will be lined with an articulated concrete block (ACB) material, or equal, to resist hydraulic forces from the water flowing in these channels.

Exterior Perimeter Channel. The northern and eastern sides of the landfill include existing perimeter channels to convey runoff from drainage terraces and downchutes, and any contributing sheet flow from adjacent areas, around the landfill and into the North Surface Water Pond. The proposed lateral expansion will continue to route runoff from the landfill final cover in this manner, using the same perimeter channels. The perimeter channel at the southern side of the landfill will receive runoff from landfill areas, as well as to receive and divert "run-on" from adjacent off-site areas around the landfill.

Culvert. There is one proposed culvert pipe at the facility, referred to as "Culvert 1". This culvert is a box culvert at the location where an access road crosses the existing perimeter channel on the northeast side of the site (near the scale area).

Surface Water Pond. (see Drawing 2-1): The proposed final conditions will include modifications to enlarge the existing North Surface Water Pond and create a series of upstream cascading connected sub-pond areas. It is noted that the term "surface water pond" is used because the pond is intended to provide a detention function (controlling the rate of surface water release from the site), as well as provide a sediment control/water quality function. Modifications to the North Surface Water Pond will allow this pond to maintain post-development discharge flow rates at or below pre-development discharge flow rates for the 25-year, 24-hour duration precipitation event.

At the western end of the perimeter channel where runoff enters the surface water pond, a grouted riprap apron will be used for erosion protection. The North Surface Water Pond outlet structure will consist of a pipe located at the pond bottom and an emergency spillway weir/channel located near the top of the pond berm. The geometry and appurtenances of the North Surface Water Pond will detain and release the surface water runoff at rates equal to or less than the pre-development discharge rates from the site as demonstrated later in this Drainage Report.

Active-Area Surface Water Controls. During ongoing landfill development and prior to final cover installation and closure, the site will utilize temporary diversion berms and contaminated water holding areas to maintain the separation of clean runoff from potentially-contaminated water. Temporary diversion berms will be placed up-gradient from active waste areas (i.e., the working face) to intercept clean runoff and route it around active areas to the surface water management system. Also, containment berms will be used to create holding areas down-gradient from the working face to hold any contaminated water that is generated, and prevent its runoff and discharge from the site. The requirements regarding active-area surface water controls are presented in the Contaminated Water Management Plan (Part IV, Appendix IVA). The calculations for sizing of the active-area surface water controls are presented in this Drainage Report, in Attachment 2F.

Interim Erosion and Sediment Control Measures. Erosion and sediment control is addressed in Section 6 of this Drainage Report. In addition, an Intermediate Cover Erosion and Sediment Control Plan (ICESCP), is provided in Attachment 2G to this Drainage Report and includes a description of the measures to be utilized during interim conditions at the site.

4. DESCRIPTION OF THE POST-DEVELOPMENT CONDITION

The post-development drainage areas will encompass the permit boundary as well as including off-site drainage areas that contribute run-on to the site, as follows:

- Within the permit boundary, the post-development conditions are the final conditions proposed in this permit amendment application that incorporate the proposed landfill and the surface water management features described in Section 3.

- Other off-site areas that contribute run-on to the site are delineated using the existing topography of those conditions, which has not changed from the pre-development conditions described in Section 2.

The post-development conditions and resulting drainage areas are delineated on Drawing 2-3, presented in Attachment 2A of this Drainage Report. The post-development surface water management features at the site and the routing of surface water was discussed in Section 3. Inspection of Drawing 2-3 shows that the post-development drainage area is 207.14 acres (the same area as the pre-development drainage area), and there are two outfalls to Village Creek in the western portion of the site (in the same locations as the pre-development outfalls). The post-development drainage sub-area to the midpoint site outfall is 83.5 acres. As mentioned, the outfall locations are the same as those used to model pre-development conditions because the drainage patterns where runoff leaves the site are not being changed from pre-development conditions. Under post-development conditions the outfalls and the relationship between the midpoint outfall and the overall site outfall are the same as described in Section 2 for pre-development conditions.

Drawing 2-3 also provides the calculated peak flow rate and the volume of runoff discharged from the site for the 25-year, 24-hour rainfall event under post-development conditions. A description of the hydrologic method and design parameters is presented subsequently in this Drainage Report. Also, in Section 5.5.1, comparisons of the pre-development and post-development conditions are discussed.

5. DRAINAGE CALCULATIONS

5.1 General

In accordance with 30 TAC §330.303(a), the surface water management system has been designed to convey the peak discharges from the 25-year, 24-hour rainfall event. Design and analysis calculations are made to demonstrate that post-development peak discharges exiting the facility are less than pre-development flows exiting the facility from the 25-year, 24-hour rainfall event. Calculations have been performed to size the drainage features and to demonstrate that flow velocities and tractive stresses in conveyance components will not cause erosion of the drainage terraces, downchute channels, perimeter channels, culvert outlets, etc. These calculations related to the site surface water management features are presented as additional attachments to the Drainage Report, and are as follows:

- Hydrology calculations (i.e., calculations of peak runoff rates and total runoff volumes for the pre-development conditions and post-development conditions) are presented in Attachment 2B. This attachment also includes the storm routing design through the on-site surface water pond and the resulting hydrology computations associated with the detention capabilities of the North Surface Water Pond.

- Hydraulic calculations for the sizing and the design of the surface water pond appurtenances (i.e., outlet control structures, outlet aprons, and anti-seep collars) are presented in Attachment 2C.

- Hydraulic calculations for the sizing and the design of the drainage terraces and downchute channels are presented in Attachment 2D.

- Hydraulic calculations for the sizing and the design of Culvert 1 and the perimeter drainage channels are presented in Attachment 2E.

- Hydrology and hydraulics calculations for active-area surface water controls are presented in Attachment 2F.

It is also noted that an additional calculation package for predicting soil loss and sizing of interim erosion and sediment controls is presented in Attachment 2G.

5.2 Design Rainfall Event

As indicated above and pursuant to 30 TAC §330.63(c)(1)(D)(i), the 25-year, 24-hour rainfall depth was utilized as the design rainfall event for the surface water management system design. The rainfall depth-duration frequency relationships for Tarrant County were obtained from the Texas Department of Transportation (TxDOT) Hydraulic Design Manual (TxDOT, 2019). A rainfall depth of 7.17 inches was chosen to represent the 25-year, 24-hour rainfall using the latest available "Atlas 14" publication (NOAA, 2018). The design rainfall depths in the hydrologic model were consistent with TxDOT (2019) methods and procedures; however, the design rainfall hyetograph was defined with a SCS Type II distribution in order to be consistent with the method utilized in the previous permit application. This rainfall intensity method for determining rainfall distribution was retained in the hydrologic model for this application for a more conservative approach, as it resulted in higher peak intensity values than the latest TxDOT (2019) Hydraulic Design Manual. Additional information concerning the design rainfall parameters is presented in Attachment 2B to this Drainage Report.

5.3 Hydrologic Model

The U.S. Army Corps of Engineers Hydrologic Engineering Center – Hydrologic Modeling System (HEC-HMS) Version 4.3 computer program was used to model the pre-development conditions and the post-development conditions. HEC-HMS is the successor to and replacement for the HEC-1 program. Modeling was used to calculate surface water runoff volumes, peak flow rates, routing of rainfall event hydrographs through perimeter channels and surface water pond, and runoff discharge quantities. Attachment 2B of this Drainage Report presents detailed drainage calculations, including a detailed discussion of the parameters used in the analyses and results of the hydrologic modeling efforts.

5.4 Hydraulics

Principles of open channel flow using Manning's equation (Chow, 1959) were used to size the perimeter drainage channels, top deck drainage terraces, sideslope drainage terraces, drainage downchute channels, and drainage culverts based on the peak flows derived from the HEC-HMS hydrologic modeling.

Manning's Equation in its general form is expressed as:

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

where: Q = discharge (cfs);

n = manning's roughness coefficient;

A = area of cross-section of flow (ft^2);

P = wetted perimeter (ft);

R = hydraulic radius (ft) = A/P; and

S_0 = longitudinal slope (ft/ft).

The average tractive stress for a given depth of flow in a channel is calculated by:

$$\tau_o = \gamma_w RS$$

where: τ_o = average tractive stress (lb/ft^2);

γ_w = unit weight of water (lb/ft^3);

R = hydraulic radius (ft); and

S = channel slope (ft/ft).

Tractive stresses, as well as flow velocities resulting from peak flows, were calculated to select the type of channel lining that would be necessary to prevent erosion of the drainage features.

Elevation-area relationships were developed for the surface water pond and subsequently inputted into the HEC-HMS model for post-development conditions. The elevation-area relationship is calculated based on the size, depth, and shape of the pond, while the elevation-outflow relationship is calculated based on the configuration of the outflow control structure. The elevation-area relationship describes the volume of storage provided by the surface water pond, which is computed based on the proposed surface water pond geometry.

As mentioned, the computations for sizing surface water management system components are found in the following attachments to this Drainage Report:

- Attachment 2B – Hydrology;
- Attachment 2C – Surface Water Pond Appurtenances Design Calculations;
- Attachment 2D – Drainage Terraces and Downchute Channels; and
- Attachment 2E – Culverts and Perimeter Drainage Channels.

5.5 Calculation Results Summary

5.5.1 Discharge Comparisons

Table 5.5.1-1 summarizes the pre- and post-development peak discharges, total discharge volume, and the time to the peak discharge rate. The pre- and post-development drainage sub-areas contributing to the discharge at the midpoint site outfall are 95.7 acres and 83.5 acres, respectively. The pre- and post-development drainage areas contributing to the discharge at the overall site outfall is 207.14 acres for both scenarios. The pre- and post-development nodal network diagrams on Figure 2B-3 and Figure 2B-4, respectively, in Attachment 2B present the delineation of drainage areas to both the midpoint site outfall and overall site outfall.

TABLE 5.5.1-1

SUMMARY OF PEAK DISCHARGE CONDITIONS AT SITE OUTFALL (PRE- VS. POST-DEVELOPMENT COMPARISON)

LOCATION	ITEM	PRE-DEVELOPMENT CONDITIONS (25-YEAR EVENT)	POST-DEVELOPMENT CONDITIONS (25-YEAR EVENT)
MIDPOINT SITE OUTFALL	PEAK DISCHARGE (CFS)	515.4	515.4
	TOTAL RUNOFF VOLUME (AC-FT)	36.3	33.6
	TIME TO PEAK DISCHARGE (MIN)	2	1
	PEAK VELOCITY (FPS)	10.6	5
OVERALL SITE OUTFALL	PEAK DISCHARGE (CFS)	802.6	797.1
	TOTAL RUNOFF VOLUME (AC-FT)	78.7	82.0
	TIME TO PEAK DISCHARGE (MIN)	5	1
	PEAK VELOCITY (FPS)	9	5

Examination of the calculation results shown above indicates that the predicted peak post-development discharge rates are less than the peak pre-development discharge rates at each site outfall. The computed runoff volumes are similar (within less than 10%) for pre-development and post-development conditions. As shown, the times to peak discharge are also not substantially different between pre- and post-development conditions. Under both pre-development and post-development conditions, runoff exits stormwater channels before travelling a distance overland towards the midpoint and overall site outfalls. Channel outlets under post-development conditions will be equipped with energy dissipators (riprap apron or equivalently-effective concrete dissipation device) to reduce peak velocities to low, non-erodible levels before reaching the site outfalls.

In summary, the proposed outfalls will be in the same locations as the existing outfalls, and surface water runoff under proposed post-development conditions is generally routed towards each outfall in a similar manner to pre-development conditions. The proposed drainage areas and patterns of runoff will be similar to the existing permitted pre-development drainage patterns. The peak discharge rates under post-development conditions are considered beneficial given the importance of reducing runoff during storm events. The results demonstrate that the existing pre-development drainage patterns will not be adversely affected by the proposed lateral expansion.

It is also useful to compare the predicted flows generated by the site to the overall flows in Village Creek. According to data in the FEMA *Flood Insurance Study – Tarrant County, Texas and Incorporated Areas* (Flood Insurance Study Number 48439CV001A-48439CV009A. Revised September 25, 2009), the 25-year storm event flow rate in Village Creek as it crosses the site is approximately 38,700 cfs. From this, the calculated pre-development flow rate contributed by this site is approximately 2.07% of the total flow rate in Village Creek. Similarly, the calculated post-development flow rate contributed by this site represents 2.05% of the total flow rate in Village Creek. Both of these values are small percentages of the total flow in Village Creek (with post-development being slightly smaller).

5.5.2 Surface Water Pond

The North Surface Water Pond was sized to adequately detain and pass the 25-year, 24-hour rainfall event while maintaining at least 0.5 feet of freeboard for the 25-year event, and to hold the 100-year, 24-hour rainfall event without overtopping the berm crest. As previously described, the pond has smaller sub-ponds in series. These sub-ponds are referred to as Series 1, Series 2, Series 3, and Series 4 (from downstream to upstream, respectively). The HEC-HMS model was used to calculate surface water runoff volumes, peak flow rates, peak water surface elevations, and routing of rainfall event hydrographs through the North Surface Water Pond and its series of sub-ponds. The results of this analysis are summarized below in Table 5.5.2-1.

TABLE 5.5.2-1

SURFACE WATER POND WATER LEVELS AND DETENTION CAPACITY

Parameter	25-Year Event				100-Year Event			
	North Pond Series 1	North Pond Series 2	North Pond Series 3	North Pond Series 4	North Pond Series 1	North Pond Series 2	North Pond Series 3	North Pond Series 4
Peak Water Surface Elevation (ft, MSL)	597.9	603.3	611.3	622.6	598.4	603.9	613.6	624.8
Available Freeboard to Pond Crest (ft)	0.6	0.7	2.7	2.4	0.1	0.1	0.4	0.2
Peak Storage per Pond (ac-ft)	10.5	1.2	1.6	1.1	11.3	1.4	2.6	1.8

As shown in the above table, adequate freeboard is provided for the 25-year, 24-hour rainfall event in all North Surface Water Pond Series. Additionally, the surface water volume received in the North Surface Water Pond Series from runoff during the 100-year, 24-hour rainfall event is not expected to overtop the berm crests.

5.5.3 Perimeter Channels

Perimeter channels have been designed to convey the peak flows from the 25-year, 24-hour rainfall event, while maintaining at least 0.5 feet of freeboard. Additionally, perimeter channels were designed with the capacity to convey the 100-year, 24-hour rainfall event without overtopping. Tractive stresses and velocities for peak flows during the 25-year, 24-hour rainfall event have been computed and channel linings have been selected to withstand the predicted tractive stresses. Drawing 2-4, Perimeter Drainage Channel Plans With Stationing, shows the designation and layout of the perimeter drainage channels. Drawings 2-5 and 2-6 present the perimeter drainage channel profiles. A table summarizing channel widths, depths, and slopes is provided on Drawing 2-10, and calculations pertaining to the perimeter drainage channel design are presented in Attachment 2E to this Drainage Report. Table 5.5.3-1 summarizes the peak 25-year, 24-hour and peak 100-year, 24-hour rainfall event design flows in the proposed perimeter channels.

TABLE 5.5.3-1

PERIMETER DRAINAGE CHANNEL RESULTS

Channel Segment Designation	25-Yr Peak Flow Rate (ft^3/s)	25-Yr Peak Flow Depth (ft)	25-Yr Peak Flow Velocity (ft/s)	25-Yr Peak Tractive Stress (lb/ft^2)	Freeboard (ft)	Proposed Channel Lining Material
Perimeter Reach A1	36.00	1.07	5.43	0.21	1.93	Geomembrane
Perimeter Reach A2	196.10	1.80	8.13	0.39	1.20	Geomembrane
Perimeter Reach A3	217.70	0.72	19.90	2.93	0.78	Geomembrane
Perimeter Reach A4	216.90	1.28	14.28	1.32	2.22	Geomembrane
Perimeter Reach A5	405.90	2.34	11.55	0.73	1.16	Geomembrane
Perimeter Reach B1	30.80	1.28	6.24	0.94	0.52	Native Vegetation
Perimeter Reach C1	275.60	2.14	8.93	0.45	1.36	Geomembrane
Perimeter Reach C2	275.20	2.14	8.92	0.45	1.36	Geomembrane
Perimeter Reach C3	465.10	2.77	10.28	0.55	0.73	Geomembrane
Perimeter Reach C4	358.60	2.42	9.73	0.51	1.08	Geomembrane

5.5.4 Drainage Terraces

The top deck and sideslope drainage terrace layout is presented on the Facility Surface Water Management Plan, Drawing 2-1. Details of both the top deck and sideslope drainage terraces are presented on Drawing 2-8, and calculations pertaining to the design of these structures are presented in Attachment 2D to this Drainage Report. Drainage terraces have been designed to convey the peak flows from the 25-year, 24-hour rainfall event, while maintaining a minimum of 0.5 feet of freeboard. Additionally, the drainage terraces have been designed with the capacity to convey the 100-year, 24-hour rainfall event without overtopping. Based on the calculated peak tractive stresses, native vegetation or grass lining was selected as the lining of the channels in order to resist erosion of the channel during a 25-year rainfall event. Table 5.5.4-1 summarizes the peak 25-year, 24-hour design flows for the each of the top deck drainage terraces.

TABLE 5.5.4-1

TOP DECK DRAINAGE TERRACE RESULTS

Terrace Designation	25-Yr Peak Flow Rate (ft^3/s)	25-Yr Peak Flow Depth (ft)	25-Yr Peak Flow Velocity (ft/s)	25-Yr Peak Tractive Stress (lb/ft^2)	Freeboard (ft)	Proposed Channel Lining Material
A-1	66.2	0.97	6.07	0.96	1.03	Native Vegetation
B-1	19.6	0.67	3.75	0.42	1.33	Native Vegetation
C-1	61.9	1.18	3.86	0.36	0.82	Native Vegetation

5.5.5 Downchute Channels

Downchute channels have been designed to convey the peak flows from the 25-year, 24-hour rainfall event, while maintaining a minimum of 0.5 feet of freeboard, and to convey peak flows from the 100-year, 24-hour rainfall event based on the surface water management system layout presented on the Facility Surface Water Management Plan, Drawing 2-1. Details of the downchute channels are presented on Drawings 2-8 and 2-9, and calculations pertaining to the downchute channel designs are presented in Attachment 2D to this Drainage Report. Table 5.5.5-1 summarizes the peak 25-year, 24-hour rainfall event design calculations for each of the downchute channels. Erosion control at the downstream outlet of the downchute channels will consist of aprons that are a continuation of the downchute channel lining (e.g., articulated concrete block (ACB)) as described in Attachment 2D and shown in Drawings 2-8 and 2-9.

TABLE 5.5.5-1

DOWNCHUTE CHANNEL RESULTS

Downchute Designation	25-Yr Peak Flow Rate (ft^3/s)	25-Yr Peak Flow Depth (ft)	25-Yr Peak Flow Velocity (ft/s)	25-Yr Peak Tractive Stress (lb/ft^2)	Freeboard (ft)	Proposed Channel Lining Material
Downchute A	167.70	0.70	17.00	12.48	1.30	ACB 800 [1]
Downchute B	96.00	0.86	16.96	12.44	1.14	ACB 800 [1]
Downchute C	156.20	0.74	17.27	12.79	1.26	ACB 800 [1]
Downchute D	76.60	0.95	16.54	11.98	1.05	ACB 800 [1]
Downchute E	133.40	0.76	17.13	12.63	1.24	ACB 800 [1]

Note: [1] Channel Lock ACB system, or a lining system having equivalent resistance to tractive stress, may be used as the lining material for downchute channels.

5.5.6 Culverts

As mentioned, there is one proposed culvert (Culvert 1, see Drawing 2-1) at the site. The culvert was designed to function during a 25-year, 24-hour rainfall event and the 100-year, 24-hour rainfall event for the post-development design. The hydraulic design of Culvert 1 (box culvert) is presented in Attachment 2E to this Drainage Report. Culvert 1 was evaluated by utilizing the HY-8 Culvert Analysis Program v.7.5 (HY-8) developed by the Federal Highway Administration (FHWA). The performance of the culvert is modeled and assessed based on boundary conditions of the structure, culvert configuration, peak flow criteria, and tailwater levels. The tailwater levels were selected based on the computed water depth in the downstream perimeter channel predicted at the time the Culvert 1 is predicted to experience peak flows for the respective design rainfall events.

Note that as part of the facility design, there are control structures (outlet pipes) for each North Surface Water Pond Series (connection). These pipes have been adequately sized as presented herein, but are not assigned the term “culverts”. The design of the surface water pond series’ outlet pipes is part of the surface water pond design presented in Attachment 2C. This is because the design of the surface water pond and resulting discharge rate is influenced by the outlet pipe to which the surface water runoff is routed into through the pond series. Therefore the surface water pond performance (discharge flows, pond elevations, etc.) was evaluated and designed in tandem with the outlet pipes. Design of the surface water pond appurtenances (which refers to anti-seep collars and riprap aprons at the pond outlets) is also presented in Attachment 2C.

6. EROSION AND SEDIMENT CONTROL

6.1 General

The facility has been designed to minimize soil erosion losses, thereby providing effective erosional stability to top deck surfaces and external embankment side slopes during all phases of landfill operation, closure, and post-closure care. The surface water management system design described in this Drainage Report accomplishes this utilizing properly-sized and designed drainage terraces, downchute channels, perimeter drainage channels, culvert, and surface water pond. These features provide for positive drainage of runoff from the final cover system and surrounding site areas and within acceptable tolerances for stresses that could cause erosion.

Additionally, temporary grassing/stabilization, diversions, and other best management practices (BMPs) will be used to minimize soil erosion and sedimentation during intermediate conditions. These BMPs along with other measures utilized while landfill slopes have intermediate cover are discussed in the Intermediate Cover Erosion and Sediment Control Plan (ICESCP), which is provided in Attachment 2G to this Drainage Report. The rainfall intensity determination method utilized in calculations for the design of the intermediate cover features is consistent with the TxDOT (2019) methods and procedures. As areas of the landfill reach final grade, the final cover system, which includes vegetation and other final long-term surface water management system components located on the sideslopes and the top deck areas, will be installed.

6.2 Soil Loss Minimization

The long-term effects of erosion have been evaluated using the Revised Universal Soil Loss Equation (RUSLE) for the intermediate and final cover surfaces. These analyses are more thoroughly discussed for the intermediate cover and final cover surfaces in Appendix 2G-1 of Attachment 2G and in Attachment 3E of the Site Development Plan, respectfully. When landfill slopes are surfaced with intermediate cover prior to receiving final cover, measures will be taken to minimize soil erosion and loss. These measures are discussed in the ICESCP located in Attachment 2G of this Drainage Report. Surface water conveyance structures have been designed for landfill areas with both intermediate and final cover systems. Flow velocities have been estimated for these conveyance structures to evaluate if erosion controls, other than grassing, are required (e.g., concrete lining, geomembranes, geosynthetic erosion control materials, riprap lining materials, etc.).

6.3 Seeding and Stabilization Activities

Temporary and permanent stabilization will be used during the construction and operation of the facility to minimize soil erosion and sedimentation. Temporary stabilization will be performed as described in the ICESCP (see Attachment 2G).

Permanent stabilization will be performed in conjunction with final cover system construction (for the landfill) and final closure of the facility (for other disturbed areas), as described in the Closure Plan (Part III, Attachment 7). In particular, refer to Section 3.4 of the Final Cover Quality Control Plan (FCQCP) in Part III, Attachment 7B for a description of the permanent stabilization specifications and installation procedures.

6.4. Surface Water Maintenance Plan

6.4.1 General

During site construction activities and site operations, inspection and maintenance of disturbed areas and their surface water management system features will be conducted in accordance with the facility's Texas Pollutant Discharge Elimination System (TPDES) Multi-Sector General storm water permit. Written records of these inspections and maintenance activities will be maintained as required by the TPDES permits, as further discussed in Part IV – Site Operating Plan (SOP), Section 24.

During the post-closure care period for the facility, inspections will be performed as indicated in Section 3 of the Post-Closure Plan located in Attachment 8 to the SDP.

6.4.2 Site Maintenance Activities

In general, the following procedures will be followed when deemed necessary by the inspections performed as part of the TPDES permit and as further discussed in Section 24 of the SOP, to maintain and ensure functionality of the surface water management system and erosion and sedimentation controls:

- Eroded areas or areas with ponding water will be regraded to their original slopes and reseeded or covered with an erosion resistant material. Upgrades to the original design specifications can be considered at this remedial stage depending upon the severity of systems degradation.

- Additional temporary erosion protection and sediment control measures using established BMPs will be implemented (seeding, temporary berms, ditches, silt fences, erosion mat, check dams, silt traps, etc.), as necessary, during operation to

- minimize the amount of erosion and sedimentation. These measures can be removed once the erosion has been stopped and long-term vegetation is established and permanent conveyance structures are in place.

- Piped structures (culvert, pond outlet pipes, etc.) will be kept free of debris to allow flows to achieve the design.

- Vegetated water conveyance areas will be mowed periodically to encourage healthy growth and to maintain design flow capacities and erosion resistance.

- Temporary diversion berms will be constructed up gradient of the active working face to limit surface water run-on to waste operations. The temporary containment berms downslope of working areas, interphase berms, or temporary cell berms in interim areas (as appropriate) will also serve to contain surface water runoff down gradient of active working areas. Any surface water that comes in contact with waste will be handled as contaminated water and kept separate from clean runoff.

- Erosion control structures and drainage features such as the surface water pond will be cleaned periodically (removal of debris and sediment) in order to maintain design capacity. The surface water pond will be cleaned by removing sediment using a backhoe, front-end loader, dozer or other similar equipment. The excavated sediment will be transported to designated areas of the site for spreading and drying (must be surrounded by adequate temporary erosion controls).

- Areas of distressed vegetation will be identified and re-vegetated.

- Damaged or eroded drainage terraces, downchute channels, perimeter channels, and culverts will be repaired.

- Excess silt, weeds and other debris accumulated in drainage channels and other conveyances will be removed to restore their design configuration, followed by re-vegetating the disturbed areas as appropriate.

The decision on whether or not maintenance or repairs of site surface water features are needed and the timing on implementing any remedies will be selected based on the severity of the erosion or damage compared to the disturbance that will be caused by the repair and seasonal factors (weather patterns, growing season, etc.).

7. PROTECTION FROM FLOODING

As described previously in Section 1.4 of this Drainage Report, the landfill is not located within the 100-year floodway, nor is it located within the 100-year floodplain due to the presence of the FEMA-approved and now-existing levee. Additionally, the limit of fill for construction of not just the landfill itself, but also the perimeter landfill-related features requiring fill as proposed by this permit amendment application (e.g., the landfill perimeter berms, the surface water pond berms), is outside of the 100-year floodplain.

The FEMA floodplain map and backup information (Part II, Appendix IIF) show that the 100-year flood profile elevations in Village Creek adjacent to the site range from an elevation of about 593 ft, MSL adjacent to the southern areas of the site, to about 589 ft, MSL adjacent to northern areas of the site. The flood protection levee around the western side of the landfill has now been constructed. Its location can be seen on a floodplain map presented in Part II, Appendix IIF (see Drawing IIF-1). Its location can also be seen on drawings within this Drainage Report (e.g., see Drawing 2-1). In 2008 as part of a permit modification for a 10-ft landfill height increase, the Permit MSW-1983B Site Development Plan was revised and included additional information on the levee layout, design, and floodplain analysis of the post-levee-construction 100-year flood levels. The floodplain analyses and related information documented no increase to the 100-year flood elevations of Village Creek due to the levee. The documentation also demonstrated that the levee construction would not significantly restrict flow of the 100-year flood nor significantly reduce the temporary water storage capacity of the 100-year floodplain of Village Creek. Copies of relevant information from Permit MSW-1983C on the levee layout, cross sections, the approved CLOMR, correspondence, and a map showing the resulting 100-year floodplain location in relation to the existing Fort Worth C&D Landfill, are included in Appendix IIF of this permit application.

The existing levee was constructed to a minimum elevation of 595 ft, MSL. By comparing the levee elevation to the 100-year flood profile elevations reported above, it can be seen that in all cases, more than 3-ft of freeboard is provided between the 100-year flood elevation and the limit of waste elevation at the edges of the landfill. The existing levee will remain in-place for this proposed expansion. The 100-year flood elevations on the FEMA map (Drawing IIF-1) will remain valid and unaffected by this project since there will be no encroachment on the 100-year floodway. As mentioned, the previous levee approvals documented there would be no change in 100-year flood elevations of Village Creek due to the levee, nor would it restrict flow of the 100-year flood or reduce the temporary water storage capacity of the 100-year floodplain. By keeping the floodway free of encroachment, there will be no increase in flood heights (substantial or otherwise) by this project.

8. DRAINAGE FEATURE INSTALLATION SCHEDULE

The phased landfill development configurations are illustrated in Part II of the permit amendment application. Specifically, Part II, Attachment 2A, Drawings IIA-19 through IIA-21 show the sequence of development and the layout of the related features at given points in time. The notes on these drawings provide a description of each phase of development and address the timing of the installation of the final cover drainage features that have been discussed in this Drainage Report. The remainder of this section provides a summary of the installation schedule for drainage features.

Overall, as the landfill is developed, the landfill will have temporary grassing/stabilization, diversions, and other best management practices (BMPs) installed on top deck and external facing slopes in accordance with the Intermediate Cover Erosion and Sediment Control Plan (ICESCP) provided in Attachment 2G. Areas that have achieved final waste grades will have the final cover system installed incrementally; and the final landfill drainage features (i.e., final cover with topsoil and vegetation, drainage terraces, and downchutes) required for those areas being capped will be installed in conjunction with the final cover installation.

With respect to perimeter surface water management system features (perimeter ditches and surface water pond), those final features will be installed prior to when runoff generated by the landfill areas (i.e., above-grade filling) contributes flow towards those perimeter features.

More specifically, from the information presented on the aforementioned phase development plans in Part II of the permit amendment application, it is expected that the final perimeter drainage features will be installed according to following approximate schedule:

- Initial Conditions: Prior to construction of the first phase of the lateral expansion, waste filling of existing sectors will continue in existing (currently constructed) sectors, and the existing perimeter stormwater management features will continue to be used.

- When the interim waste grades of Sector 3C go above-grade (i.e., above the elevation of the adjacent perimeter berm), the North Surface Water Pond modifications will be constructed, and clean storm water runoff will be routed through the northern perimeter storm water channel and North Surface Water Pond, accordingly.

- When Sector 4 (southern lateral expansion area) is constructed, the adjacent perimeter channel that will route storm water around the southern part of the site and towards the west will also be constructed.

- When Sector 5 (northeast lateral expansion area) is constructed, the remainder of the final perimeter storm water features will be constructed (i.e., the perimeter channel adjacent to this sector, along with Culvert 1).

9. CONCLUSION

This Drainage Report has been prepared to demonstrate that the facility design complies with the requirements of 30 TAC §330.303 and to address the applicable requirements of 30 TAC Chapter 330, Subchapter G. The Drainage Report is accompanied by engineering design drawings, supporting hydrology calculations, and hydraulic structural design calculations for the site's drainage features. The following conclusions summarize the results of the drainage analysis and design:

- The drainage design criteria selected meet the requirements of 30 TAC Chapter 330.
- The surface water management system drainage structures (terraces, downchutes, ditches, and culvert) are designed to convey peak flows from the 25-year rainfall event with 0.5 feet of freeboard.
- The surface water pond capacities and outlet structure are designed to manage the 25-year rainfall event and with erosion protection to attenuate the velocity and dissipate the energy at each outfall.
- Erosion will be minimized through the interim and permanent design features and best management practices described herein.
- The post-development discharge rates from the site are less than or equal to the pre-development discharge rates, and the discharge volumes and time-to-peak discharge for the pre- and post-development conditions are similar.
- The calculated 25-year pre-development and post-development flow rates contributed by this site represents about 2.1% of the total flow rate in Village Creek for that storm event. This shows that the site's contribution to the total peak flow in Village Creek is relatively small, and it is approximately equal (slightly smaller) under post-development conditions.
- The landfill is not within the 100-year floodway or 100-year floodplain, nor will waste filling occur in the 100-year floodplain as a part of the expansion of the landfill. The landfill is protected from the 100-year frequency flood event.
- The post-development drainage patterns will be similar to the existing pre-development permitted drainage patterns and will direct surface water runoff to the same outfall locations. The existing pre-development drainage patterns will not be adversely altered.

10. REFERENCES

Chow, V.T (1959). *Open Channel-Hydraulics*, McGraw-Hill.

NOAA (2018). Point Precipitation Frequency Estimates, National Oceanic and Atmospheric Administration, Atlas 14, Volume 11, Version 2.0. Available online: https://hdsc.nws.noaa.gov/hdsc/pfds/, accessed November 2019, site latitude: 32.6326º, longitude: -97.2375º.

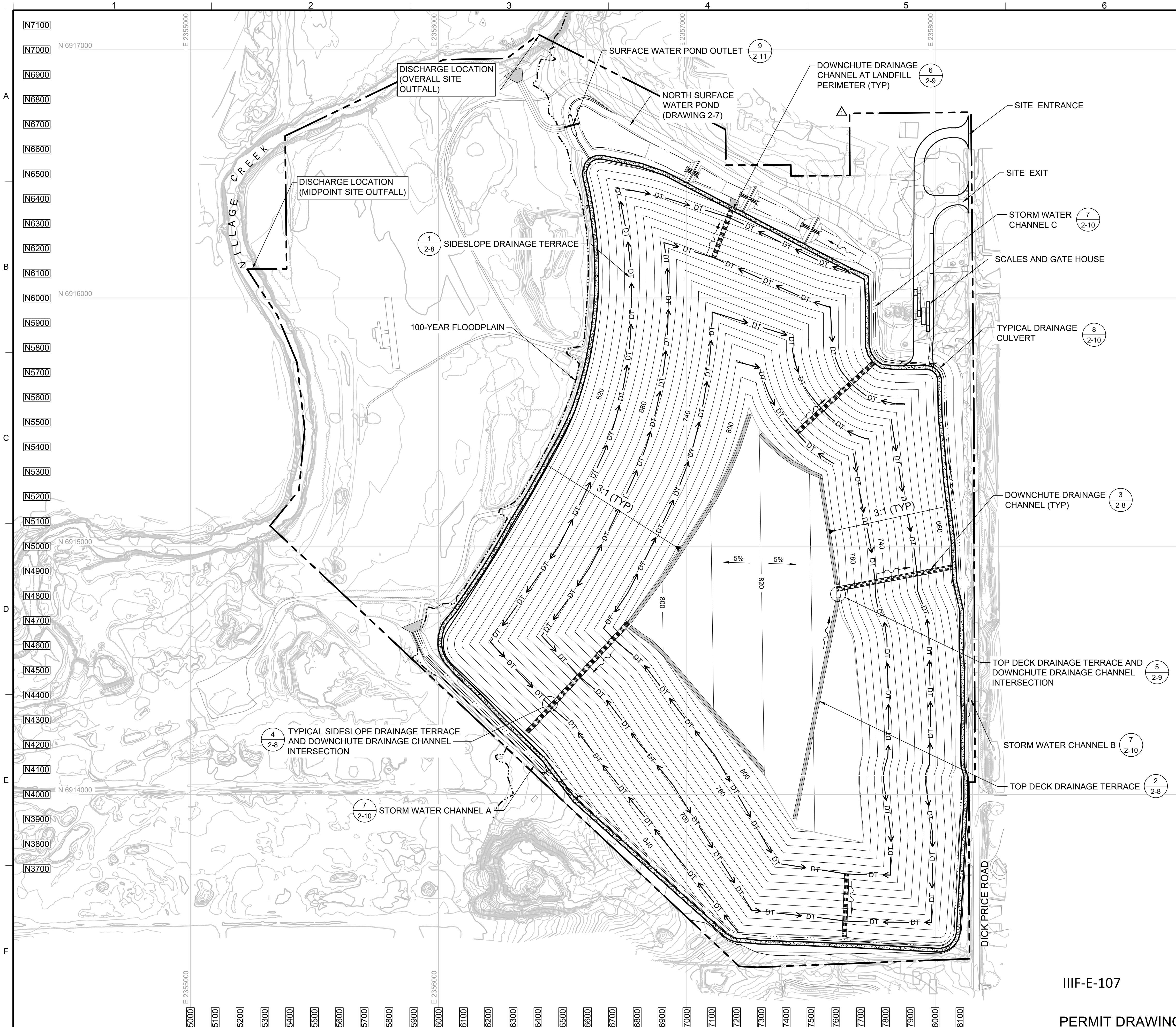
TCEQ (2018). Surface Water Drainage and Erosional Stability Guidelines for a Municipal Solid Waste Landfill, Regulatory Guidance 417 (RG-417), Texas Commission on Environmental Quality, Waste Permits Division, Revised May 2018.

TxDOT (2019). *Hydraulic Design Manual*, Texas Department of Transportation, revised September 2019.

ATTACHMENT 2A

SURFACE WATER MANAGEMENT SYSTEM DRAWINGS

LIST OF DRAWINGS		
Drawing No.	Title	Drawing Date (latest revision)
2-1	Facility Surface Water Management Plan	December 2020
2-2	Pre-Development Plan With Drainage Patterns	May 2020
2-3	Post-Development Plan With Drainage Patterns	December 2020
2-4	Perimeter Drainage Channel Plan With Stationing	May 2020
2-5	Drainage Channel A and B Profiles	May 2020
2-6	Drainage Channel C Profile	May 2020
2-7	North Surface Water Pond	May 2020
2-8	Surface Water Management System Details I	December 2020
2-9	Surface Water Management System Details II	May 2020
2-10	Surface Water Management System Details III	May 2020
2-11	Surface Water Management System Details IV	May 2020



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PERMIT DRAWING

LEGEND

580	EXISTING GROUND ELEVATION CONTOUR (FT, MSL) (NOTE 1)
	EXISTING ROAD
x — x	EXISTING FENCE
	EXISTING BUILDING
	EXISTING WATER LINE
N 6917000 E 2355000	COORDINATE GRID (NOTE 2)
N7100	SITE GRID
	PERMIT BOUNDARY
DT ← DT ←	SIDESLOPE DRAINAGE TERRACE AND FLOW DIRECTION
	DOWNCHUTE DRAINAGE CHANNEL
	TOP DECK DRAINAGE TERRACE
	PERIMETER DRAINAGE CHANNEL
650	PROPOSED GROUND ELEVATION CONTOUR (FT, MSL) (NOTE 3)
	100-YEAR FLOODPLAIN (NOTE 6)

0 200' 400'
SCALE IN FEET

NOTES:

1. THE EXISTING TOPOGRAPHIC BASE MAP WAS COMPILED FROM PHOTOGRAMMETRIC METHODS BASED ON AERIAL PHOTOGRAPHY PERFORMED ON 06 MARCH 2019 BY DALLAS AERIAL SURVEYS, INC.
2. ELEVATIONS ARE IN FEET ABOVE MEAN SEA LEVEL (FT, MSL), AS DEFINED BY THE NORTH AMERICAN VERTICAL DATUM (NAVD) OF 1988. STATE PLANE COORDINATE GRID BASED ON TEXAS STATE PLANE COORDINATE SYSTEM, TEXAS NORTH CENTRAL ZONE (4202), NORTH AMERICAN DATUM OF 1983 (NAD-83).
3. PROPOSED FINISHED GRADE WITHIN THE LIMIT OF FINAL COVER REFERS TO TOP OF FINAL COVER SYSTEM (TOP OF THE TOPSOIL COMPONENT OF THE FINAL COVER SYSTEM). LIMIT OF FINAL COVER SYSTEM REFERS TO THE TOE OF SLOPE OF THE TOPSOIL LAYER. OUTSIDE OF THE LIMIT OF FINAL COVER, THE PROPOSED CONTOURS REFER TO FINISHED GRADE.
4. INFORMATION ON SURFACE WATER POND APPURTENANCES IS PROVIDED ON DRAWINGS 2-8 AND 2-9.
5. 100-YEAR FLOODPLAIN LIMITS ARE FROM FEMA MAPS AND APPROVED CONDITIONAL LETTER OF MAP REVISION (CLOMR), AS DESCRIBED IN PART II, NARRATIVE REPORT, SECTIONS 10.1.2 AND 10.1.3 AND SHOWN IN DRAWING IIF-1.

STATE OF TEXAS
SCOTT M. GRAVES
86557
LICENSED PROFESSIONAL ENGINEER

12/14/2020

FOR PERMIT PURPOSES ONLY

REV	DATE	DESCRIPTION	DRN	APP
1	DEC 2020	RESPONSE TO NOD 2	JJV	SMG
-	MAY 2020	INITIAL SUBMITTAL TO TCEQ	JJV	SMG

WASTE CONNECTIONS, INC.
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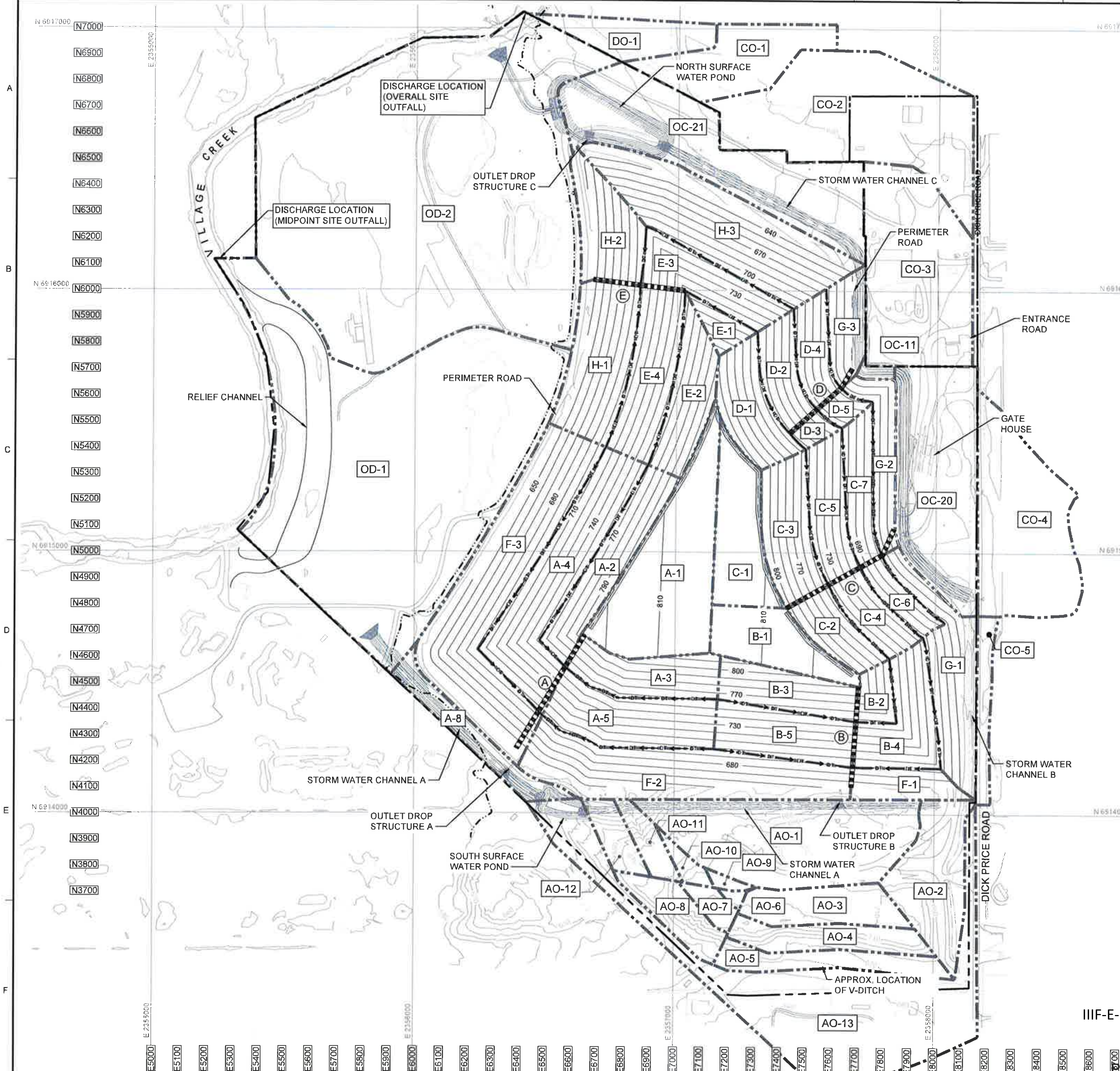
TEXAS ENG. FIRM REGISTRATION NO. 1182
8217 SHOAL CREEK BLVD, SUITE 200
AUSTIN, TEXAS 78757
PHONE: 512.451.4003

TITLE:
FACILITY SURFACE WATER MANAGEMENT PLAN

PROJECT:
FORT WORTH C&D LANDFILL EXPANSION
PERMIT AMENDMENT APPLICATION - PERMIT NO. MSW-1983D

PROJECT NO.: GW6953	DESIGN BY: BK	REVIEWED BY: BG	PART NO.: III	DRAWING: 2-1
FILE: GW6953P2-1	DRAWN BY: JJV / KH	APPROVED BY: SMG		

DRAWING: Austin P:\CADD\Projects\Ft worth c&d landfill\PERMIT\landfill expansion (gw6953)\DRAWINGS\GW6953P2-1.dwg PLOTTED: Dec 07, 2020 - 2:33pm



PRE-DEVELOPMENT SUMMARY			
DRAINAGE BASIN DESIGNATION	DRAINAGE AREA (ACRES)	25-YEAR PEAK FLOW RATE (CFS)	25-YEAR RUNOFF VOLUME (AC-FT)
AO-1	6.69	41.80	2.16
AO-10	0.35	2.20	0.11
AO-11	0.73	4.60	0.24
AO-12	0.85	5.30	0.27
AO-13	10.25	59.90	3.31
AO-2	3.09	19.40	1.00
AO-3	1.96	12.30	0.63
AO-4	1.74	10.90	0.56
AO-5	1.60	10.00	0.52
AO-6	0.11	0.70	0.04
AO-7	0.46	2.90	0.15
AO-8	1.38	8.70	0.45
AO-9	0.17	1.10	0.06
A-1	5.63	42.70	2.54
A-2	2.41	20.20	1.09
A-3	2.11	17.70	0.95
A-4	4.50	37.60	2.03
A-5	2.90	24.30	1.31
A-8	1.52	12.70	0.69
B-1	2.21	16.80	1.00
B-2	0.67	5.60	0.30
B-3	1.86	15.60	0.84
B-4	2.26	18.90	1.02
B-5	2.26	18.90	1.02
CO-1	3.02	17.30	0.97
CO-2	8.44	45.80	2.72
CO-3	6.64	49.00	2.81
CO-4	5.73	34.90	1.85
CO-5	0.94	5.90	0.30
C-1	2.67	21.40	1.21
C-2	1.31	11.00	0.59
C-3	2.29	19.20	1.03
C-4	1.21	10.10	0.55
C-5	1.75	14.60	0.79
C-6	0.77	6.40	0.35
C-7	1.39	11.60	0.63
DO-1	1.74	9.90	0.56
D-1	2.04	17.10	0.92
D-2	1.35	11.30	0.61
D-3	0.34	2.80	0.15
D-4	1.16	9.70	0.52
D-5	0.37	3.10	0.17
E-1	0.55	4.60	0.25
E-2	2.09	17.50	0.95
E-3	2.05	17.10	0.93
E-4	2.63	22.00	1.19
F-1	1.11	9.30	0.50
F-2	5.35	44.80	2.42
F-3	7.29	60.50	3.29
G-1	3.04	25.40	1.37
G-2	1.47	12.30	0.66
G-3	1.25	10.40	0.56
H-1	3.22	26.90	1.45
H-2	2.17	18.10	0.98
H-3	5.82	48.70	2.63
OC-11	0.17	1.40	0.08
OC-20	6.18	45.30	2.62
OC-21	7.33	40.70	2.36
OD-1	24.25	84.70	7.82
OD-2	30.30	148.20	9.77

LEGEND

- EXISTING GROUND ELEVATION CONTOUR (FT, MSL) (NOTE 1)
- EXISTING ROAD
- EXISTING FENCE
- EXISTING BUILDING
- EXISTING WATER LINE
- N 6917000 / E 2355000 — COORDINATE GRID (NOTE 2)
- PERMIT BOUNDARY
- 650 — PERMITTED FINISHED GROUND ELEVATION CONTOUR
- N7100 — SITE GRID
- DT — FINAL COVER DRAINAGE TERRACE AND FLOW DIRECTION (NOTE 5)
- FINAL COVER DOWNCHUTE DRAINAGE CHANNEL
- (A) DOWNCHUTE DRAINAGE CHANNEL DESIGNATION
- DRAINAGE AREA BOUNDARY
- CO-2 — DRAINAGE AREA DESIGNATION
- 100-YEAR FLOODPLAIN

0 200' 400'
SCALE IN FEET

NOTES:

1. EXISTING TOPOGRAPHIC BASE MAP COMPILED FROM PHOTOGRAMMETRIC METHODS BASED ON AERIAL PHOTOGRAPHY PERFORMED ON 06 MARCH 2019 BY DALLAS AERIAL SURVEYS, INC.
2. ELEVATIONS ARE IN FEET ABOVE MEAN SEA LEVEL (FT, MSL), AS DEFINED BY THE NORTH AMERICAN VERTICAL DATUM (NAVD) OF 1988. COORDINATE GRID BASED ON TEXAS STATE PLANE COORDINATE SYSTEM, TEXAS NORTH CENTRAL ZONE (4202), NORTH AMERICAN DATUM OF 1983 (NAD-83).
3. CURRENT PERMITTED SURFACE WATER MANAGEMENT SYSTEM PLAN TAKEN FROM MSW PERMIT 1983C SHEET 6E.1.
4. THE PURPOSE OF THIS DRAWING IS TO PRESENT THE DRAINAGE AREAS MODELED IN THE RUNOFF CALCULATION PACKAGE, ATTACHMENT 2B, FOR THE PRE-DEVELOPMENT CONDITION.

STATE OF TEXAS
SCOTT M. GRAVES
86557
LICENSED PROFESSIONAL ENGINEER
5/7/2020

FOR PERMIT PURPOSES ONLY

REV	DATE	DESCRIPTION	DRN	APP
-	MAY 2020	INITIAL SUBMITTAL TO TCEQ	JJV	SMG

WASTE CONNECTIONS, INC.
Connect with the Future

TEXAS REGIONAL LANDFILL COMPANY, LP

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4144 DICK PRICE ROAD
FORT WORTH, TEXAS 76140
PHONE: 817.516.7777

Geosyntec consultants
GEOSYNTEC CONSULTANTS, INC.
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8217 SHOAL CREEK BLVD, SUITE 200
AUSTIN, TEXAS 78757
PHONE: 512.451.4003

TITLE:
PRE-DEVELOPMENT PLAN WITH DRAINAGE PATTERNS

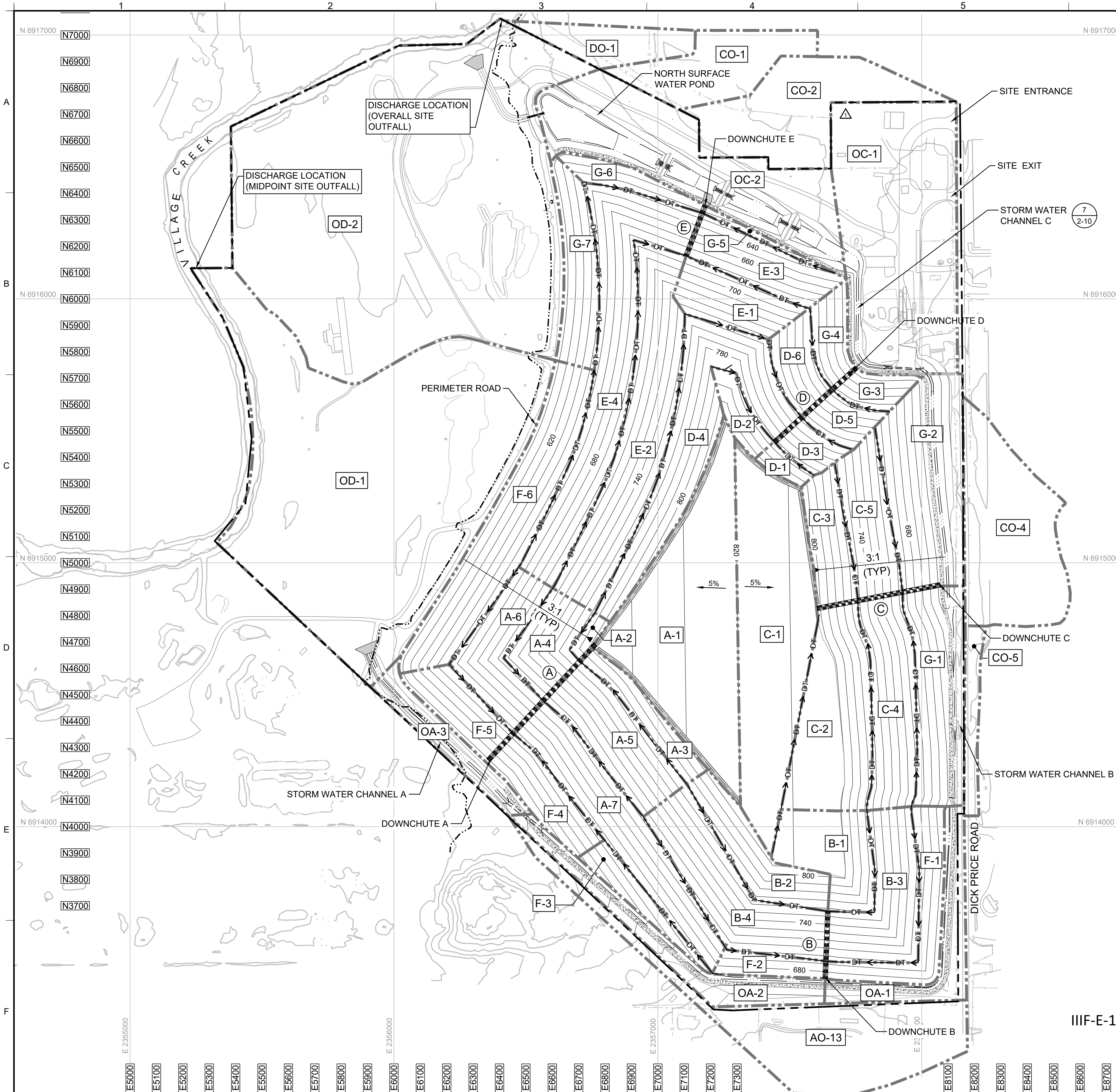
PROJECT:
FORT WORTH C&D LANDFILL EXPANSION
PERMIT AMENDMENT APPLICATION - PERMIT NO. MSW-1983D

PROJECT NO.: GW6953	DESIGN BY: SMG	REVIEWED BY: BG	PART NO.: III	DRAWING: 2-2
FILE: GW6953P2-2	DRAWN BY: JJV / KH	APPROVED BY: SMG		

IIIF-E-108

PERMIT DRAWING

DRAWING: Austin P:\CADD\Projects\Ft worth c&d landfill\PERMIT\landfill expansion (gw6953)\DRAWINGS\GW6953P2-2.dwg PLOTTED: May 05, 2020 - 2:28pm



LEGEND

580	EXISTING GROUND ELEVATION CONTOUR (FT, MSL) (NOTE 1)
	EXISTING ROAD
	EXISTING BUILDING
	EXISTING WATER LINE
N 6917000 E 2355000	COORDINATE GRID (NOTE 2)
N7100	SITE GRID
	PERMIT BOUNDARY
650	PROPOSED GROUND ELEVATION CONTOUR (FT, MSL) (NOTE 3)
DT ← DT	FINAL COVER DRAINAGE TERRACE AND FLOW DIRECTION (NOTE 5)
	FINAL COVER DOWNCHUTE DRAINAGE CHANNEL
A	DOWNCHUTE DRAINAGE CHANNEL DESIGNATION
	PERIMETER DRAINAGE CHANNEL
	DRAINAGE AREA BOUNDARY
G-3	DRAINAGE AREA DESIGNATION
	100-YEAR FLOODPLAIN

0 200' 400'
SCALE IN FEET

POST-DEVELOPMENT SUMMARY

DRAINAGE BASIN DESIGNATION	DRAINAGE AREA (ACRE)	25-YEAR PEAK FLOW RATE (CFS)	25-YEAR RUNOFF VOLUME (AC-FT)
A-1	8.82	66.20	3.98
A-2	0.25	2.10	0.11
A-3	1.22	10.20	0.55
A-4	1.71	14.30	0.77
A-5	2.85	23.80	1.29
A-6	2.49	20.80	1.12
A-7	4.10	34.30	1.85
AO-13	6.81	39.80	2.20
B-1	2.47	19.60	1.12
B-2	2.21	18.50	1.00
B-3	3.04	25.40	1.37
B-4	3.94	33.00	1.78
C-1	7.77	61.90	3.51
C-2	4.62	36.10	2.09
C-3	1.71	14.30	0.77
C-4	3.16	26.40	1.43
C-5	2.27	19.00	1.03
CO-1	3.02	17.30	0.97
CO-2	4.85	26.70	1.56
CO-4	5.73	34.90	1.85
CO-5	0.94	5.90	0.30
D-1	0.38	3.20	0.17
D-2	0.77	6.40	0.35
D-3	0.71	5.90	0.32
D-4	5.22	43.70	2.36
D-5	0.87	7.30	0.39
D-6	1.21	10.10	0.55
DO-1	1.74	9.90	0.56
E-1	1.87	15.60	0.84
E-2	5.71	47.80	2.58
E-3	2.11	17.70	0.95
E-4	6.26	52.40	2.83
F-1	2.07	17.30	0.93
F-2	0.76	6.40	0.34
F-3	1.00	8.40	0.45
F-4	1.72	14.40	0.78
F-5	1.55	13.00	0.70
F-6	5.35	44.80	2.42
G-1	3.68	30.80	1.66
G-2	5.10	42.70	2.30
G-3	0.76	6.40	0.34
G-4	1.04	8.70	0.47
G-5	0.26	2.20	0.12
G-6	0.94	7.90	0.42
G-7	2.23	18.60	1.01
OA-1	2.24	18.70	1.01
OA-2	2.66	22.10	1.20
OA-3	1.53	12.80	0.69
OC-1	10.52	71.80	4.45
OC-2	7.62	42.30	2.46
OD-1	24.72	96.60	7.97
OD-2	30.56	149.50	9.86

NOTES:

1. EXISTING TOPOGRAPHIC BASE MAP COMPILED FROM PHOTOGRAMMETRIC METHODS BASED ON AERIAL PHOTOGRAPHY PERFORMED ON 06 MARCH 2019 BY DALLAS AERIAL SURVEYS, INC.
2. ELEVATIONS ARE IN FEET ABOVE MEAN SEA LEVEL (FT, MSL), AS DEFINED BY THE NORTH AMERICAN VERTICAL DATUM (NAVD) OF 1988. COORDINATE GRID BASED ON TEXAS STATE PLANE COORDINATE SYSTEM, TEXAS NORTH CENTRAL ZONE (4202), NORTH AMERICAN DATUM OF 1983 (NAD-83).
3. THE PURPOSE OF THIS DRAWING IS TO PRESENT THE DRAINAGE AREAS MODELED IN THE RUNOFF CALCULATION PACKAGE, ATTACHMENT 2B, FOR THE POST-DEVELOPMENT CONDITION.
4. SEE DRAWING 2-4 FOR PERIMETER CHANNEL DESIGNATIONS USED IN RUNOFF CALCULATION PACKAGE, ATTACHMENT 2B AND 2E.
5. SEE DRAWING 2-1 FOR SURFACE WATER DESIGN USED IN RUNOFF CALCULATION PACKAGE, ATTACHMENT 2B AND 2E.

STATE OF TEXAS
SCOTT M. GRAVES
86557
LICENSED
PROFESSIONAL ENGINEER

12/14/2020

FOR PERMIT PURPOSES ONLY

REV	DATE	DESCRIPTION	DRN	APP
⚠	DEC 2020	RESPONSE TO NOD 2	JJV	SMG
-	MAY 2020	INITIAL SUBMITTAL TO TCEQ	JJV	SMG

WASTE CONNECTIONS, INC.
Connect with the Future

TEXAS REGIONAL LANDFILL COMPANY, LP

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Geosyntec consultants
GEOSYNTEC CONSULTANTS, INC.
TEXAS ENG. FIRM REGISTRATION NO. 1182
8217 SHOAL CREEK BLVD, SUITE 200
AUSTIN, TEXAS 78757
PHONE: 512.451.4003

TITLE:
POST-DEVELOPMENT PLAN WITH DRAINAGE PATTERNS

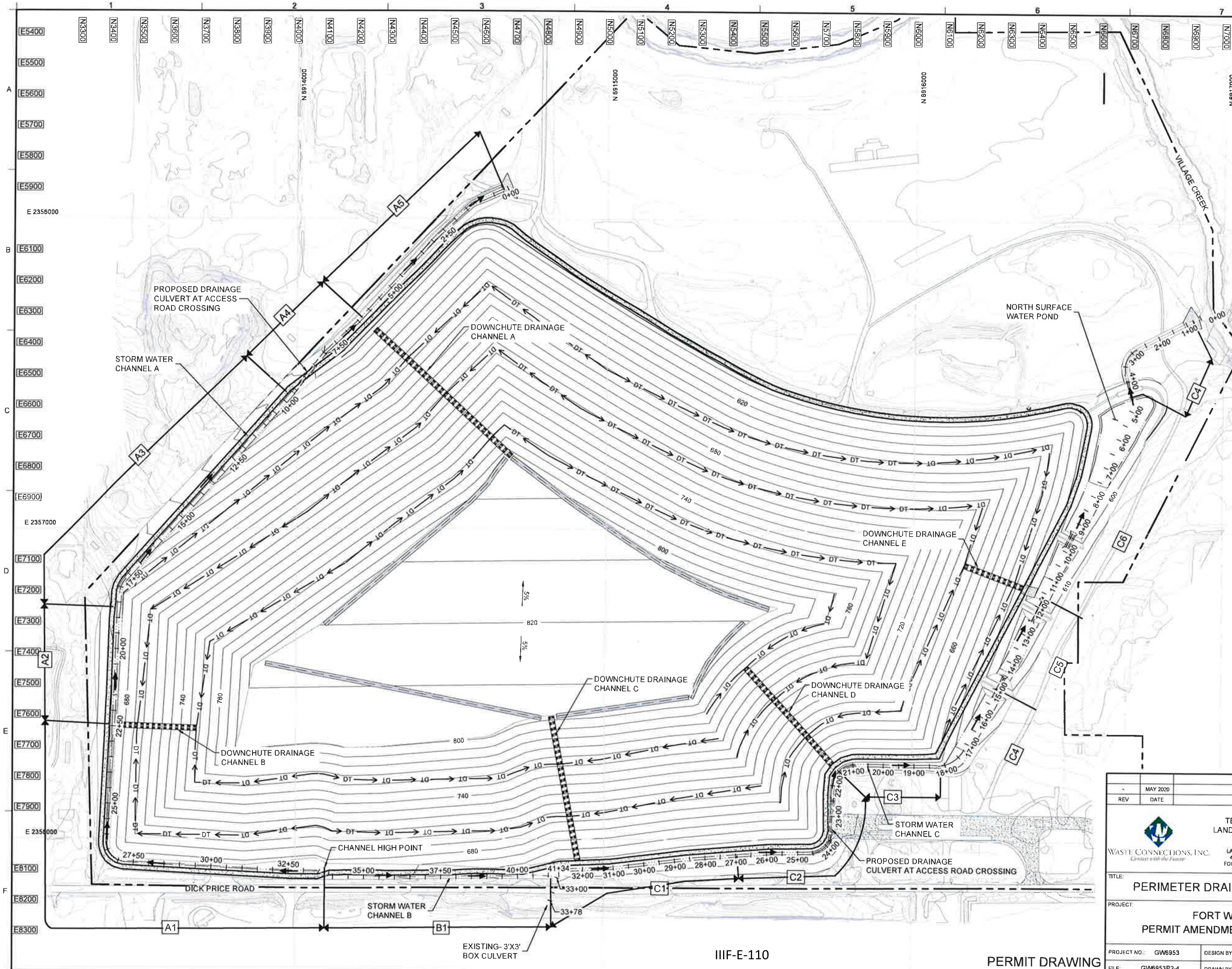
PROJECT:
FORT WORTH C&D LANDFILL EXPANSION
PERMIT AMENDMENT APPLICATION - PERMIT NO. MSW-1983D

IIIF-E-109

PERMIT DRAWING

PROJECT NO.: GW6953	DESIGN BY: BK	REVIEWED BY: BG	PART NO.: III	DRAWING: 2-3
FILE: GW6953P2-3	DRAWN BY: JJV / KH	APPROVED BY: SMG		

DRAWING: Austin P:\CADD\Projects\FtWorth c&d landfill\PERMIT\landfill expansion (gw6953)\DRAWINGS\GW6953P2-3.dwg PLOTTED: Dec 07, 2020 - 2:34pm



LEGEND

- EXISTING GROUND ELEVATION CONTOUR (FT, MSL) (NOTE 1)
- EXISTING WATER LINE
- N 6917000 E 2355000 — COORDINATE GRID (NOTE 2)
- N7100 — SITE GRID
- PERMIT BOUNDARY
- 650 — PROPOSED GROUND ELEVATION CONTOUR (FT, MSL)
- DT — SIDESLOPE DRAINAGE TERRACE
- DOWNCHUTE DRAINAGE CHANNEL
- TOP DECK DRAINAGE TERRACE
- C4 — CHANNEL REACH DESIGNATION
- PERIMETER CHANNEL FLOW DIRECTION
- 25+00 — DRAINAGE CHANNEL CENTERLINE ALIGNMENT AND STATIONING

0 150' 300'
SCALE IN FEET

NOTES:

1. THE EXISTING TOPOGRAPHIC BASE MAP WAS COMPILED FROM PHOTOGRAMMETRIC METHODS BASED ON AERIAL PHOTOGRAPHY PERFORMED ON 06 MARCH 2019 BY DALLAS AERIAL SURVEYS, INC.
2. ELEVATIONS ARE IN FEET ABOVE MEAN SEA LEVEL (FT, MSL), AS DEFINED BY THE NORTH AMERICAN VERTICAL DATUM (NAVD) OF 1988. STATE PLANE COORDINATE GRID BASED ON TEXAS STATE PLANE COORDINATE SYSTEM, TEXAS NORTH CENTRAL ZONE (4202), NORTH AMERICAN DATUM OF 1983 (NAD-83).
3. REFER TO PART III, ATTACHMENT 2, DRAWING 2-1 FOR THE SURFACE WATER MANAGEMENT FEATURES ON THE FINAL COVER SYSTEM (NOT SHOWN ON THIS DRAWING FOR CLARITY.)
4. SEE ATTACHMENT 2E FOR DESIGN OF PERIMETER DRAINAGE CHANNELS.

STATE OF TEXAS
SCOTT M. GRAVES
86557
LICENSED PROFESSIONAL ENGINEER
5/7/2020

FOR PERMIT PURPOSES ONLY

REV	DATE	DESCRIPTION	DRN	APP
-	MAY 2020	INITIAL SUBMITTAL TO TCEQ	JJV	SMG

WASTE CONNECTIONS, INC.

TEXAS REGIONAL LANDFILL COMPANY, LP

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8217 SHOAL CREEK BLVD, SUITE 200
AUSTIN, TEXAS 78757
PHONE: 512.451.4003

TITLE: PERIMETER DRAINAGE CHANNEL PLAN WITH STATIONING

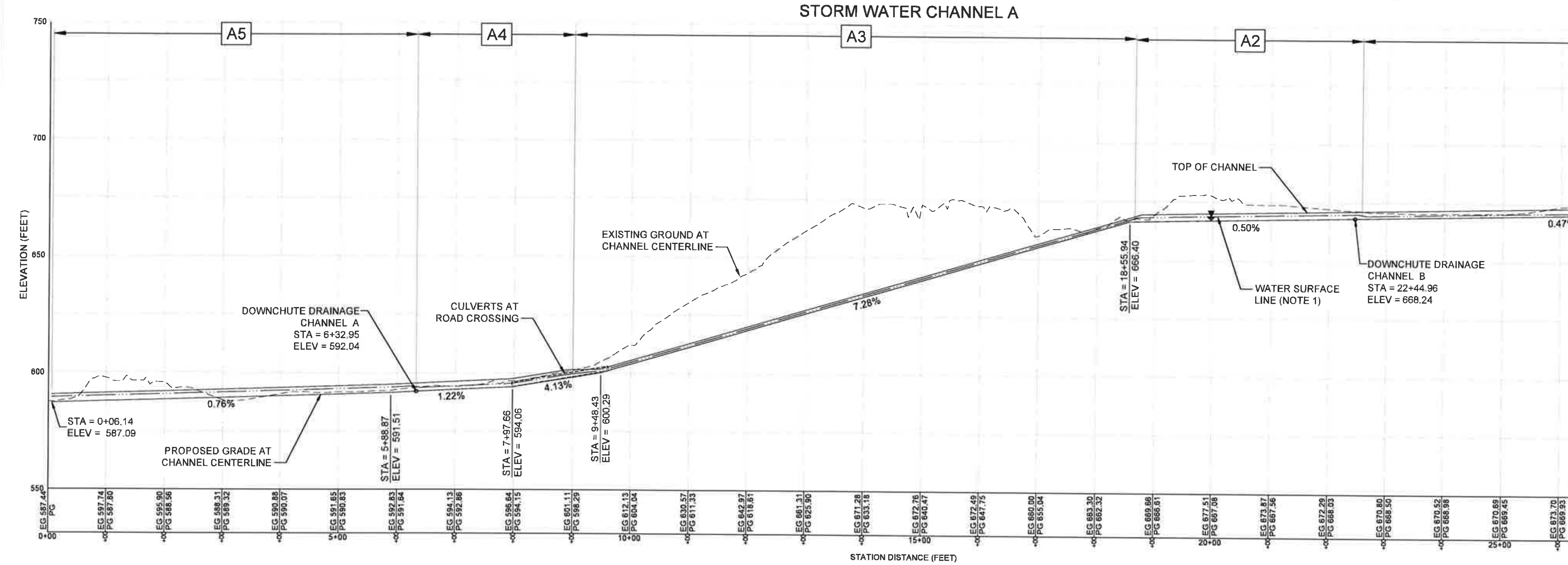
PROJECT: FORT WORTH C&D LANDFILL EXPANSION
PERMIT AMENDMENT APPLICATION - PERMIT NO. MSW-1983D

PROJECT NO.: GW6953	DESIGN BY: BK	REVIEWED BY: BG	PART NO.: III	DRAWING: 2-4
FILE: GW6953P2-4	DRAWN BY: JJV / KH	APPROVED BY: SMG		

PERMIT DRAWING

DRAWING: Austin P:\CADD\Projects\FW worth c&d landfill expansion (gw6953)\DRAWINGS\GW6953P2-4.dwg PLOTTED: May 05, 2020 - 2:29pm

STORM WATER CHANNEL A

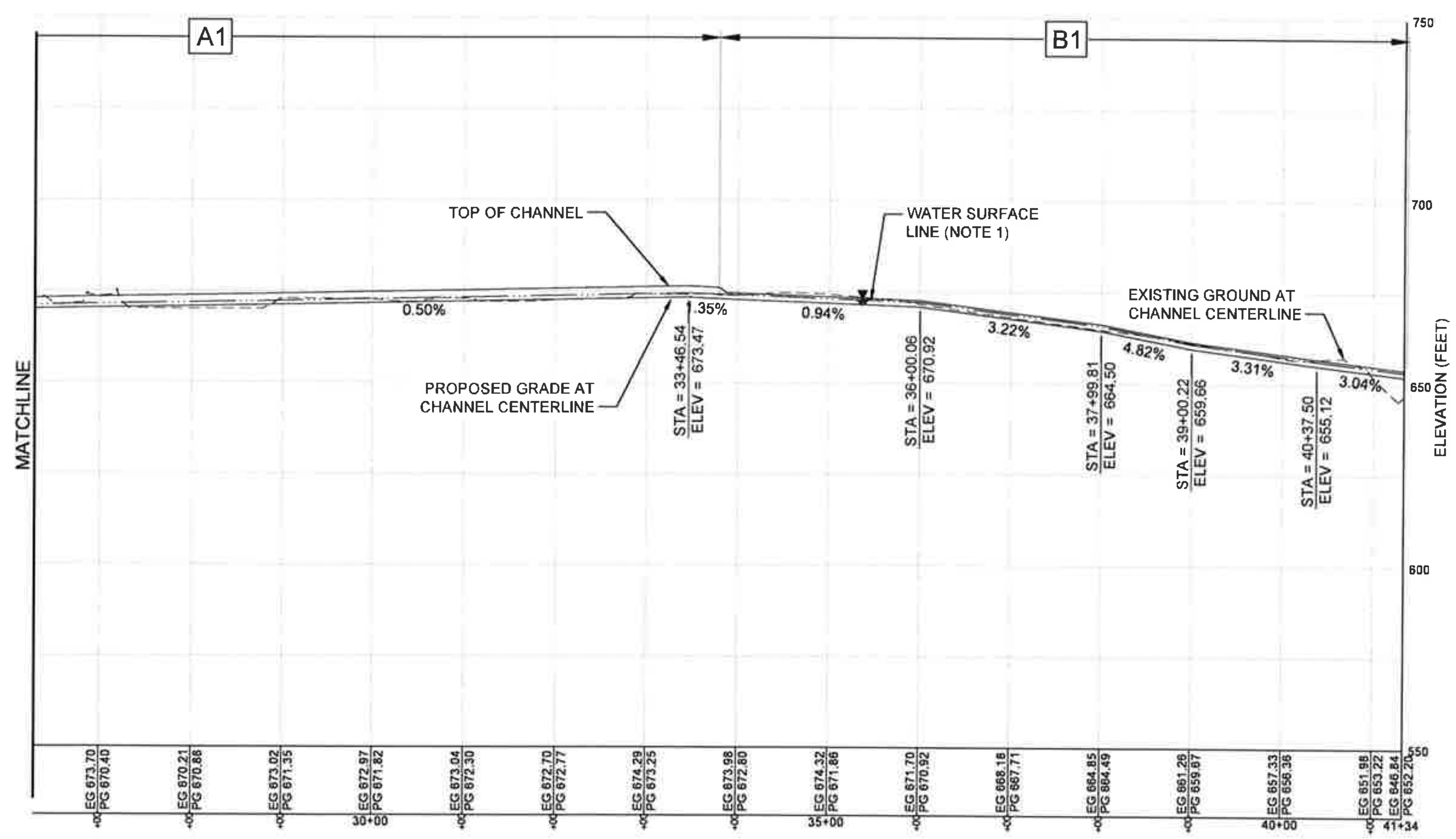


LEGEND

(line)	PROPOSED GRADE ALONG CHANNEL CENTERLINE
(line)	WATER SURFACE LINE (NOTE 1)
A1	CHANNEL REACH DESIGNATION
PG 650.00	PROPOSED GRADE ELEVATION
EG 598.00	EXISTING GRADE ELEVATION

0 100' 200'
HORIZONTAL SCALE IN FEET

0 25 50'
VERTICAL SCALE IN FEET



NOTES:

1. WATER SURFACE LINE REPRESENTS THE 25-YEAR, 24-HOUR NORMAL DEPTH OF FLOW.
2. SEE DRAWING 2-10 FOR TABLE OF PERIMETER DRAINAGE CHANNEL DESIGNATIONS, DIMENSIONS (WIDTH, DEPTH), AND CHANNEL LINING TYPE. SEE ATTACHMENT 2E FOR PERIMETER CHANNEL SIZING DESIGN.
3. PERIMETER DRAINAGE CHANNEL REACHES WILL BE CONSTRUCTED IN ACCORDANCE WITH THE INSTALLATION SCHEDULE DESCRIBED IN IN ATTACHMENT 2 (DRAINAGE REPORT, SECTION 8). THIS INCLUDES MITIGATING AREAS WHERE DISCREPANCIES EXIST BETWEEN THE EXISTING GRADE OF THE CHANNEL AND THE DESIGN GRADE. ADDITIONALLY, PERIMETER CHANNELS WILL BE INSPECTED AND MAINTAINED DURING OPERATIONS AND AFTER CLOSURE (CONSISTENT WITH SECTION 5 OF THE INTERMEDIATE COVER EROSION AND SEDIMENT CONTROL PLAN AND SECTION 3.3 OF THE POST-CLOSURE PLAN, RESPECTIVELY) WHICH WILL INCLUDE MITIGATING AREAS WHERE DISCREPANCIES OCCUR BETWEEN THE EXISTING GROUND ELEVATION AND THE DESIGN ELEVATION/GRADE OF THE PERIMETER CHANNELS.

STATE OF TEXAS
SCOTT M. GRAVES
86557
LICENSED PROFESSIONAL ENGINEER

5/7/2020

FOR PERMIT PURPOSES ONLY

REV	DATE	DESCRIPTION	DRN	APP
-	MAY 2020	INITIAL SUBMITTAL TO TCEQ	JJV	SMG

WASTE CONNECTIONS, INC.
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TEXAS REGIONAL LANDFILL COMPANY, LP

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8217 SHOAL CREEK BLVD, SUITE 200
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PHONE: 512.451.4003

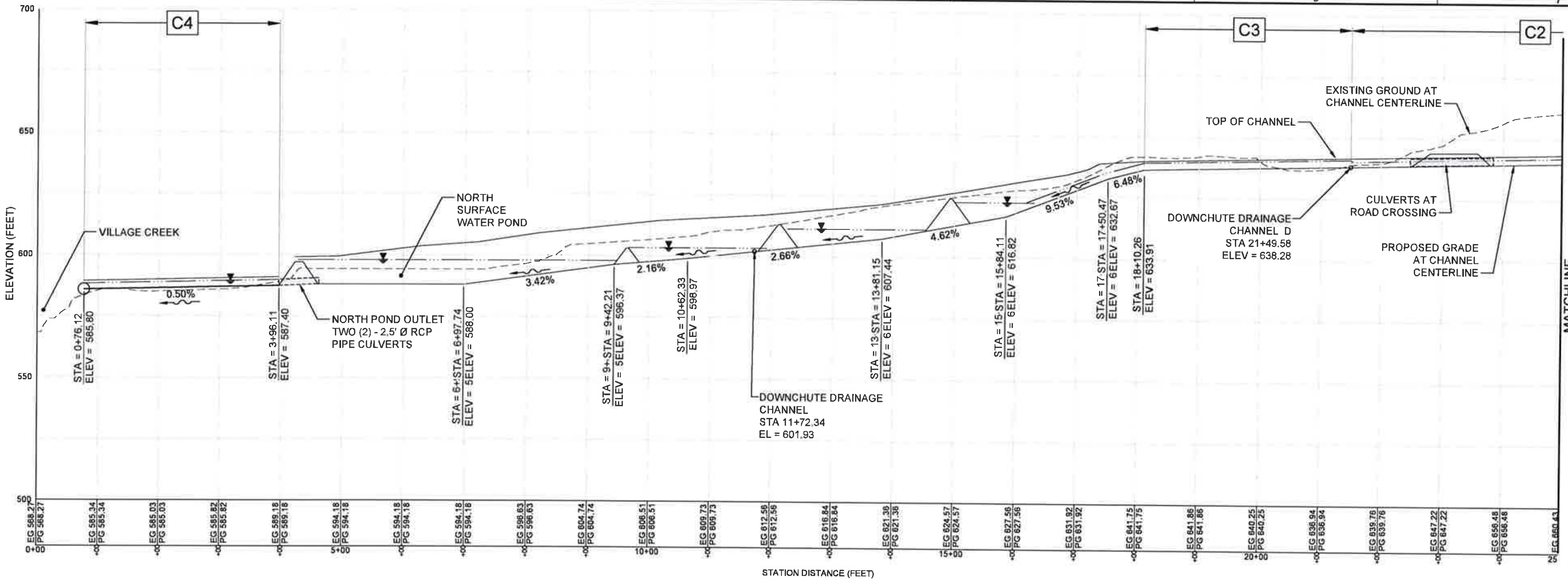
TITLE:
STORM WATER CHANNEL A AND B PROFILES

PROJECT:
FORT WORTH C&D LANDFILL EXPANSION
PERMIT AMENDMENT APPLICATION - PERMIT NO. MSW-1983D

PROJECT NO.: GW6953	DESIGN BY: BK	REVIEWED BY: BG	PART NO.: III	DRAWING: 2-5
FILE: GW6953P2-5	DRAWN BY: JJV / KH	APPROVED BY: SMG		

PERMIT DRAWING

DRAWING: Austin P:\CADD\Projects\Ft worth c&d landfill\PERMIT\landfill expansion (gw6953)\DRAWINGS\GW6953P2-5.dwg PLOTTED: May 05, 2020 - 2:30pm

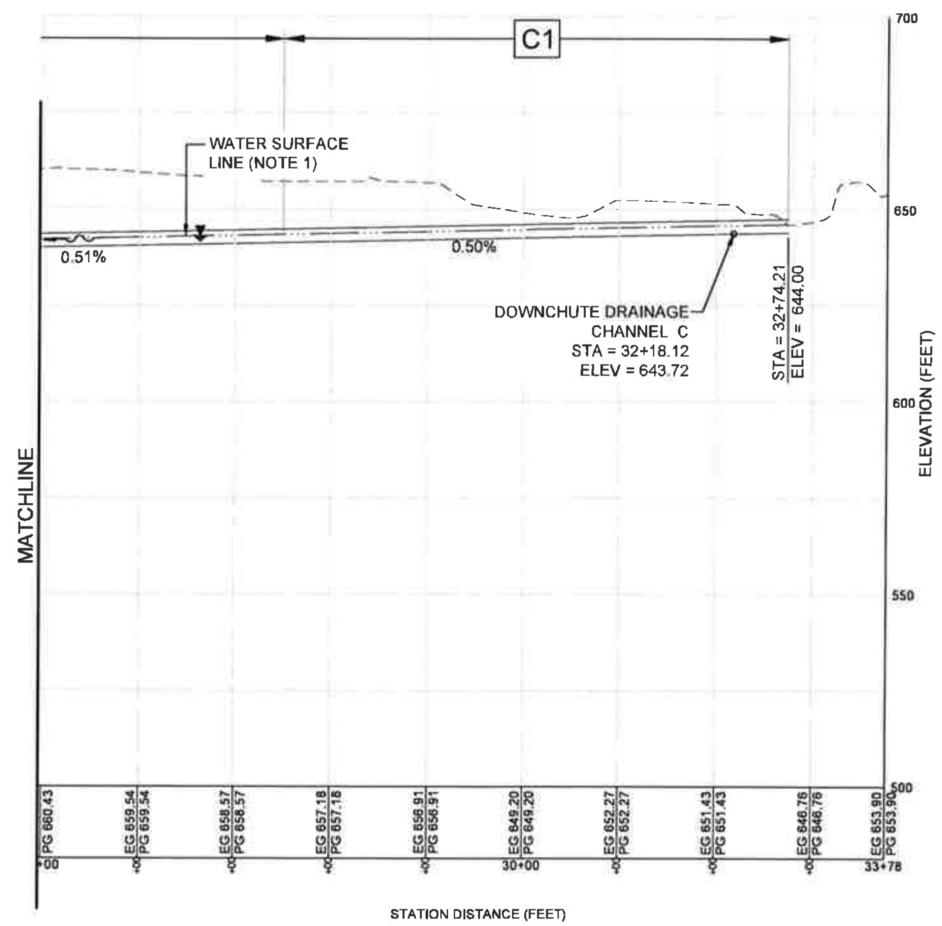


LEGEND

- PROPOSED GRADE ALONG CHANNEL CENTERLINE
- WATER SURFACE LINE (NOTE 1)
- C4 — CHANNEL REACH DESIGNATION
- PG 650.00 — PROPOSED GRADE ELEVATION
- EG 598.00 — EXISTING GRADE ELEVATION

0 100' 200'
HORIZONTAL SCALE IN FEET

0 25 50'
VERTICAL SCALE IN FEET



NOTES:

1. WATER SURFACE LINE REPRESENTS THE 25-YEAR, 24-HOUR NORMAL DEPTH OF FLOW.
2. SEE DRAWING 2-10 FOR TABLE OF PERIMETER DRAINAGE CHANNEL DESIGNATIONS, DIMENSIONS (WIDTH, DEPTH), AND CHANNEL LINING TYPE. SEE ATTACHMENT 2E FOR PERIMETER CHANNEL SIZING DESIGN.
3. PERIMETER DRAINAGE CHANNEL REACHES WILL BE CONSTRUCTED IN ACCORDANCE WITH THE INSTALLATION SCHEDULE DESCRIBED IN IN ATTACHMENT 2 (DRAINAGE REPORT, SECTION 8). THIS INCLUDES MITIGATING AREAS WHERE DISCREPANCIES EXIST BETWEEN THE EXISTING GRADE OF THE CHANNEL AND THE DESIGN GRADE. ADDITIONALLY, PERIMETER CHANNELS WILL BE INSPECTED AND MAINTAINED DURING OPERATIONS AND AFTER CLOSURE (CONSISTENT WITH SECTION 5 OF THE INTERMEDIATE COVER EROSION AND SEDIMENT CONTROL PLAN AND SECTION 3.3 OF THE POST-CLOSURE PLAN, RESPECTIVELY) WHICH WILL INCLUDE MITIGATING AREAS WHERE DISCREPANCIES OCCUR BETWEEN THE EXISTING GROUND ELEVATION AND THE DESIGN ELEVATION/GRADE OF THE PERIMETER CHANNELS.

STATE OF TEXAS
SCOTT M. GRAVES
86557
LICENSED PROFESSIONAL ENGINEER
5/7/2020

FOR PERMIT PURPOSES ONLY

REV	DATE	DESCRIPTION	DRN	APP
-	MAY 2020	INITIAL SUBMITTAL TO TCEQ	JJV	SMG

WASTE CONNECTIONS, INC. Connect with the Future

TEXAS REGIONAL LANDFILL COMPANY, LP

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TEXAS ENG. FIRM REGISTRATION NO. 1182
8217 SHOAL CREEK BLVD, SUITE 200
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PHONE: 512.451.4003

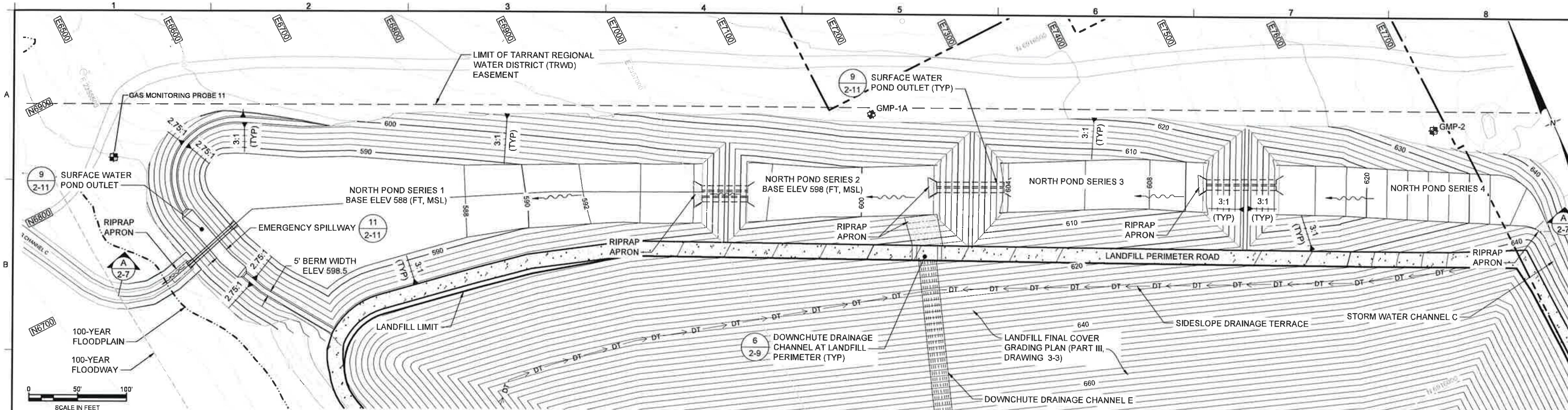
TITLE: PROFILE CHANNEL C

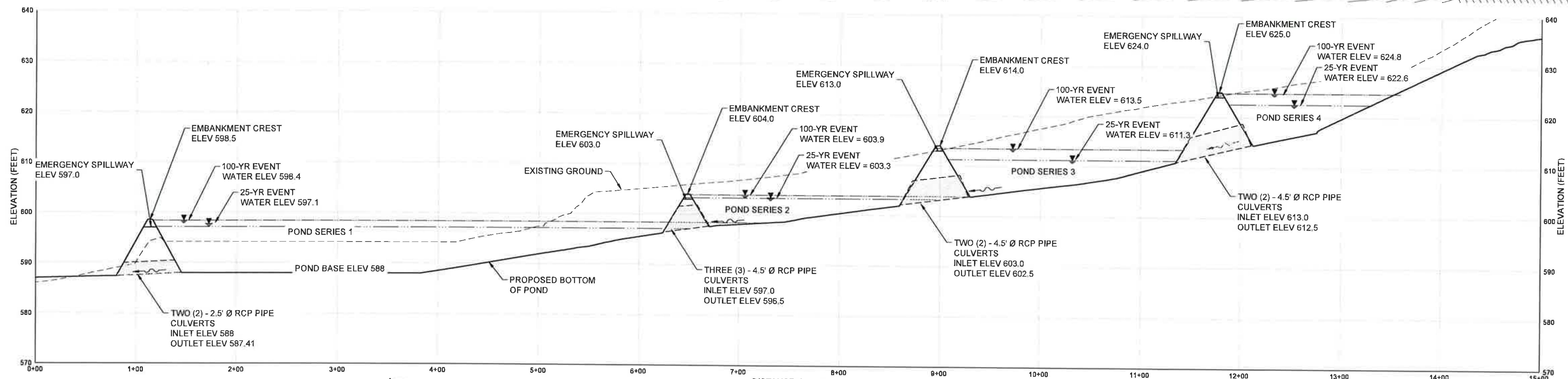
PROJECT: FORT WORTH C&D LANDFILL EXPANSION
PERMIT AMENDMENT APPLICATION - PERMIT NO. MSW-1983D

PROJECT NO.:	GW6953	DESIGN BY:	BK	REVIEWED BY:	BG	PART NO.:	DRAWING:
FILE:	GW6953P2-5	DRAWN BY:	JJV / KH	APPROVED BY:	SMG	III	2-6

PERMIT DRAWING

DRAWING: Austin P:\CADD\Projects\Ft worth c&d landfill\PERMIT\landfill expansion (gw6953)\DRAWINGS\GW6953P2-5.dwg PLOTTED: May 06, 2020 - 2:30pm





SECTION A

VERTICAL SCALE IN FEET

SCALE IN FEET

LEGEND

590	EXISTING GROUND ELEVATION CONTOUR (FT, MSL) (NOTE 1)
	EXISTING WATER LINE
N 6917000 E 2355000	COORDINATE GRID (NOTE 2)
N7100	SITE GRID
	PERMIT BOUNDARY
650	PROPOSED GROUND ELEVATION CONTOUR (FT, MSL) (NOTE 2)
	100-YEAR FLOODPLAIN (NOTE 3)
	100-YEAR FLOODWAY (NOTE 3)

NOTES:

1. THE EXISTING TOPOGRAPHIC BASE MAP WAS COMPILED FROM PHOTOGRAMMETRIC METHODS BASED ON AERIAL PHOTOGRAPHY PERFORMED ON 06 MARCH 2019 BY DALLAS AERIAL SURVEYS, INC.
2. ELEVATIONS ARE IN FEET ABOVE MEAN SEA LEVEL (FT, MSL), AS DEFINED BY THE NORTH AMERICAN VERTICAL DATUM (NAVD) OF 1988. STATE PLANE COORDINATE GRID BASED ON TEXAS STATE PLANE COORDINATE SYSTEM, TEXAS NORTH CENTRAL ZONE (4202), NORTH AMERICAN DATUM OF 1983 (NAD-83).
3. 100-YEAR FLOODPLAIN LIMITS ARE FROM FEMA MAPS AND APPROVED CONDITIONAL LETTER OF MAP REVISION (CLOMR), AS DESCRIBED IN PART II, NARRATIVE REPORT, SECTIONS 10.1.2 AND 10.1.3 AND SHOWN IN DRAWING IIF-1.

5/7/2020

STATE OF TEXAS
SCOTT M. GRAVES
86557
LICENSED PROFESSIONAL ENGINEER

FOR PERMIT PURPOSES ONLY

REV	DATE	DESCRIPTION	DRN	APP
-	MAY 2020	INITIAL SUBMITTAL TO TCEQ	JJV	SMG

WASTE CONNECTIONS, INC.

TEXAS REGIONAL LANDFILL COMPANY, LP

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Geosyntec consultants
GEOSYNTEC CONSULTANTS, INC. TEXAS ENG. FIRM REGISTRATION NO. 1182 8217 SHOAL CREEK BLVD, SUITE 200 AUSTIN, TEXAS 78757 PHONE: 512.451.4003

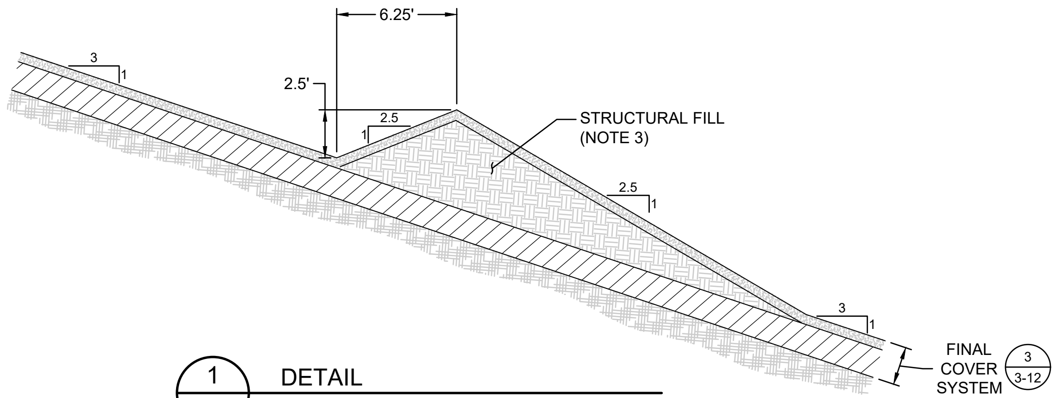
TITLE: NORTH SURFACE WATER POND

PROJECT: FORT WORTH C&D LANDFILL EXPANSION PERMIT AMENDMENT APPLICATION - PERMIT NO. MSW-1983D

PROJECT NO.: GW6953	DESIGN BY: BK	REVIEWED BY: BG	PART NO.: III	DRAWING: 2-7
FILE: GW6953P2-8	DRAWN BY: JJV / KH	APPROVED BY: SMG		

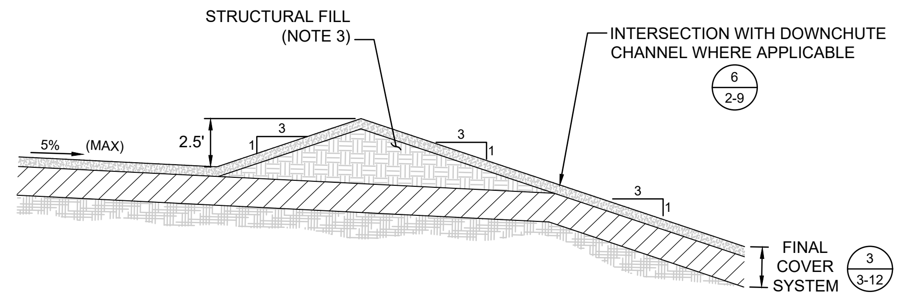
PERMIT DRAWING

DRAWING: Austin P:\CADD\Projects\Fort worth c&d landfill\PERMIT\landfill expansion (gw6953)\DRAWINGS\GW6953P2-8.dwg PLOTTED: May 06, 2020 - 2:30pm



1 / 2-1 DETAIL
SIDESLOPE DRAINAGE TERRACE

0 5' 10'
SCALE IN FEET



2 / 2-1 DETAIL
TOP DECK DRAINAGE TERRACE

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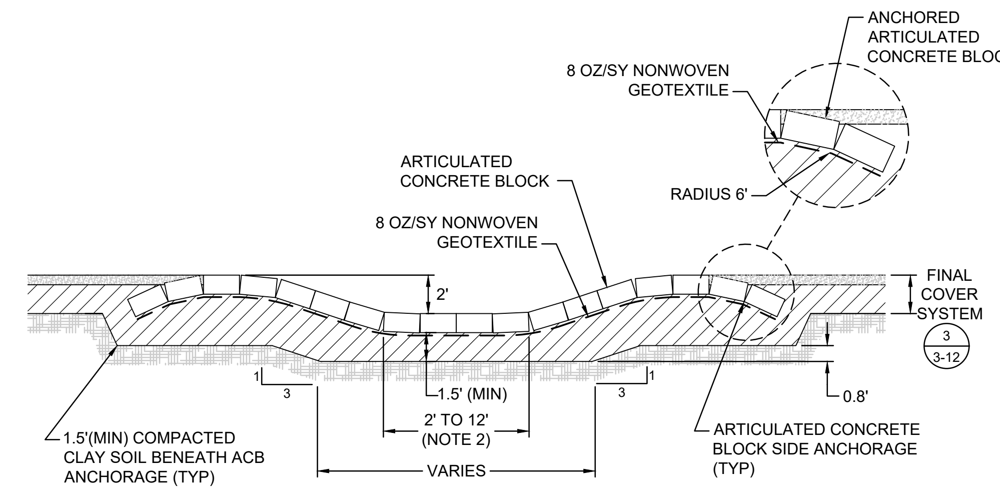
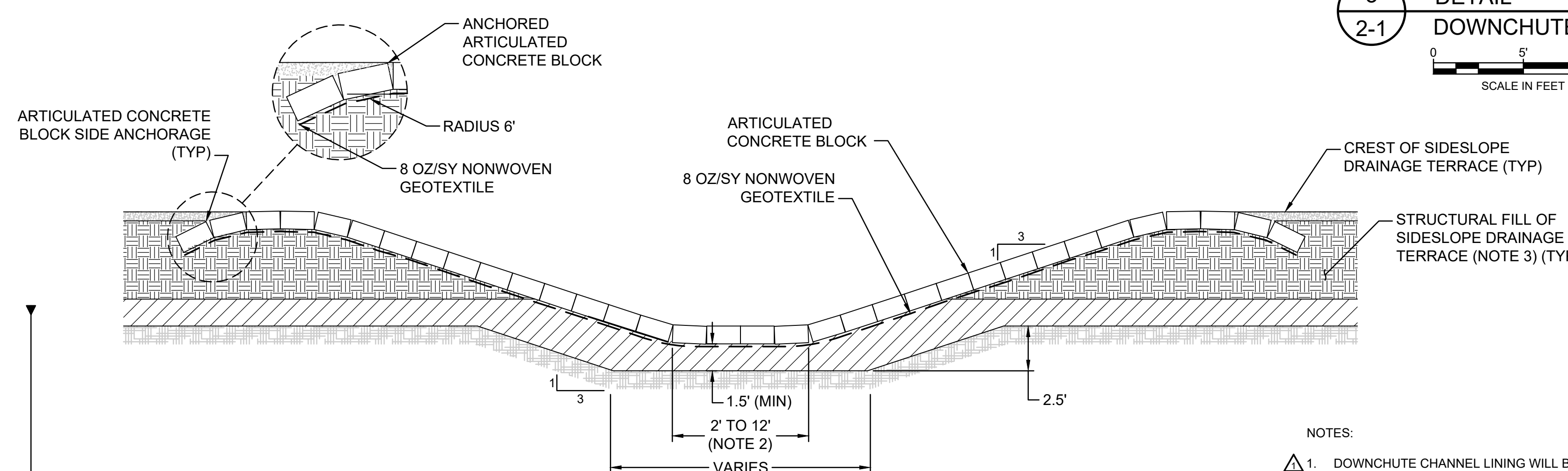
SCALE IN FEET



3 / 2-1 DETAIL
DOWNCHUTE DRAINAGE CHANNEL (NOTE 1)

0 5' 10'
SCALE IN FEET

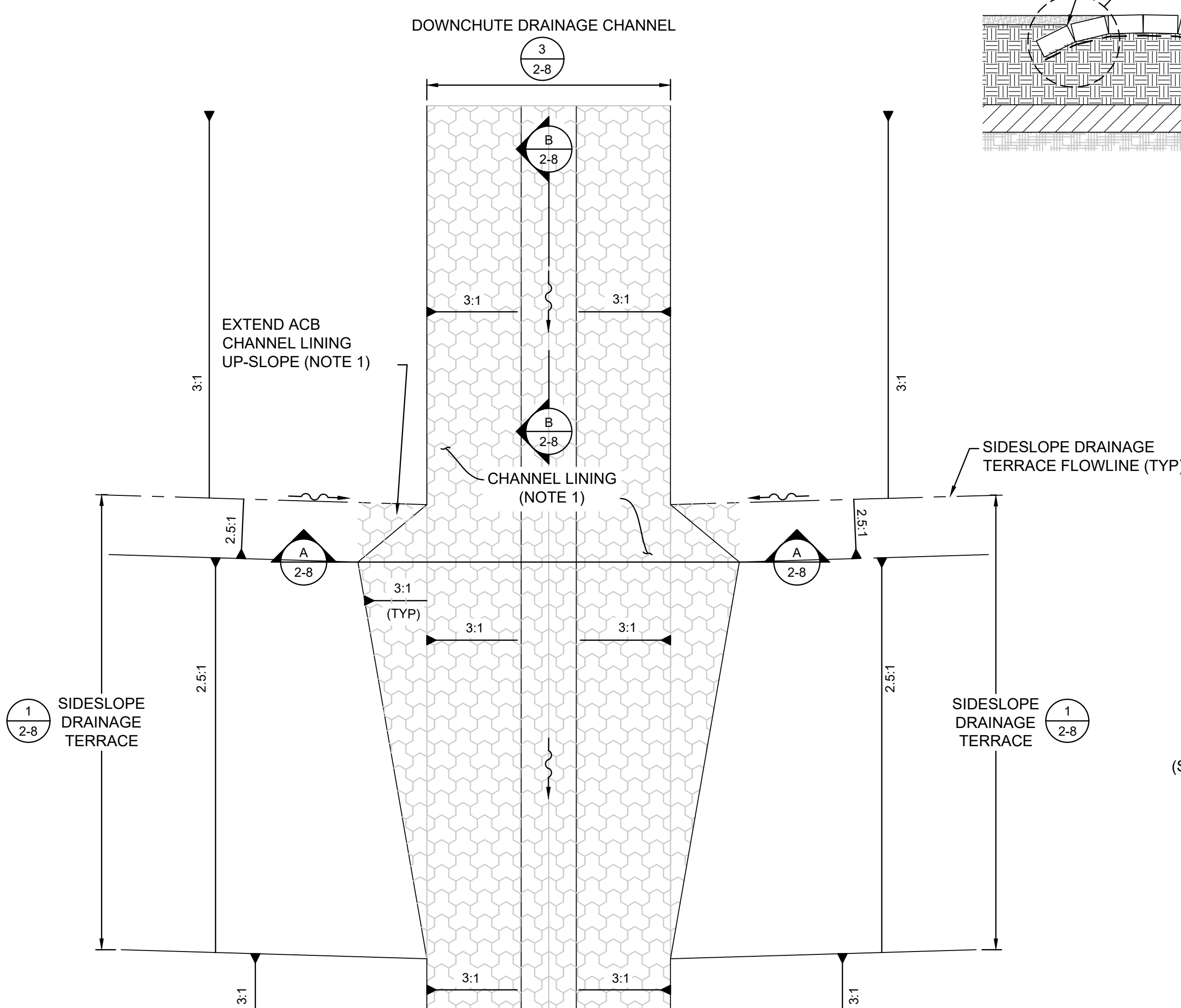


A / 2-8 SECTION
DOWNCHUTE DRAINAGE CHANNEL SECTION (NOTE 1)

SCALE: N.T.S.

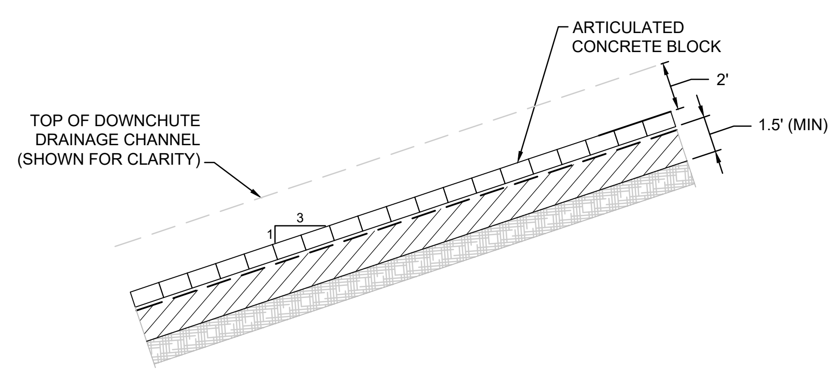
NOTES:

1. DOWNCHUTE CHANNEL LINING WILL BE ARTICULATED CONCRETE BLOCK (ACB) AS SHOWN. ALTERNATIVELY, AN EQUIVALENT CHANNEL LINING SYSTEM (e.g., GABION/RENO MATTRESS, RIPRAP, GEOMEMBRANE) MAY BE USED, PROVIDED THAT THE ALTERNATE SYSTEM SHALL BE DESIGNED BY A TEXAS LICENSED PROFESSIONAL ENGINEER (P.E.) TO PROVIDE A PERMISSIBLE TRACTIVE STRESS GREATER THAN OR EQUAL TO 16.5 POUNDS PER SQUARE FOOT; BE GEOTECHNICALLY STABLE; AND MEET THE HYDRAULIC SIZING CRITERIA SET FORTH IN PART III, ATTACHMENT 2D FOR THE DESIGN STORM.
2. REFER TO PART III, ATTACHMENT 2D FOR DOWNCHUTE CHANNEL WIDTHS, AND DRAWING 2-3 FOR DOWNCHUTE DESIGNATIONS.
3. STRUCTURAL FILL SOIL ASSOCIATED WITH THE DRAINAGE FEATURES SHOWN ON THIS DRAWING SHALL BE NATURAL SOIL, CLEAN AND UNCONTAMINATED, FREE OF ORGANIC MATTER (I.E., ROOTS, VEGETATION), DEBRIS, FROZEN MATERIAL, OR EXCESSIVE MOISTURE. STRUCTURAL FILL SOIL SHALL HAVE 98 PERCENT BY WEIGHT SMALLER THAN 3 INCHES IN SIZE, AND SHALL BE PLACED IN 12-INCH THICK (MAXIMUM) LIFTS. EACH LIFT SHALL BE COMPACTED TO A DENSITY OF AT LEAST 92% OF THE STANDARD PROCTOR MAXIMUM DRY DENSITY (ASTM D 698).



4 / 2-1 DETAIL
TYPICAL SIDESLOPE DRAINAGE TERRACE AND DOWNCHUTE DRAINAGE CHANNEL INTERSECTION

0 10' 20'
SCALE IN FEET



B / 2-8 SECTION
DOWNCHUTE DRAINAGE CHANNEL SECTION (NOTE 1)

0 4' 8'
SCALE IN FEET

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REV	DATE	DESCRIPTION	DRN	APP
1	DEC 2020	RESPONSE TO NOD 2	JJV	SMG
-	MAY 2020	INITIAL SUBMITTAL TO TCEQ	JJV	SMG

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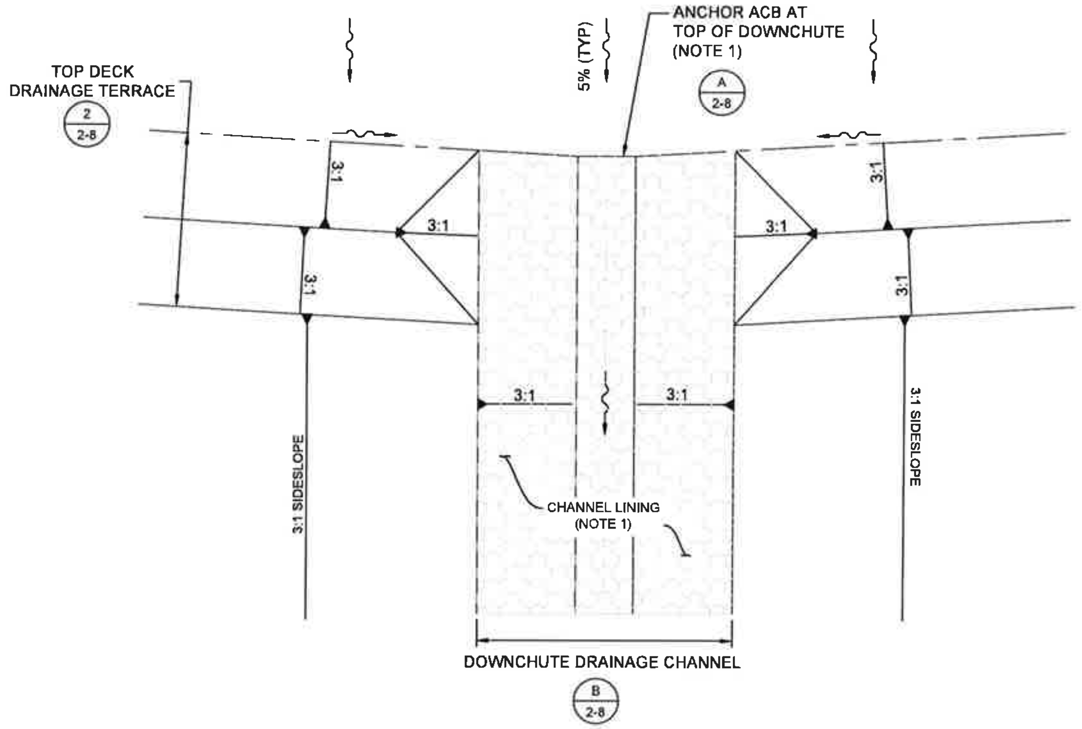
TITLE:
SURFACE WATER MANAGEMENT SYSTEM DETAILS I

PROJECT:
FORT WORTH C&D LANDFILL EXPANSION
PERMIT AMENDMENT APPLICATION - PERMIT NO. MSW-1983D

PROJECT NO.: GW6953	DESIGN BY: BK	REVIEWED BY: BG	PART NO.: III	DRAWING: 2-8
FILE: GW6953P2-10	DRAWN BY: JJV / KH	APPROVED BY: SMG		

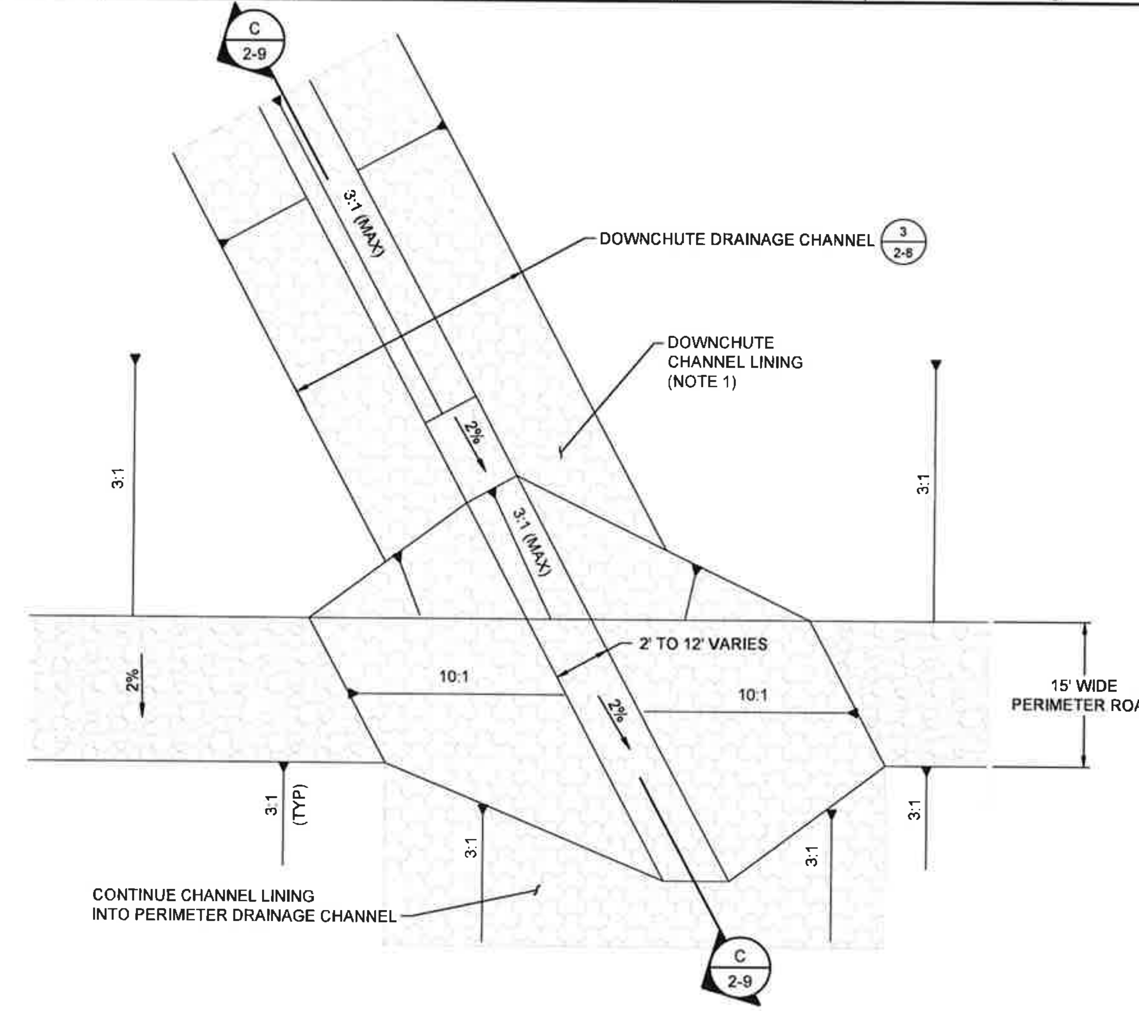
PERMIT DRAWING

DRAWING: Austin P:\CADD\Projects\Ft worth c&d landfill\PERMIT\landfill expansion (gw6953)\DRAWINGS\GW6953P2-10.dwg PLOTTED: Dec 07, 2020 - 2:34pm



5 2-1 DETAIL
TOP DECK DRAINAGE TERRACE AND DOWNCHUTE DRAINAGE CHANNEL INTERSECTION

0 10' 20' SCALE IN FEET

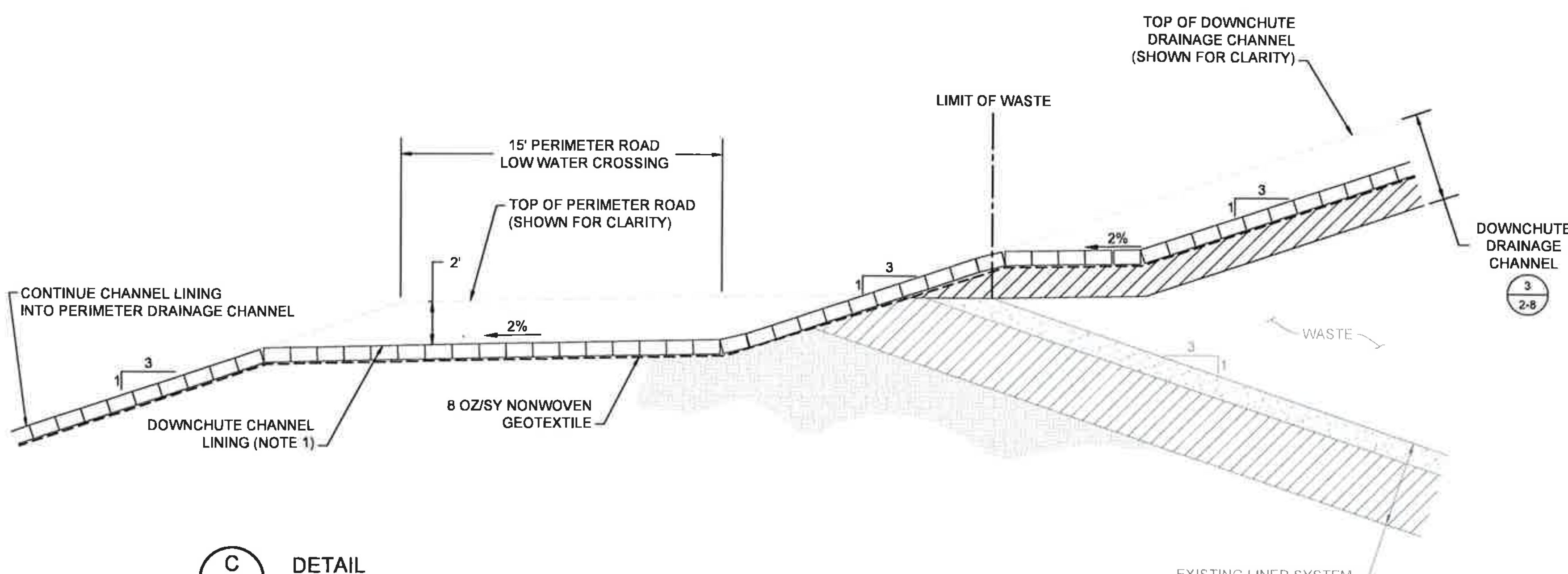


6 2-1 DETAIL
DOWNCHUTE DRAINAGE CHANNEL AT LANDFILL PERIMETER (TYP)

0 10' 20' SCALE IN FEET

NOTE:

1. DOWNCHUTE CHANNEL LINING WILL BE ARTICULATED CONCRETE BLOCK (ACB) OR APPROVED EQUIVALENT ALTERNATE CHANNEL LINING SYSTEM (e.g., GABION/RENO-MATTRESS, RIPRAP, GEOMEMBRANE) WITH A PERMISSIBLE TRACTIVE STRESS OF GREATER THAN OR EQUAL TO 16.5 POUNDS PER SQUARE FOOT



C 2-9 DETAIL
DOWNCHUTE DRAINAGE CHANNEL AT LANDFILL PERIMETER

0 4' 8' SCALE IN FEET

STATE OF TEXAS
SCOTT M. GRAVES
86557
LICENSED PROFESSIONAL ENGINEER
5/7/2020

FOR PERMIT PURPOSES ONLY

REV	DATE	DESCRIPTION	DRN	APP
-	MAY 2020	INITIAL SUBMITTAL TO TCEQ	JJV	SMG

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TITLE: SURFACE WATER MANAGEMENT SYSTEM DETAILS II

PROJECT: FORT WORTH C&D LANDFILL EXPANSION
PERMIT AMENDMENT APPLICATION - PERMIT NO. MSW-1983D

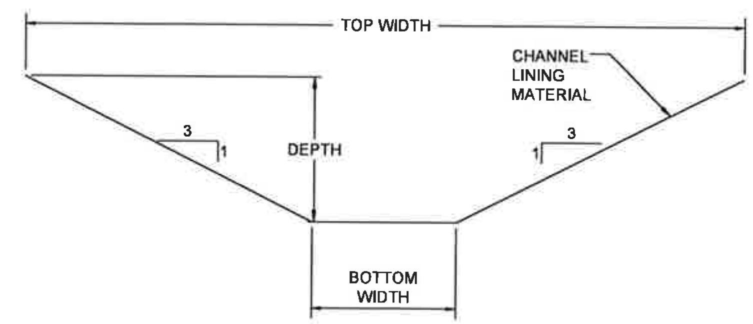
PROJECT NO.: GW6953	DESIGN BY: BK	REVIEWED BY: BG	PART NO.: III	DRAWING: 2-9
FILE: GW6953P2-12	DRAWN BY: JJV / KH	APPROVED BY: SMG		

PERMIT DRAWING

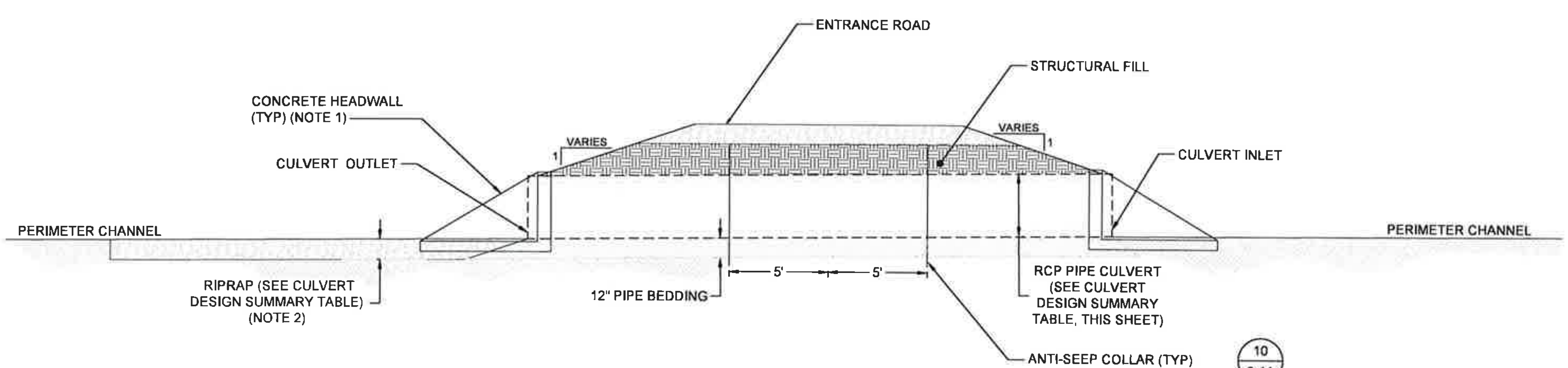
DRAWING: Austin P:\CADD\Projects\Ft worth c&d landfill\PERMIT\landfill expansion (gw6953)\DRAWINGS\GW6953P2-12.dwg PLOTTED: May 05, 2020 - 2:31pm

STORM WATER CHANNEL SCHEDULE

CHANNEL REACH SEGMENT	CHANNEL SHAPE	CHANNEL SLOPE (FT/FT)	MANNING'S N VALUE	BOTTOM WIDTH (FT)	DEPTH (FT)	SIDE SLOPES (H:V)	TOP WIDTH (FT)	25-YEAR, 24-HOUR PEAK FLOW, Q (CFS)	100-YEAR, 24-HOUR PEAK FLOW, Q (CFS)	CHANNEL LINING
PERIMETER REACH A1	TRAPEZOID	0.005	0.015	3.0	3.0	3:1	21.0	36.00	48.60	60-MIL HDPE GEOMEMBRANE
PERIMETER REACH A2	TRAPEZOID	0.005	0.015	8.0	3.0	3:1	26.0	198.10	278.00	60-MIL HDPE GEOMEMBRANE
PERIMETER REACH A3	TRAPEZOID	0.073	0.015	13.0	1.5	3:1	22.0	217.70	298.40	60-MIL HDPE GEOMEMBRANE
PERIMETER REACH A4	TRAPOZOID	1.220	0.015	8.0	3.5	3:1	22.0	216.90	298.20	60-MIL HDPE GEOMEMBRANE
PERIMETER REACH A5	TRAPOZOID	0.760	0.015	8.0	3.5	3:1	22.0	405.90	553.90	60-MIL HDPE GEOMEMBRANE
PERIMETER REACH B1	TRIANGULAR	0.025	0.027	8.0	1.8	3:1	10.8	30.80	41.50	NATIVE VEGETATION
PERIMETER REACH C1	TRAPEZOID	0.005	0.015	8.0	3.5	3:1	29.0	275.60	376.60	60-MIL HDPE GEOMEMBRANE
PERIMETER REACH C2	TRAPEZOID	0.005	0.015	8.0	3.5	3:1	29.0	275.20	375.40	60-MIL HDPE GEOMEMBRANE
PERIMETER REACH C3	TRAPEZOID	0.005	0.015	8.0	3.5	3:1	29.0	465.10	639.10	60-MIL HDPE GEOMEMBRANE
PERIMETER REACH C4	TRAPEZOID	0.005	0.015	8.0	3.5	3:1	29.0	358.60	577.40	60-MIL HDPE GEOMEMBRANE



7 / 2-1 DETAIL
DRAINAGE CHANNEL
SCALE: N.T.S.



8 / 2-1 DETAIL
TYPICAL DRAINAGE CULVERT
SCALE: NTS

NOTE:

1. CONCRETE HEADWALL WILL BE TEXAS DEPARTMENT OF TRANSPORTATION (TXDOT) STANDARD DETAIL FW-0 AS SHOWN IN ATTACHMENT 2E AND ON THIS SHEET.
2. RIPRAP APRON SHALL BE A MINIMUM OF 2' THICK AND SHALL HAVE D_{50} = 12" AND D_{MAX} = 24"; APRON WIDTH = 24' MIN.

STATE OF TEXAS
SCOTT M. GRAVES
86557
LICENSED PROFESSIONAL ENGINEER
5/7/2020

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REV	DATE	DESCRIPTION	DRN	APP
-	MAY 2020	INITIAL SUBMITTAL TO TCEQ	JJV	SMG

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TITLE: SURFACE WATER MANAGEMENT SYSTEM DETAILS III

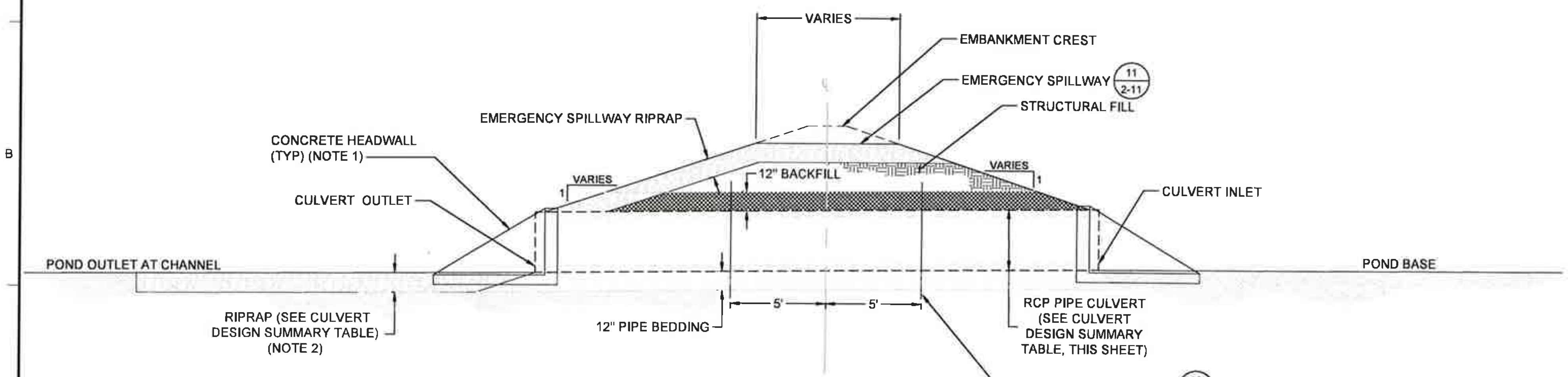
PROJECT: FORT WORTH C&D LANDFILL EXPANSION
PERMIT AMENDMENT APPLICATION - PERMIT NO. MSW-1983D

PROJECT NO.: GW6953	DESIGN BY: BK	REVIEWED BY: BG	PART NO.: III	DRAWING: 2-10
FILE: GW6953P2-13	DRAWN BY: JJV / KH	APPROVED BY: SMG		

PERMIT DRAWING

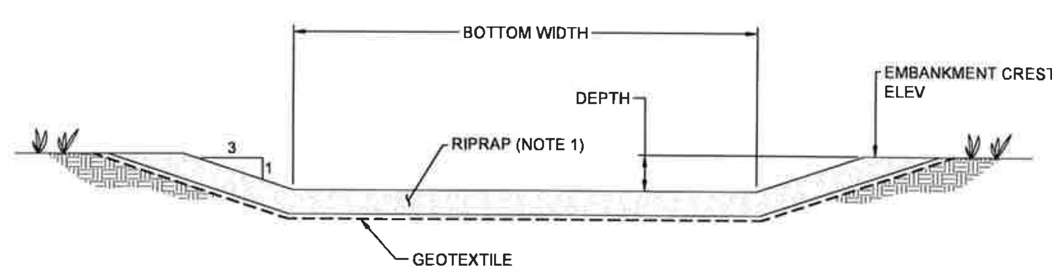
DRAWING: Austin P:\CADD\Projects\Ft worth c&d landfill\PERMIT\landfill expansion (gw6953)\DRAWINGS\GW6953P2-13.dwg PLOTTED: May 05, 2020 - 2:31pm

CULVERT DESIGN SUMMARY TABLE									
CULVERT	25-YEAR, 24-HOUR PEAK FLOW, Q (CFS)	100-YEAR, 24-HOUR PEAK FLOW, Q (CFS)	APPROX. LENGTH (FT)	SLOPE (%)	DESCRIPTION	INLET ELEVATION (FT, MSL)	OUTLET ELEVATION (FT, MSL)	MINIMUM D_{50} RIPRAP SIZE (IN)	MINIMUM RIPRAP THICKNESS (FT)
Culvert 1	275.20	375.40	127.00	0.51	3-4'X4.5' BOX	639.46	638.81	5	1.0

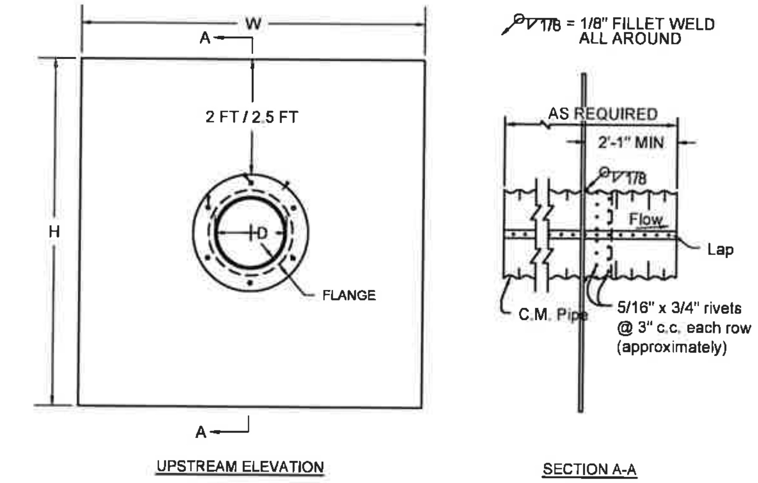


9 / 2-11 DETAIL
SURFACE WATER POND OUTLET
SCALE: NTS

SUMMARY OF EMERGENCY SPILLWAY DESIGN PARAMETERS			
SPILLWAY LOCATION	BOTTOM WIDTH (FT)	DEPTH (FT)	EMBANKMENT CREST ELEV (FT, MSL)
NORTH POND SERIES 1	80	1.5	598.5
NORTH POND SERIES 2		1.0	604.0
NORTH POND SERIES 3		1.0	614.0
NORTH POND SERIES 4		1.0	625.0



11 / 2-11 DETAIL
EMERGENCY SPILLWAY
SCALE: NTS



CULVERT	PIPE DIAM (D) (INCHES)	SHEET DIAM (H) (INCHES)	SHEET DIAM (D) (INCHES)
NORTH POND SERIES 1	30	78	78 (2 FT VERT & HORIZ PROJECTION)
NORTH POND SERIES 2	54	102	102 (2 FT VERT & HORIZ PROJECTION)
NORTH POND SERIES 3	54	108	108 (2 FT VERT & HORIZ PROJECTION)
NORTH POND SERIES 4	54	102	102 (2 FT VERT & HORIZ PROJECTION)

NOTES:

1. MAKE PIPE CONNECTIONS AS NEEDED TO ASSURE A WATERTIGHT SYSTEM.
2. APPLY SILICON CAULK ON THE SEAMS AS NEEDED TO INSURE A GOOD SEAL SO THAT THE COMPLETED INSTALLATION IS WATERTIGHT.

10 / 2-11 DETAIL
ANTI-SEEP COLLAR
SCALE: N.T.S.

NOTE:

1. CONCRETE HEADWALL WILL BE TEXAS DEPARTMENT OF TRANSPORTATION (TXDOT) STANDARD DETAIL FW-0 AS SHOWN IN ATTACHMENT 2E AND ON THIS SHEET.
2. RIPRAP APRON SHALL BE A MINIMUM OF 2' THICK AND SHALL HAVE D_{50} = 12" AND D_{MAX} = 24"; APRON WIDTH = 24' MIN.

STATE OF TEXAS
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5/7/2020

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REV	DATE	DESCRIPTION	DRN	APP

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TITLE: SURFACE WATER MANAGEMENT SYSTEM DETAILS IV

PROJECT: FORT WORTH C&D LANDFILL EXPANSION
PERMIT AMENDMENT APPLICATION - PERMIT NO. MSW-1983D

PROJECT NO.: GW6953	DESIGN BY: BK	REVIEWED BY: BG	PART NO.: III	DRAWING: 2-11
FILE: GW6953P2-15	DRAWN BY: JJV / KH	APPROVED BY: SMG		

PERMIT DRAWING

DRAWING: Austin P:\CADD\Projects\Ft worth c&d landfill\PERMIT\landfill expansion (gw6953)\DRAWINGS\GW6953P2-15.dwg PLOTTED: May 05, 2020 - 2:31pm

APPENDIX IIIF-F

EROSION CONTROL PLAN FOR ALL PHASES OF LANDFILL OPERATION

Includes pages IIIF-F-1 through IIIF-F-15

STATE OF TEXAS

CHARLES R. MARSH

105073

LICENSED

PROFESSIONAL ENGINEER

02/09/2023

CONTENTS

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02/09/2023

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 the 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 Title 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., drainage swales, letdown structures, and/or sedimentation ponds) 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 currently TCEQ-approved Site Development Plan (SDP).

Inspection, maintenance, and recordkeeping requirements are included in the Site Operating Plan (SOP). 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 Site Development Plan. Additionally, "temporary" is defined as the time between construction of intermediate cover and the construction of final cover. Temporary erosion controls are those controls which may be installed or constructed within 180 days from when the intermediate cover is constructed and in place until permanent controls are constructed for the final cover.

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 may 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.

The structural controls may 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 Sheet IIIF-F-10, stormwater runoff will be collected in swales and conveyed to drainage letdown structures down the 33.3 percent slope to the perimeter drainage system. The primary goal will be to establish the vegetative cover percentage and swale spacing distance indicated in the swale design summary table on Sheet IIIF-F-11 on all external top dome surfaces and external embankment slopes. These criteria will result in a net soil loss of 50 tons/acres/year or less for each drainage swale and letdown combination specified on Sheets IIIF-F-10 and IIIF-F-11 (refer to Section 2.1 for additional information).

Mulch, woodchips, compost or straw/hay 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 straw/hay may be used to stabilize recently graded or seeded areas. If needed, the mulch, woodchips, compost or straw/hay 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 or straw/hay will be placed so as not to inhibit the growth of vegetation. In the event that the indicated vegetative ground cover required for a specific swale spacing distance is not obtained within 180 days after intermediate cover is placed on a top dome or external side slope, mulch, woodchips, compost or straw/hay may be used as a secondary measure to limit soil loss to 50 tons/acre/year or less until vegetation is established. In the above referenced cases, other erosion protection measures will only be used upon prior written authorization by TCEQ (e.g., permit modification). Stormwater discharge from the site must comply with the current TPDES for the site. The discharge locations for the site are identified in Appendix IIIF as a part of the final drainage design and cannot be revised based on this ECP. Design and use of temporary erosion control measures cannot result in offsite discharge exceeding the peak flow rates, volumes, or velocities listed in Table 4-1 of Appendix IIIF.

As an alternative to mulch, wood chips, compost, or straw/hay, a detention/ sedimentation pond may be used as a secondary measure to limit the discharge of eroded soil loss to 50 tons/acre/year or less (refer to Section 2.2 for additional information) if the required 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 (e.g., 60 percent vegetation on the external embankment slopes and top dome surfaces).

2.1 Drainage Swale and Letdown Structure Requirements

Sheet IIIF-F-10 shows a typical layout for erosion control structures, including temporary add-on swales and drainage letdowns. Sheet IIIF-F-11 provides a swale design summary, which includes spacing and vegetative cover requirements for the swales. Supporting calculations for the specifications listed on Sheet IIIF-F-11 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/sec and sheet flow velocity for "nearly bare ground" is less than 3.5 ft/sec (consistent with Title 30 TAC §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). Sheet IIIF-F-12 shows letdown details and the letdown design summary. As shown on Sheet IIIF-F-12, the letdowns will 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 Sheet IIIF-F-12. As noted on Sheet IIIF-F-12, the use of pipe letdowns will be limited to 1 inlet per letdown.

As noted on Sheet IIIF-F-10, the acceptable soil loss is determined for each acre on the top dome surfaces and external embankment side slopes. The soil loss for top dome surfaces and external embankment side slopes will vary depending on swale spacing and percent vegetative cover (refer to Sheet IIIF-F-11 for soil loss estimates). If certain percent vegetation cover is not achieved, a sediment control pond will be temporarily used for sediment capture to reduce the discharge of eroded soil from the external slopes to a rate that is equal to or less than 50 tons/acre/year. The swale spacing as shown on Sheet IIIF-F-11 for top dome and side slope surfaces is based on the limiting soil loss of 50 tons/acres/year. If a vegetative coverage and swale spacing configuration results in a soil loss greater 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.

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. Numerous erosion control structures have been installed at the site that conform to the requirements of this ECP, and these structures will remain in place and continue to serve as erosion control measures until they are decommissioned.

As stated previously, the primary goal is to obtain the required vegetation coverage percentage for each condition (e.g., swale spacing).

2.2 Sedimentation Pond Design

As noted on Sheets IIIF-F-10 and IIIF-F-11, if vegetative 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. Sheet IIIF-F-13 provides a procedure for determining the required pond size. Supporting calculations for the procedure listed on Sheet IIIF-F-13 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. This document will also include a statement that notes how the temporary sedimentation pond, the pond outlet, and any related perimeter channels were constructed consistent with the requirements of the Site Development Plan. Sheet IIIF-F-14 shows the different options for typical pond outlet structures.

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 required 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 the required vegetation coverage on the 20-acre area. If in early June it becomes apparent that the percent vegetation will be less than the required coverage on June 29, then the operator may install a sedimentation pond downstream of the 20-acre area, consistent with the requirements shown on Sheet IIIF-F-13. 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 required percent vegetation specification is met.

If a sedimentation pond is used as a source to maintain a 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.

As stated previously, the primary goal 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 29th date used in the above example). Once the required vegetation percentage is achieved, then the sedimentation pond will no longer be needed (but may remain in-place as an additional BMP until the site reaches the permitted final configuration). If the percent vegetation does not meet the required 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) used in conjunction with the above erosion control measures are listed below.

- Check Dams – These structures will be used in channels to slow down flow velocities and improve sediment capture.
- Silt Fences – These structures will 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.
- Erosion Booms – These structures may be used in capturing sediment and for diversion of flow for controlling sediment discharge.

These erosion control measures will be used on slopes to help control erosion loss. Rock check dams will be used in the detention/sedimentation pond. Refer to Sheet IIIF-F-15 for details of typical BMPs.

Nonstructural controls that will be used at the site to minimize erosion loss include: plans and designs to minimize disruption of the natural features, drainage, topography, and 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; 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. Other BMPs will only be utilized upon prior written authorization (e.g., permit modification) by TCEQ.

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 structural erosion control features and 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. Structural erosion control measures consist of drainage swales, letdown structures, and detention ponds.

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, and letdowns installed). Inspection requirements and schedules are listed in the SOP for all drainage features, including intermediate cover areas. If the required percent vegetation coverage is not achieved within the 180-day period, secondary erosion control measures such as mulch, wood chips or compost will be used to limit the soil loss to the 50 tons/acre/year or less. Other erosion protection measures will only be utilized upon prior written authorization (e.g., permit modification) by TCEQ. In addition, a detention/sedimentation pond may also be used until the required 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 the required vegetation cover is achieved and the date that the secondary measure is no longer needed will also be documented in the Site Operating Record. The dates and locations of installation of erosion and sediment control 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. Inspection and maintenance of the erosion and sediment control structures of the top dome surfaces and external embankment side slopes will follow the same schedule and methods as described in Section 4.24 of the facility's SOP.

For example, as stated in Section 4.18.3 of the current Site Operating Plan (SOP), intermediate cover areas are inspected weekly and within 72 hours of a rainfall event of 0.5 inches or more, or as soon as the areas are accessible, for proper placement, thickness, erosion, and compaction. Additionally, Section 4.23 of the SOP also requires inspections of perimeter channels and ponds to ensure they are functioning as designed (e.g., excess sediment removed, outlet structures intact, and erosion control measures intact, etc.) on a weekly basis and after a rainfall event of 0.5 inches or more, or as soon as the areas are accessible.

During the inspection of structural controls (e.g., vegetation over intermediate cover areas), if significant soil loss is identified in 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 cover areas will be borrowed from sedimentation ponds or any other soil source. If sediment collected from wet retention pond(s) (e.g., Pond or temporary sedimentation ponds) is used for erosion layer replenishment, it will be stockpiled outside the ponds to dry out prior to being used for intermediate cover layer replenishment. Soil borrowed from other soil sources may be used as intermediate cover layer and erosion layer replenishment soil.

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 required 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. In accordance with Title 30 TAC §330.165(g), previously stabilized surfaces will be repaired within 5 days of detection of the disturbance of these surfaces.

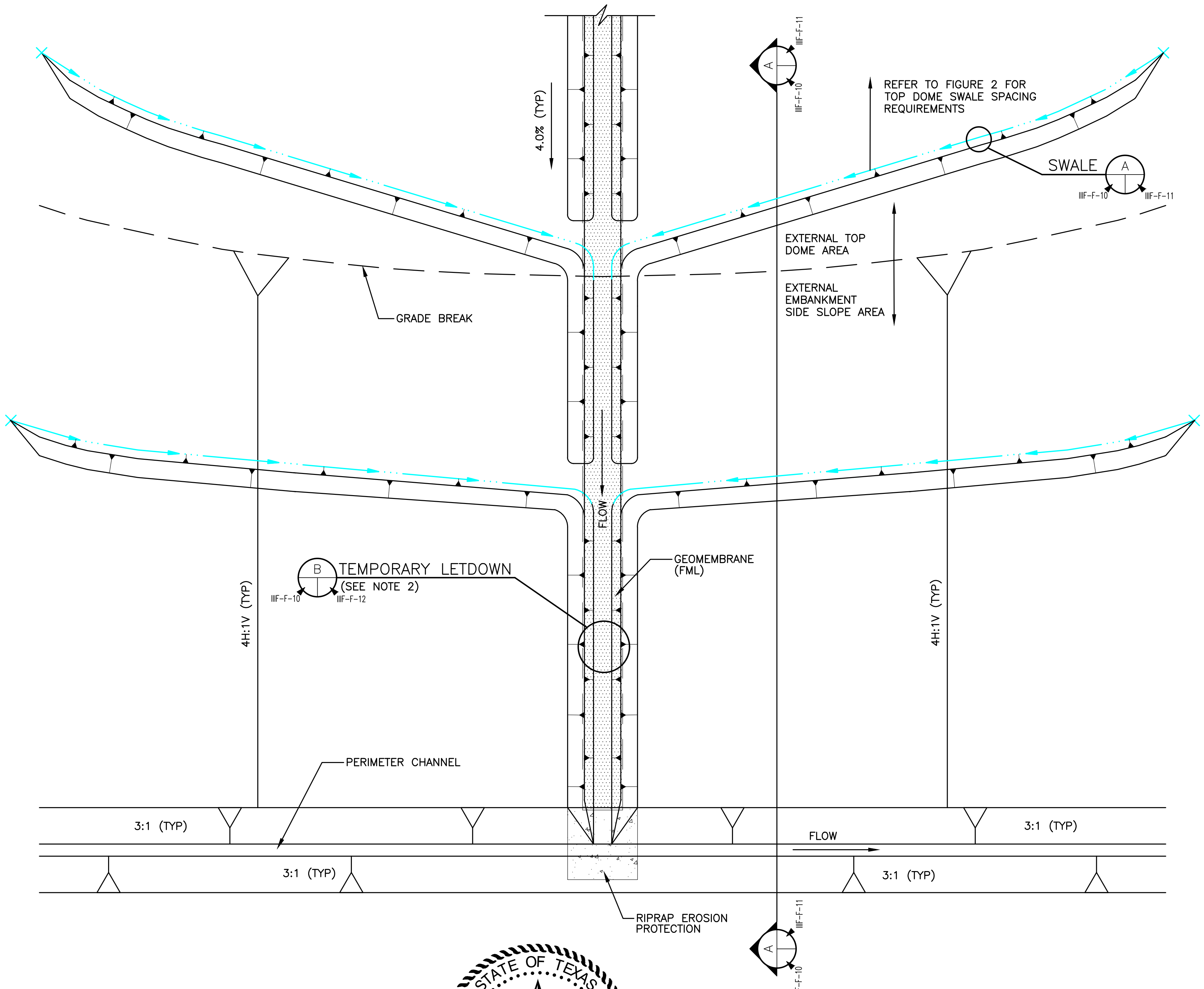
3.0 Erosion Control Plan for Daily Cover Areas and Intermediate Cover Areas for Non-External Side Slopes

BMPs will be employed to control erosion. BMPs will 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 Appendix IIIA-A of the Site Development Plan. 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 vegetative 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 Section 2.3 of Appendix IIIF.
- Erosion will be controlled by vegetation in drainage structures with flow velocities less than or equal to 5 ft/sec. For drainage structures with flow velocities greater than 5 ft/sec, rock riprap or gabions will be used for surface reinforcement. Other erosion protection measures will only be utilized upon prior written authorization (e.g., permit modification) by TCEQ.

Typical erosion control features are shown on Sheet IIIF-F-15. Inspection items and schedules are listed in the SOP for all drainage features, daily cover, and intermediate cover areas.

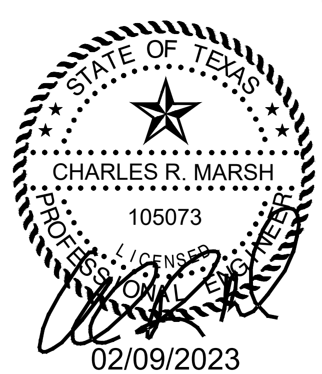


LEGEND

- 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 SHEET IIIF-F-11 FOR SOIL LOSS ESTIMATES).
2. TEMPORARY LETDOWN IS SHOWN AS AN OPEN CHANNEL WITH A GEOMEMBRANE LINER. AS NOTED ON SHEET IIIF-F-12, OTHER CHANNEL LININGS MAY BE USED (e.g., GABIONS, GROUT, 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 SHEET IIIF-F-12.



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DESIGN BY: BPY
REVIEWED BY: CRM

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

PREPARED FOR
TEXAS REGIONAL LANDFILL COMPANY, LP

REVISIONS

NO.	DATE	DESCRIPTION

EROSION CONTROL PLAN
TYPICAL EROSION CONTROL STRUCTURE LAYOUT

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

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SHEET IIIF-F-10

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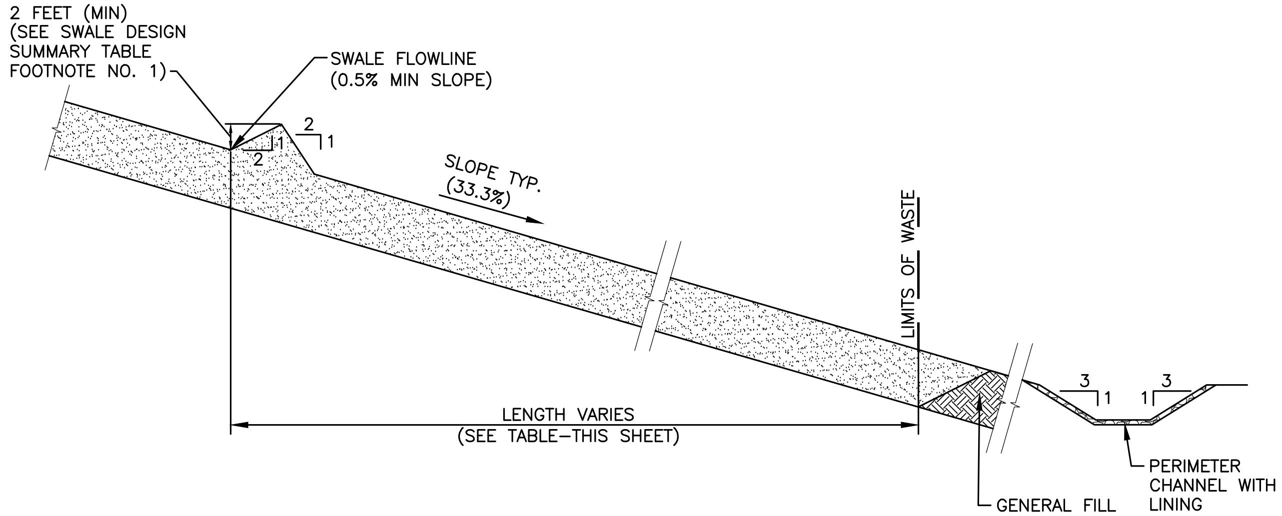
O:\0771\356\EXPANSION 2022\PART III\IIIF\IIIF-F\IIIF-F-10-TYPICAL SWALE SPACING.dwg, jwilson, 1:2



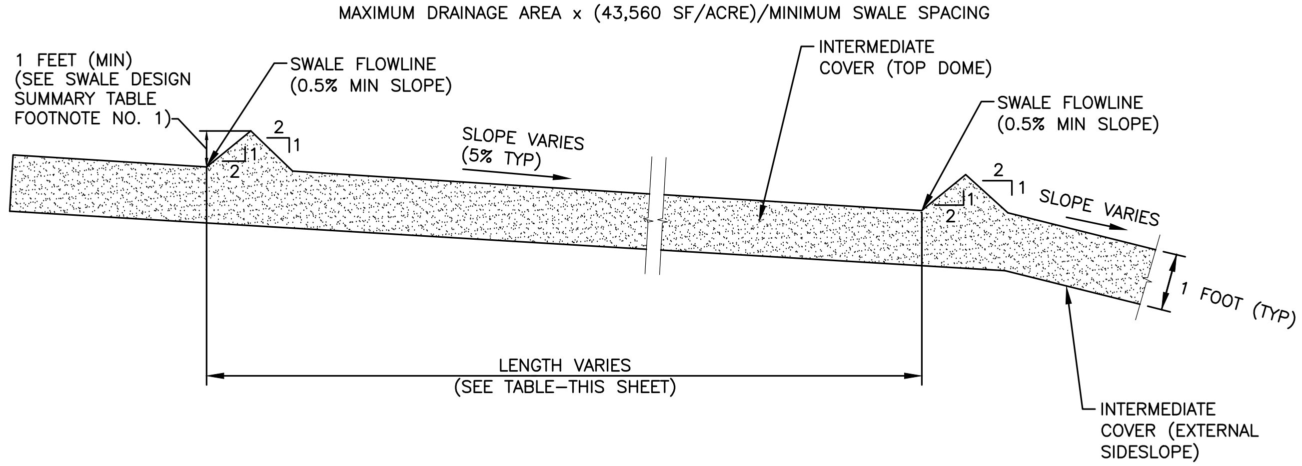
A SIDE SLOPE DRAINAGE SWALE
IIIF–F–10 IIIF–F–11

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, 5%)	28.4	200	6,176
TOP SLOPE (1.5 FT SWALE, 5%)	13.2	200	2,870
TOP SLOPE (1 FT SWALE, 5%)	4.5	200	973
SIDE SLOPE (2 FT SWALE, 33.3%)	6.2	105	2,553

THE MINIMUM SWALE SPACING IS USED TO DETAIN THE MAXIMUM SWALE LENGTH GIVEN THAT THE AREA IS FIXED. MINIMUM SWALE SPACING IS OBTAINED FROM THE CALCULATIONS PROVIDED ON PAGE IIIF–F–1–10.

[2] MAXIMUM SWALE LENGTH CALCULATED USING THE FOLLOWING EQUATION:

MAXIMUM DRAINAGE AREA x (43,560 SF/ACRE)/MINIMUM SWALE SPACING



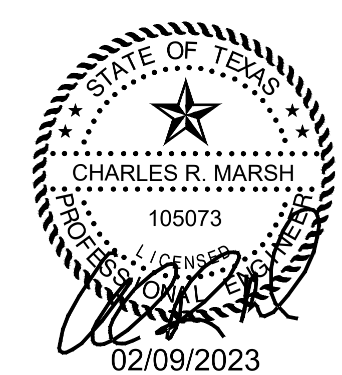
A TOP DOME SURFACE DRAINAGE SWALE
IIIF–F–10 IIIF–F–11

SWALE DESIGN SUMMARY[1]							
TOP SLOPE (5%)				SIDE SLOPE (33.3%)			
VEGETATIVE COVER PERCENTAGE	DISTANCE BETWEEN SWALES[3] (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	200	1.8	NO	60	105	23.1	NO
70	200	0.7	NO	70	105	9.4	NO
80	200	0.6	NO	80	105	7.2	NO
90	200	0.3	NO	90	105	3.5	NO
60	500	2.9	NO	60	200	31.7	NO
70	500	1.2	NO	70	200	12.8	NO
80	500	0.9	NO	80	200	9.8	NO
90	500	0.4	NO	90	200	4.8	NO
60	700	3.4	NO	60	300	48.7	NO
70	700	1.4	NO	70	300	19.7	NO
80	700	1.1	NO	80	300	15.1	NO
90	700	0.5	NO	90	300	7.4	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 120 FEET FOR SIDE SLOPES AND A DISTANCE OF 200 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 IS LESS THAN 50 TONS/ACRE/YEAR AND THE TOP SLOPE IS LESS THAN 50 TONS/ACRE/YEAR.

[3] NUMBERS INDICATE THE MAXIMUM SWALE SPACING FOR A GIVEN VEGETATIVE COVER PERCENTAGE.



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DESIGN BY: BPY
REVIEWED BY: CRM

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TBPE REGISTRATION NO. F–3727

PREPARED FOR
TEXAS REGIONAL LANDFILL COMPANY, LP

REVISIONS		
NO.	DATE	DESCRIPTION

EROSION CONTROL PLAN
SWALE DESIGN SUMMARY

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

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SHEET IIIF–F–11

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OPEN CHANNEL GEOMEMBRANE LETDOWN DESIGN SUMMARY

DESIGN IS APPLICABLE FOR A DRAINAGE AREA UP TO 30.0 ACRES (TOP DECK AND SIDE SLOPE).

33.3% SLOPE
MAXIMUM FLOW DEPTH = 0.41 FT.
BOTTOM WIDTH = 8 FT.

5% SLOPE
MAXIMUM FLOW DEPTH = 0.72 FT.
BOTTOM WIDTH = 8 FT.

OPEN CHANNEL GABION AND ROCK RIPRAP LETDOWN DESIGN SUMMARY

DESIGN IS APPLICABLE FOR A DRAINAGE AREA UP TO 30.0 ACRES (SIDE SLOPE AND TOP DECK).

33.3% SLOPE
MAXIMUM FLOW DEPTH = 0.93 FT.
BOTTOM WIDTH = 8 FT.

5% SLOPE
MAXIMUM FLOW DEPTH = 1.58 FT.
BOTTOM WIDTH = 8 FT.

OPEN CHANNEL GROUTED RIPRAP LETDOWN DESIGN SUMMARY

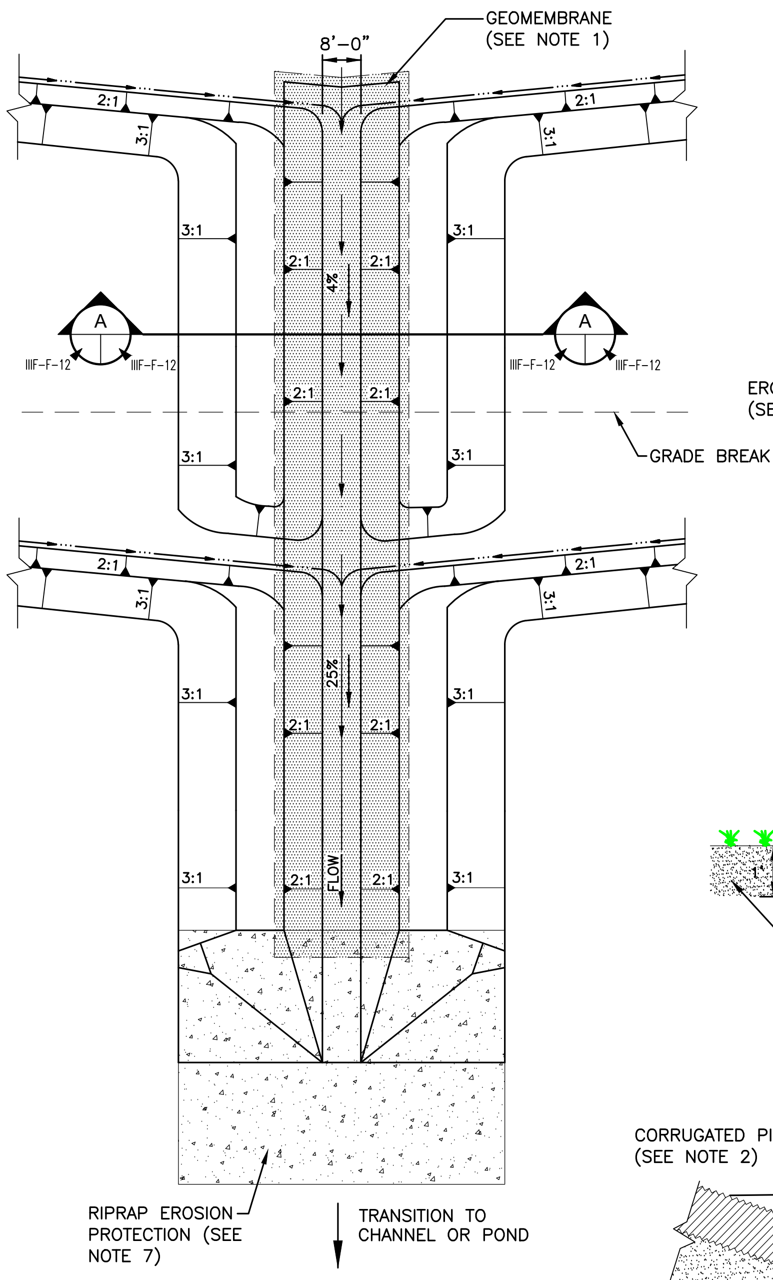
DESIGN IS APPLICABLE FOR A DRAINAGE AREA UP TO 30.0 ACRES (TOP DECK AND SIDE SLOPE) AND 5% TOP DECK.

33.3% SLOPE
MAXIMUM FLOW DEPTH = 0.79 FT.
BOTTOM WIDTH = 8 FT.

5% SLOPE
MAXIMUM FLOW DEPTH = 1.34 FT.
BOTTOM WIDTH = 8 FT.

OPEN CHANNEL TURF REINFORCEMENT LETDOWN DESIGN SUMMARY

SEE GROUTED RIPRAP LETDOWN DESIGN.



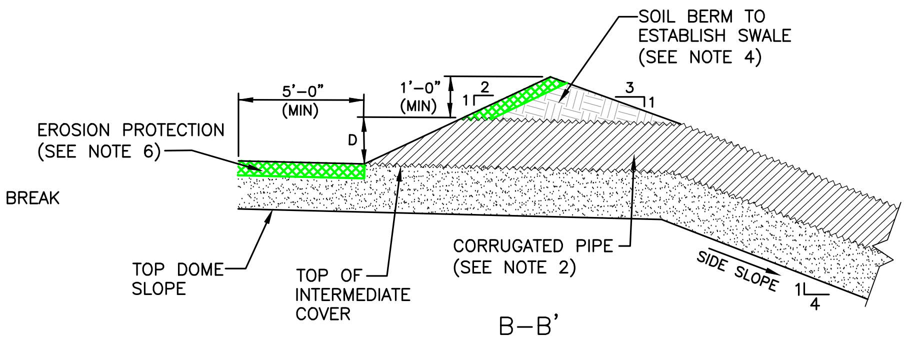
TEMPORARY OPEN CHANNEL LETDOWN
B IIIF-F-10 IIIF-F-12

0 20 40
SCALE IN FEET

33.3% PIPE LETDOWN DESIGN SUMMARY*
(USE OF PIPE LETDOWN IS LIMITED TO 1-INLET)

DRAINAGE AREA (ACRE)	DESIGN FLOW RATE (CFS)	REQUIRED PIPE DIAMETER (FT)
4.2	22.6	2
5.9	32.0	3

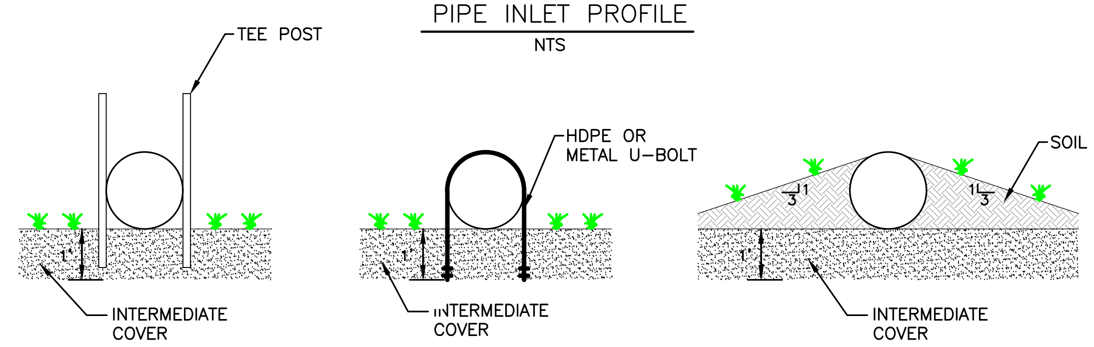
* SEE NOTE 4



B-B'

PIPE INLET PROFILE

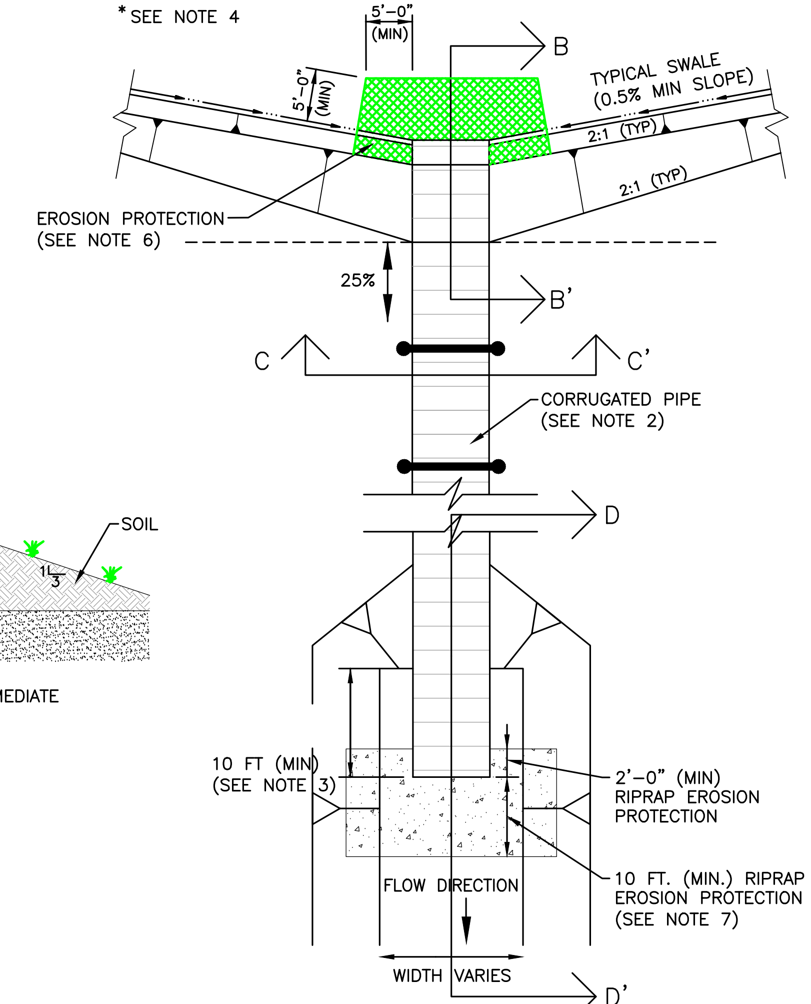
NTS



C-C'

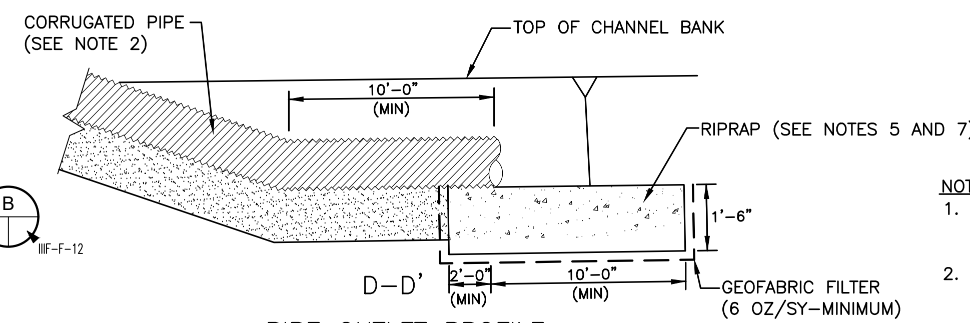
ANCHORING DETAILS

NTS



TEMPORARY PIPE LETDOWN

NTS



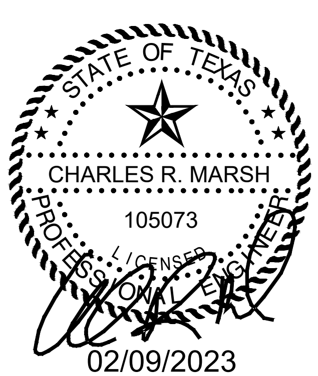
D-D'

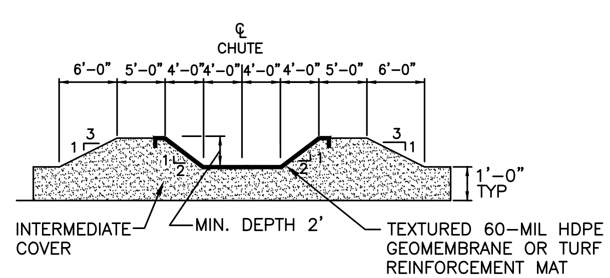
PIPE OUTLET PROFILE
(SEE NOTE 3)

NTS

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 OR 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-2-24 FOR HYDRAULIC ANALYSIS.
5. RIPRAP APRON DESIGN IS PROVIDED ON PAGES IIIF-F-2-36 AND 37. D_{50} FOR RIPRAP 8-INCHES IS MINIMUM.
6. RIPRAP, GROUT, GROUTED RIPRAP, GABIONS, GEOMEMBRANE, EXISTING VEGETATION OR TURF REINFORCEMENT MAY BE USED FOR EROSION PROTECTION.
7. OTHER EROSION PROTECTION (e.g., RIPRAP, GROUT, GROUTED RIPRAP, GABIONS OR TURF REINFORCEMENT) MAY BE USED AT TEMPORARY LETDOWN OUTFALLS.
8. REFER TO PAGE IIIF-F-2-37 FOR EROSION PROTECTION DESIGN. IF LETDOWN DISCHARGES TO A POND, 10 FEET OF RIPRAP WILL BE SUFFICIENT.

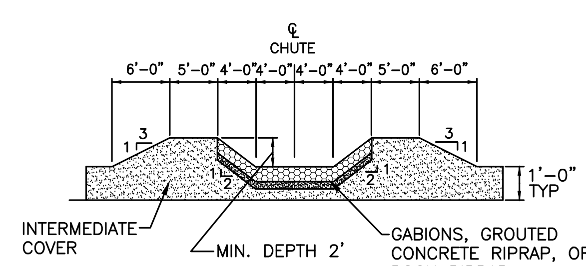




A1 IIIF-F-12 IIIF-F-12

TEMPORARY GEOMEMBRANE OR TURF REINFORCEMENT LETDOWN OPTION

0 10 20
SCALE IN FEET



A2 IIIF-F-12 IIIF-F-12

TEMPORARY GABION LETDOWN OPTION

0 10 20
SCALE IN FEET

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FILE: 0771-356-11 DESIGN BY: BPY
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REVISIONS

NO.	DATE	DESCRIPTION

Weaver Consultants Group
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EROSION CONTROL PLAN
LETDOWN DESIGN SUMMARY

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

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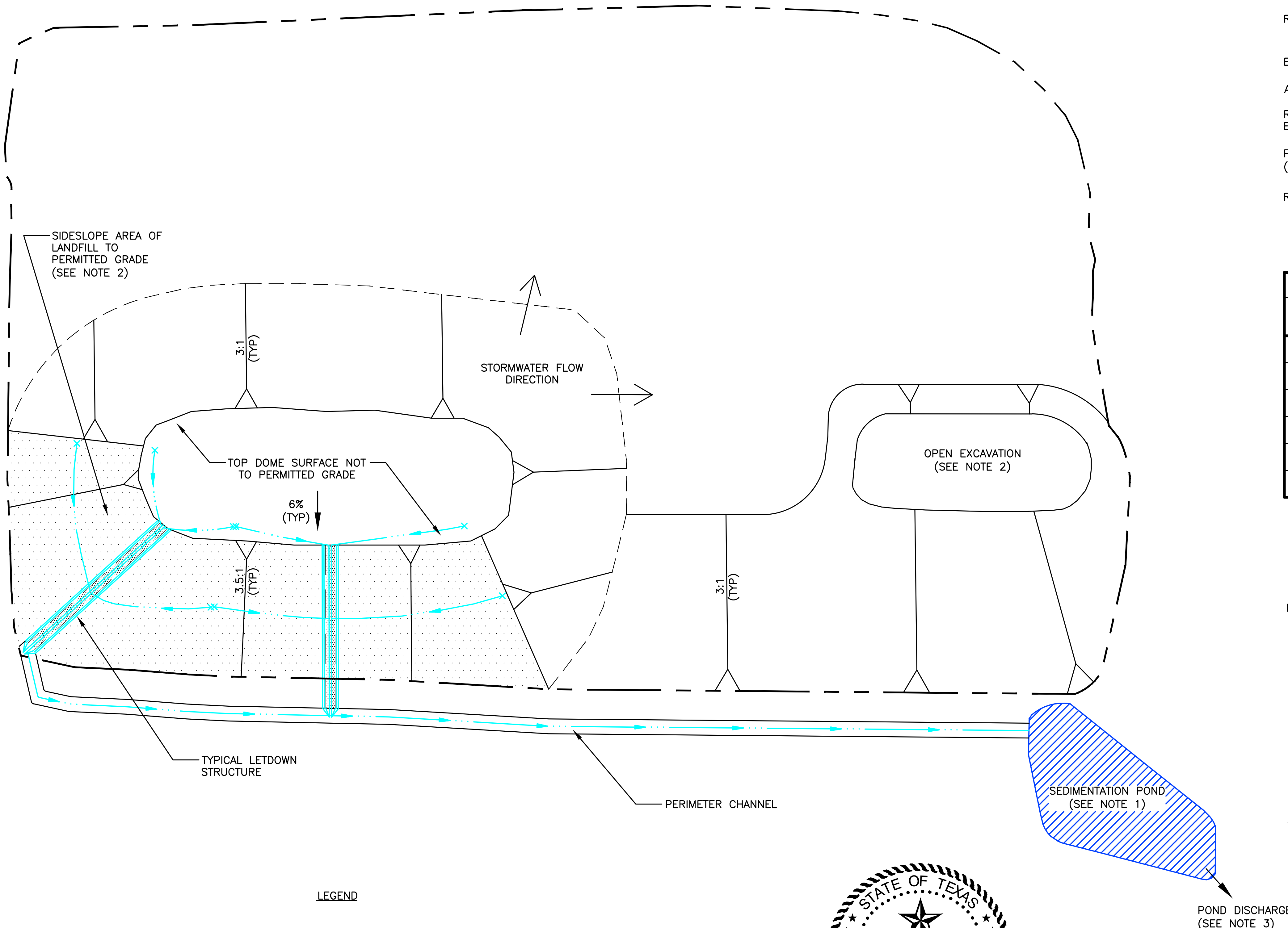

EXAMPLE CALCULATION

REQUIRED POND SIZE = EXTERNAL EMBANKMENT AREA (ACRES) X POND AREA REQUIRED/ UNIT DRAINAGE AREA FACTOR

EXTERNAL EMBANKMENT AREA DRAINING TO POND = 20 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) = 0.060

REQUIRED POND SIZE = 20 ACRES X 0.060 = 1.20 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	0.025	13.3%
70 TO 50	0.040	25.5%
80 TO 50	0.060	34.0%
90 TO 50	0.075	41.5%
100 TO 50	0.110	46.4%
200 TO 50	0.300	71.2%

[1] REFER TO APPENDIX IIIF-F-3 FOR MORE INFORMATION. THE POND DESIGN AND DEMONSTRATION ARE PROVIDED TO ENSURE THAT SEDIMENT DISCHARGE FROM THE SITE WILL BE PREVENTED DURING INITIAL ESTABLISHMENT OF VEGETATION OVER THE SIDE SLOPES 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.
3. AS STATED IN SECTION 2.2, A STATEMENT WILL BE ADDED TO THE SITE OPERATING RECORD EACH TIME A SEDIMENTATION POND IS INSTALLED TO NOTE HOW THE TEMPORARY SEDIMENTATION POND AND THE POND OUTLET WERE CONSTRUCTED CONSISTENT WITH THE REQUIREMENTS OF THE SITE DEVELOPMENT PLAN.

LEGEND

EXTERNAL SIDE SLOPE TO PERMITTED GRADE

LETDOWN

DRAINAGE SWALE

STORMWATER FLOW DIRECTION

STATE OF TEXAS
CHARLES R. MARSH
105073
LICENSED
PROFESSIONAL ENGINEER
02/09/2023

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DRAWN BY: RAA
DESIGN BY: BPY
REVIEWED BY: CRM

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

PREPARED FOR
TEXAS REGIONAL LANDFILL COMPANY, LP

REVISIONS

NO.	DATE	DESCRIPTION

EROSION CONTROL PLAN
SEDIMENT CONTROL POND PLAN

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

WWW.WCGRP.COM

SHEET IIIF-F-13

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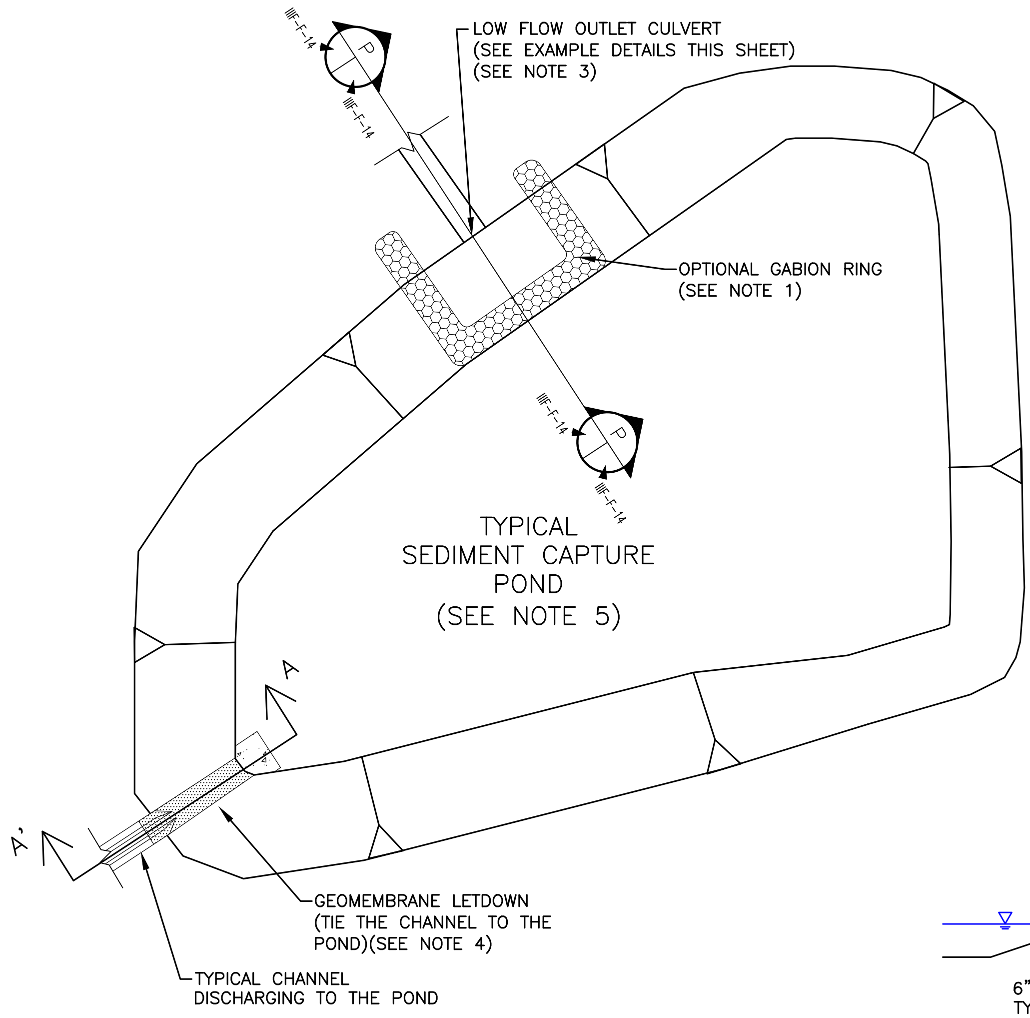
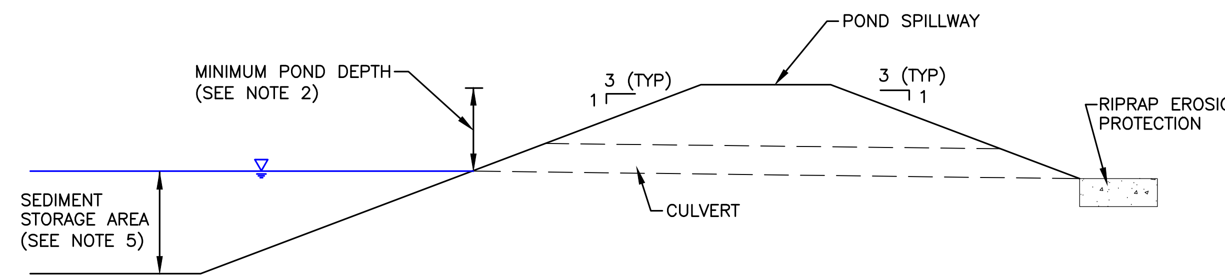
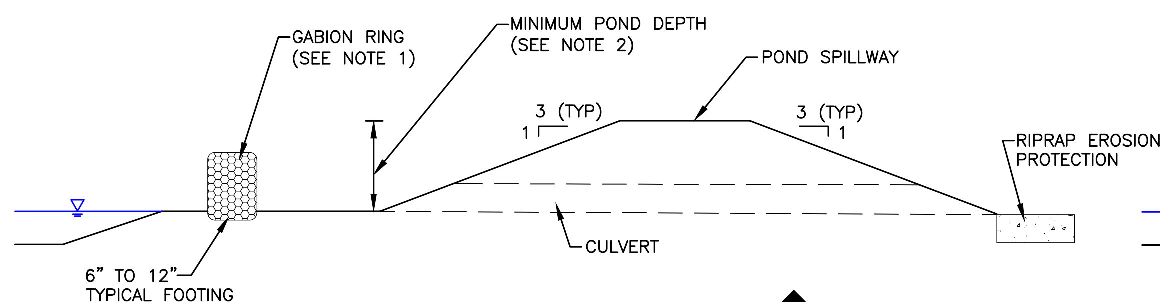
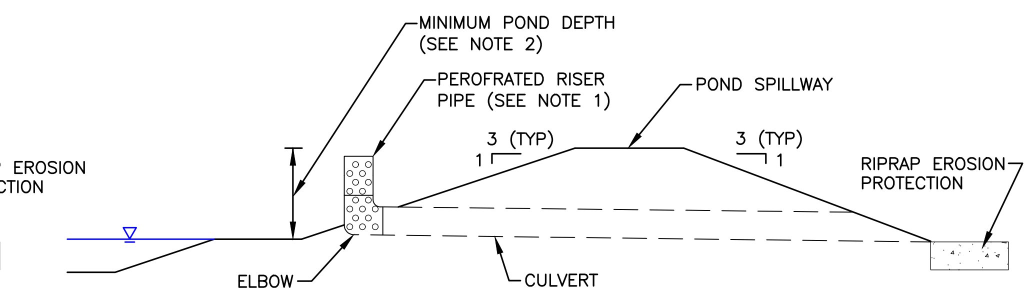




OUTLET STRUCTURE FOR PERMANENT POOL



GABION RING

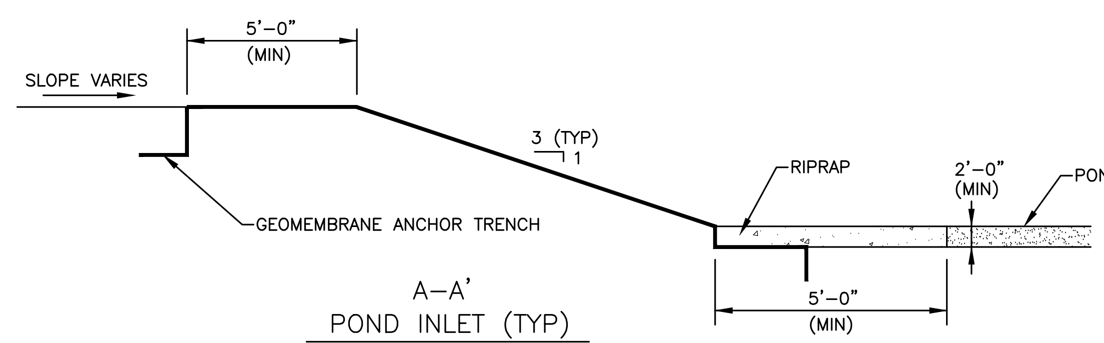


PERFORATED RISER PIPE

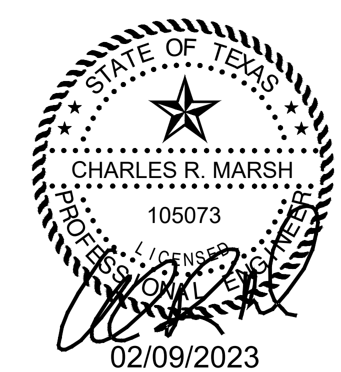
OPTIONS FOR DRY PONDS

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 BETWEEN THE LOW FLOW OUTLET FLOW LINE AND POND SPILLWAY ELEVATION.
3. IF THE POND IS INSTALLED WITHOUT A LOW FLOW OUTLET, THEN SEE NOTE 2 ON SHEET IIIF-F-13.
4. VEGETATIVE SURFACING, GROUTED RIPRAP, RIPRAP, GABIONS, OR TURF REINFORCEMENT MAY BE USED TO ENSURE THE STABILITY OF THE POND INLET.
5. POND BOTTOM AREAS WILL BE EXCAVATED BELOW THE LOW FLOW OUTLET FLOW LINE ELEVATION TO PROVIDE SEDIMENT STORAGE. SEDIMENT ACCUMULATED IN POND WILL BE REMOVED AS NEEDED TO ENSURE SEDIMENT STORAGE CAPACITY BELOW THE FLOWLINE ELEVATION OF LOW FLOW OUTLET (REFER TO SECTION 2.4 FOR ADDITIONAL INFORMATION REGARDING SEDIMENT REMOVAL).



A-A'
POND INLET (TYP)
NTS



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DATE: 12/2022 FILE: 0771-356-11 CAD: IIIF-F-14-EROS. CONTROL.DWG	DRAWN BY: RAA DESIGN BY: BPY REVIEWED BY: CRM

REVISIONS

NO.	DATE	DESCRIPTION

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

EROSION CONTROL PLAN
TYPICAL POND OUTLET STRUCTURES

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

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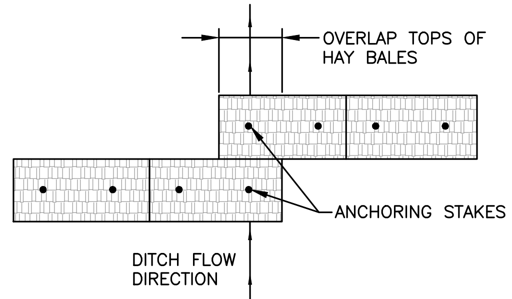
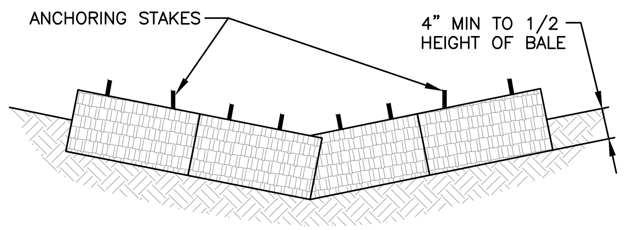
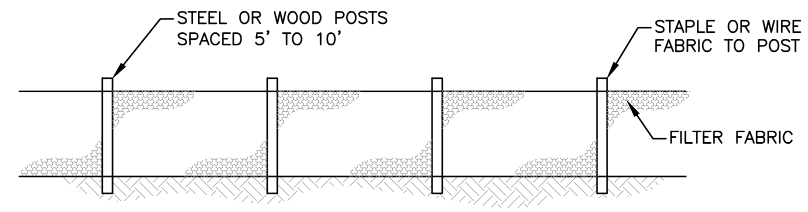


PLAN VIEW

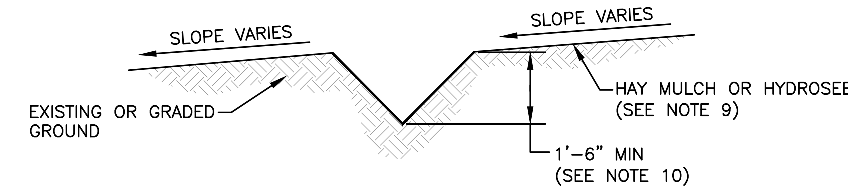


PROFILE VIEW

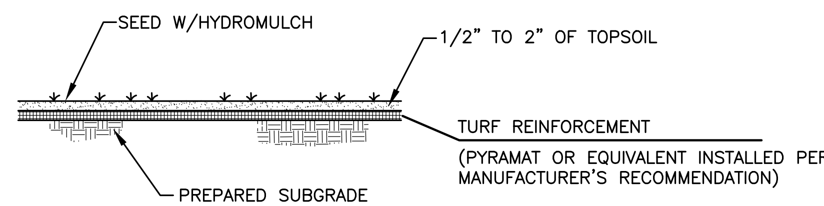
BALED HAY FOR EROSION CONTROL
NTS
(SEE NOTE 1)



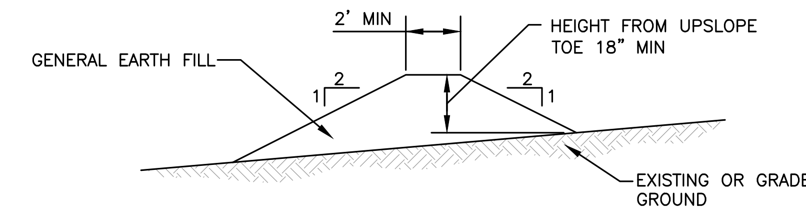
TEMPORARY SEDIMENT CONTROL FENCE
NTS
(SEE NOTE 3)



TEMPORARY DIVERSION CHANNEL
NTS
(SEE NOTE 6)

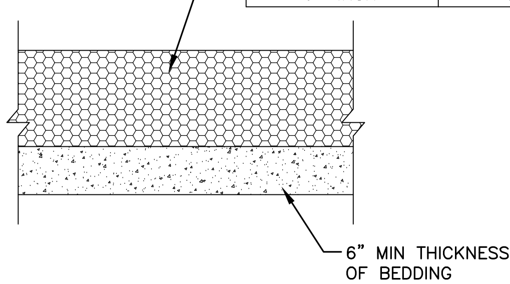


TURF REINFORCEMENT
NTS
(SEE NOTE 4)

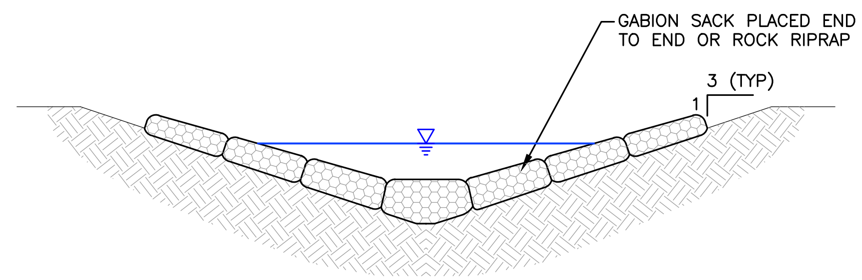


TEMPORARY DIVERSION BERM
NTS
(SEE NOTE 7)

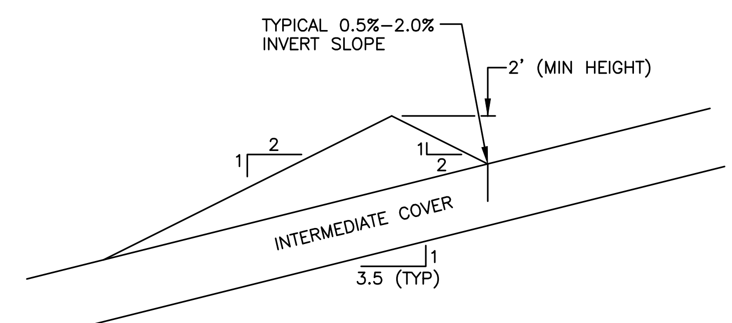
24" THICKNESS OF RIPRAP	
SIEVE SIZE SQUARE MESH	PERCENT PASSING
30 INCH	100
24 INCH	65–100
18 INCH	45–75
12 INCH	25–50
8 INCH	10–30
6 INCH	0–15



ROCK RIPRAP
NTS
(SEE NOTE 2)



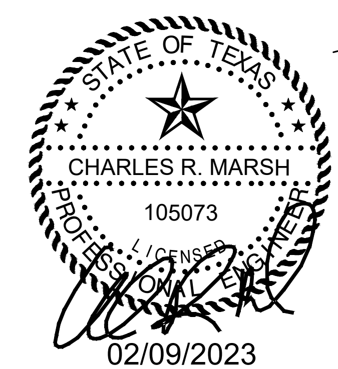
ROCK CHECK DAM
NTS
(SEE NOTE 5)



SWALE
NTS
(SEE NOTE 8)

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 MAY VARY.



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DESIGN BY: BPY
REVIEWED BY: CRM

Weaver Consultants Group
TBPE REGISTRATION NO. F-3727

PREPARED FOR
TEXAS REGIONAL LANDFILL COMPANY, LP

REVISIONS

NO.	DATE	DESCRIPTION

EROSION CONTROL PLAN
TYPICAL BMPs

FORT WORTH C&D LANDFILL
TARRANT COUNTY, TEXAS

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SHEET IIIF-F-15

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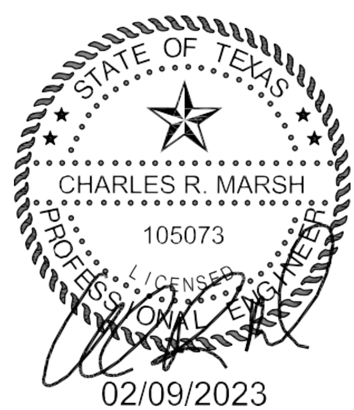
APPENDIX IIIF-F-1

TEMPORARY ADD-ON SWALE DESIGN

Includes pages IIIF-F-1-1 through IIIF-F-1-12



SWALE DESIGN

This appendix includes the expected soil loss calculations for various swale spacing intervals on the side slopes and top dome surfaces. An example calculation is provided on pages IIIF-F-1-2 through IIIF-F-1-4 for a vegetative cover of 60 percent. For the results of various percent vegetative covers and swale spacing intervals, refer to the table on page IIIF-F-1-5 and to Sheet IIIF-F-10 – Swale Design Summary. If the required percent vegetation coverage is not achieved within the 180-day period, secondary erosion control measures such as mulch, wood chips, compost or straw/hay will 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 required 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 the required 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 in 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 Johnson County is:

R = 290

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 intermediate soil will consist of soils comparable to clay. Additionally, compost will be added to intermediate soil 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 = 0.25

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. Top Slope
slope = 5 %
length = 200 ft

Case 2. Top Slope
slope = 5 %
length = 500 ft

Case 3. Top Slope
slope = 5 %
length = 700 ft

Case 4. Side Slope
slope = 33.3 %
length = 105 ft

Case 5. Side Slope
slope = 33.3 %
length = 200 ft

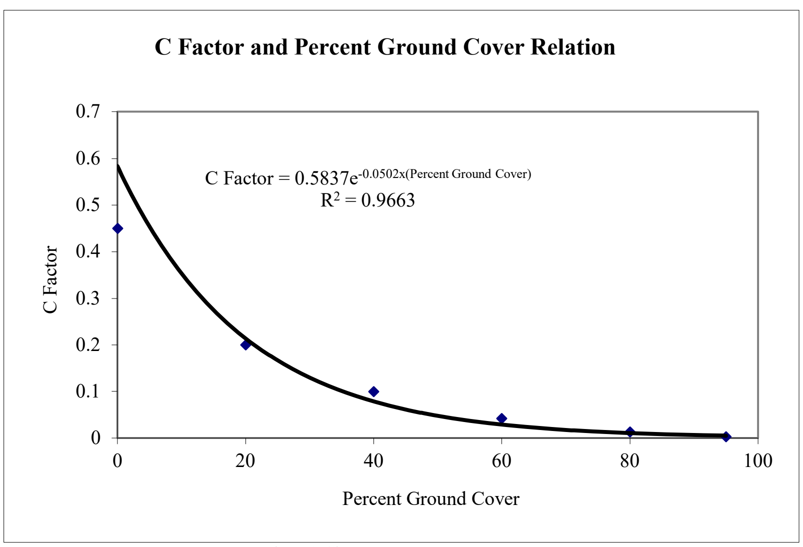
Case 6. Side Slope
slope = 33.3 %
length = 300 ft

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	5	200	0.75
2. Top Slope	5	500	1.20
3.Top Slope	5	700	1.40
4.Side Slope	33.3	105	9.50
5.Side Slope	33.3	200	13.00
6.Side Slope	33.3	300	16.00

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	0.20
40	0.10
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

Slope Condition	R	K	L_s	C	P	A (tons/ac/yr)
1. Top Slope 5% slope 200 ft length	290	0.25	0.75	0.0420	1.0	2.3
2. Top Slope 5% slope 500 ft length	290	0.25	1.20	0.0420	1.0	3.7
3. Top Slope 5% slope 700 ft length	290	0.25	1.40	0.0420	1.0	4.3
4. Side Slope 33.3% slope 105 ft length	290	0.25	9.50	0.0420	1.0	28.9
5. Side Slope 33.3% slope 200 ft length	290	0.25	13.00	0.0420	1.0	39.6
6. Side Slope 33.3% slope 300 ft length	290	0.25	16.00	0.0420	1.0	48.7

2. Summary
   For a summary of soil loss rates for various percentages of ground cover, see Figure 2 in Appendix IIIF-F and page IIIF-F-1-5.

SOIL LOSS ESTIMATE SUMMARY TABLE

Case	Slope (%)	Length (ft)	L_s	Percent Ground Cover	C Factor	A (tons/ac/yr)
Top Slope	5	200	0.75	60	0.042	2.3
Top Slope	5	200	0.75	70	0.017	0.9
Top Slope	5	200	0.75	80	0.013	0.7
Top Slope	5	200	0.75	90	0.0064	0.3
Top Slope	5	500	1.20	60	0.042	3.7
Top Slope	5	500	1.20	70	0.017	1.5
Top Slope	5	500	1.20	80	0.013	1.1
Top Slope	5	500	1.20	90	0.0064	0.6
Top Slope	5	700	1.40	60	0.042	4.3
Top Slope	5	700	1.40	70	0.017	1.7
Top Slope	5	700	1.40	80	0.013	1.3
Top Slope	5	700	1.40	90	0.0064	0.6
Side Slope	33.3	105	9.50	60	0.042	28.9
Side Slope	33.3	105	9.50	70	0.017	11.7
Side Slope	33.3	105	9.50	80	0.013	9.0
Side Slope	33.3	105	9.50	90	0.0064	4.4
Side Slope	33.3	200	13.00	60	0.042	39.6
Side Slope	33.3	200	13.00	70	0.017	16.0
Side Slope	33.3	200	13.00	80	0.013	12.3
Side Slope	33.3	200	13.00	90	0.0064	6.0
Side Slope	33.3	300	16.00	60	0.042	48.7
Side Slope	33.3	300	16.00	70	0.017	19.7
Side Slope	33.3	300	16.00	80	0.013	15.1
Side Slope	33.3	300	16.00	90	0.0064	7.4

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-1-2.
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.	Top Slope			Case 2.	Top Slope		
	slope =	5	%		slope =	5	%
	length =	200	ft		length =	500	ft
Case 3.	Top Slope			Case 4.	Side Slope		
	slope =	5	%		slope =	33.3	%
	length =	700	ft		length =	105	ft
Case 5.	Side Slope			Case 6.	Side Slope		
	slope =	33.3	%		slope =	33.3	%
	length =	200	ft		length =	300	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 =	2.0	ft/s
Case 2.	V =	2.0	ft/s
Case 3.	V =	2.0	ft/s
Case 4.	V =	5.0	ft/s
Case 5.	V =	5.0	ft/s
Case 6.	V =	5.0	ft/s

Note: Figure 15.2 is reproduced on page IIIF-F-1-8.

2. Compare to permissible velocities.

Summary of Velocities

	Condition	Equivalent Percent Ground Coverage	Peak Velocity (ft/s)	Permissible Velocity[1] (ft/s)
Cultivated Straight Row	5%, 200 ft	>60%	2.0	5.0
	5%, 500 ft	>60%	2.0	5.0
	5%, 700 ft	>60%	2.0	5.0
	33.3%, 105 ft	>60%	5.0	5.0
	33.3%, 200 ft	>60%	5.0	5.0
	33.3%, 300 ft	>60%	5.0	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.

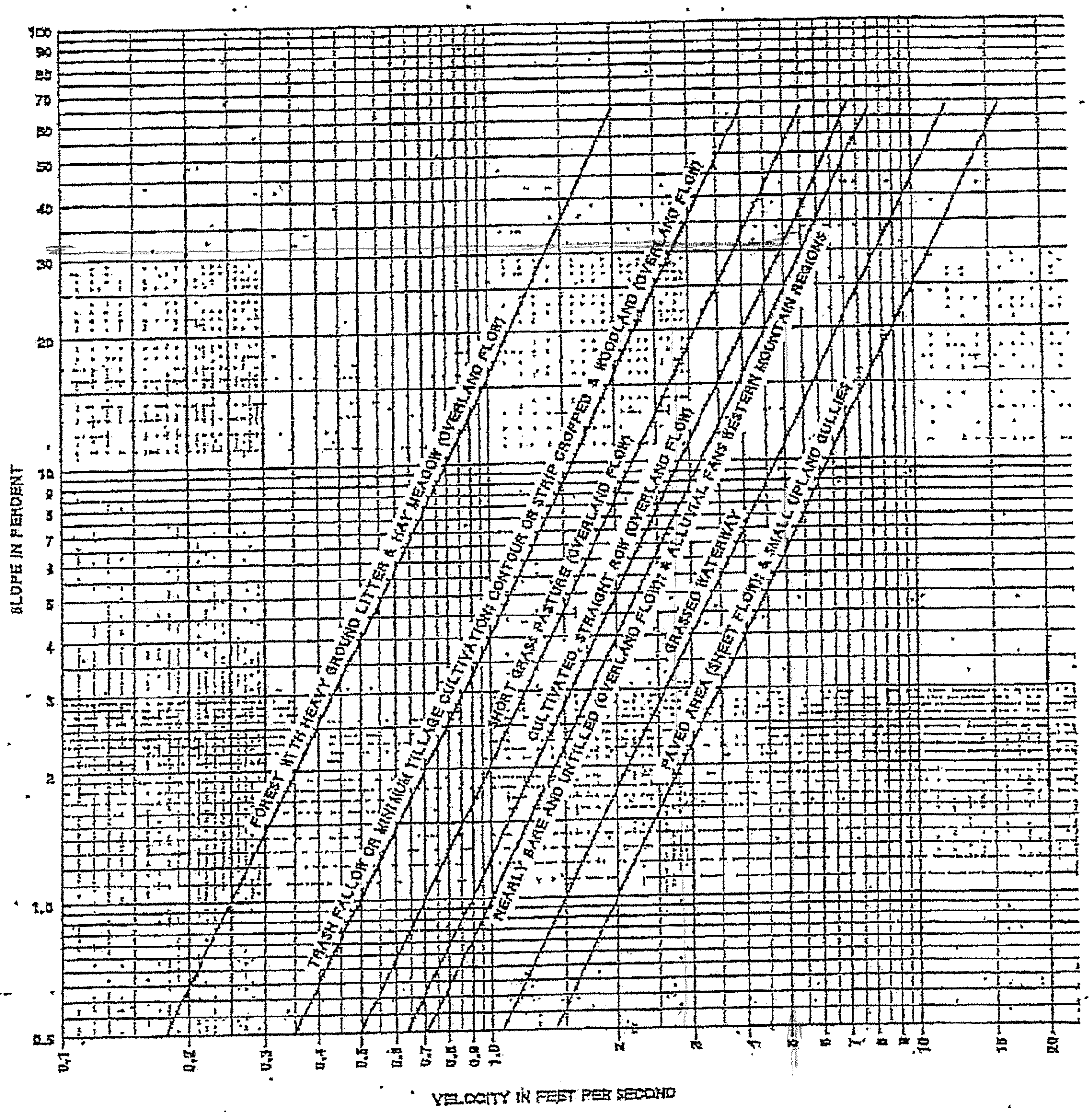


Figure 15.2.—Velocities for upland method of estimating T_c

Required: Analyze swales to determine the adequacy of the swale design.

Method:
1. Determine the 25-year, 24-hour 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, September 2019.

Solution:
1. Determine the 25-year, 24-hour flow rates for a maximum swale drainage area for top slopes and side slopes using the Rational Method.

$$Q = CIA$$

Where:
C= 0.7 (runoff coefficient, Ref 1.)
I = intensity, in/hr
A= drainage area, ac

$$I = \frac{b}{(t_c + d)^e}$$

b = 79.18 — From Ref. 1, for Tarrant County
d = 10.44 — 25-year storm event
e = 0.772
t_c is assumed to be 10 min.

I = 7.71 in/hr

For Top Slope (5%):

Maximum Drainage Area (2 ft swale) = 28.4 acres
Maximum Drainage Area (1.5 ft swale) = 13.2 acres
Maximum Drainage Area (1 ft swale) = 4.5 acres

Flow Rate (2 ft swale) =	153.0	cfs
Flow Rate (1.5 ft swale) =	71.1	cfs
Flow Rate (1 ft swale) =	24.1	cfs

For Side Slope (33.3%):

Maximum Drainage Area = 6.2 acres

Flow Rate (2 ft swale) =	33.2	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)	28.4	200	6,176
Top Slope (1.5 ft swale)	13.2	200	2,870
Top Slope (1 ft swale)	4.5	200	973
Side Slope (2 ft swale - 33.3%)	6.2	105	2,553

[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

FORT WORTH C&D LANDFILL
0771-356-11-35
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 Swale												
153.0	0.005	0.03	2	20	0	2.00	3.48	0.613	0.19	2.19	43.98	43.99
1.5 ft Top Slope Swale												
71.1	0.005	0.03	2	20	0	1.50	2.87	0.584	0.13	1.63	24.76	33.01
1 ft Top Slope Swale												
24.1	0.005	0.03	2	20	0	1.00	2.19	0.546	0.07	1.07	11.00	22.00
2 ft Side Slope Swale												
33.2	0.005	0.03	2	3.0	0	2.00	3.33	0.587	0.17	2.17	9.97	9.99

Note: Calculations were performed using the HYDROCALC HYDRAULICS program developed by Dodson and Associates (Version 2.01, 1996-2010).

Maximum flow depth is less than temporary swale height.

Design is acceptable.

Example Calculation: Calculate the normal depth for the swale for the maximum size top 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 =	153.0	cfs
S =	0.005	ft/ft
b =	0	ft
z_r =	20	(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 = 2.00 ft

R = 0.989 ft

A_f = 43.98 sf

solve for Q: $Q = 153.0$

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

$Q = VA \Rightarrow \quad V = Q/A$

$V = 3.48$ ft/s

$T = b + d(z_l + z_r)$

$T = 43.99$ ft

$$F_r = \frac{V}{(gA/T)^{0.5}}$$

$F_r = 0.613$

$$\text{Velocity Head} = \frac{V^2}{2g}$$

Velocity Head = 0.19 ft

Energy Head = water elevation + velocity head

Energy Head = 2.19 ft

APPENDIX IIIF-F-2

TEMPORARY LETDOWN DESIGN

Includes pages IIIF-F-2-1 through IIIF-F-2-37

STATE OF TEXAS
CHARLES R. MARSH
105073
LICENSED
PROFESSIONAL ENGINEER

02/09/2023

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 either 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 dissipator structure at the bottom of the letdown. Typical letdown details are shown on Sheet IIIF-F-12 – Letdown Design Summary.

This appendix includes a demonstration to show that the letdown structure sizes shown on Sheet IIIF-F-12 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 30 acres. This analysis (pages IIIF-F-2-2 through IIIF-F-2-8) is summarized on Sheet IIIF-F-12 and shows the maximum drainage areas that the 2-foot-deep chutes (8 feet minimum bottom width) are adequate to handle (i.e., the maximum flow depth calculated is less than 2.00 feet).

Also included in this appendix is an analysis for the 24-inch- and 36-inch-diameter temporary pipe letdowns for 33.3 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 Sheet IIIF-F-12, the use of pipe letdowns will be limited to 1 inlet per letdown. The design summary for geomembrane-lined letdowns and pipe letdowns is provided on Sheet IIIF-F-12.

Required: Analyze chutes to determine chute sizes for drainage areas that range from 5.00 acres to 30.0 acres.

Method:

1. Determine the 25-year, 24-hour 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, September 2019.

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 = 79.18 From Ref. 1, for Tarrant County
d = 10.44 25-year storm event
e = 0.772

t_c is assumed to be 10 min.

I = 7.71 in/hr

Area (ac)	Flow (cfs)
5.00	27.0
10.0	54.0
15.0	80.9
20.0	107.9
25.0	134.9
30.0	161.9

2. Demonstrate that the normal depth of flow for the maximum 25-year flow rate will be contained within the chute.

Please refer to Page IIIF-F-2-3 for chute hydraulic analysis output.

Uniform flow design for the geomembrane-lined chutes on 5% 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)
27.0	0.05	0.01	2	2	8	0.25	12.64	4.573	2.48	2.73	2.14	9.01
54.0	0.05	0.01	2	2	8	0.38	16.27	4.857	4.11	4.49	3.32	9.52
80.9	0.05	0.01	2	2	8	0.48	18.80	5.033	5.49	5.97	4.30	9.92
107.9	0.05	0.01	2	2	8	0.57	20.77	5.149	6.70	7.27	5.20	10.27
134.9	0.05	0.01	2	2	8	0.65	22.40	5.238	7.80	8.45	6.02	10.59
161.9	0.05	0.01	2	2	8	0.72	23.81	5.311	8.81	9.53	6.80	10.88

Uniform flow design for the geomembrane-lined chutes on 33.3% 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)
27.0	0.333	0.01	2	2	8	0.14	22.85	10.852	8.12	8.26	1.18	8.57
54.0	0.333	0.01	2	2	8	0.22	29.57	11.488	13.59	13.81	1.83	8.87
80.9	0.333	0.01	2	2	8	0.28	34.36	11.909	18.35	18.62	2.35	9.10
107.9	0.333	0.01	2	2	8	0.33	38.17	12.212	22.65	22.97	2.83	9.31
134.9	0.333	0.01	2	2	8	0.37	41.38	12.447	26.61	26.98	3.26	9.49
161.9	0.333	0.01	2	2	8	0.41	44.24	12.672	30.42	30.84	3.66	9.66

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 2.01, 1996-2010).

FORT WORTH C&D LANDFILL
0771-356-11-35
EROSION CONTROL STRUCTURE DESIGN
GABION, TURF REINFORCEMENT MAT, ROCK RIPRAP, OR CONCRETE GROUTED RIPRAP-LINED CHUTE

Chute flow design for the gabion and rock riprap-lined chutes on 5% 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)
27.0	0.05	0.04	2	2	8	0.57	5.19	1.287	0.42	0.99	5.20	10.28
54.0	0.05	0.04	2	2	8	0.85	6.55	1.356	0.67	1.52	8.25	11.40
80.9	0.05	0.04	2	2	8	1.07	7.44	1.395	0.86	1.93	10.87	12.29
107.9	0.05	0.04	2	2	8	1.26	8.15	1.425	1.03	2.29	13.25	13.04
134.9	0.05	0.04	2	2	8	1.43	8.72	1.447	1.18	2.61	15.47	13.70
161.9	0.05	0.04	2	2	8	1.58	9.21	1.464	1.32	2.89	17.58	14.31

Chute flow design for the gabion and rock riprap-lined chutes on 33.3% 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)
27.0	0.333	0.04	2	2	8	0.33	9.57	3.066	1.42	1.75	2.82	9.30
54.0	0.333	0.04	2	2	8	0.49	12.27	3.256	2.34	2.83	4.40	9.96
80.9	0.333	0.04	2	2	8	0.62	14.10	3.361	3.09	3.71	5.74	10.48
107.9	0.333	0.04	2	2	8	0.73	15.53	3.435	3.75	4.48	6.95	10.94
134.9	0.333	0.04	2	2	8	0.83	16.71	3.492	4.34	5.18	8.07	11.34
161.9	0.333	0.04	2	2	8	0.93	17.75	3.543	4.89	5.82	9.12	11.70

FORT WORTH C&D LANDFILL
0771-356-11-35
EROSION CONTROL STRUCTURE DESIGN
GABION, TURF REINFORCEMENT MAT, ROCK RIPRAP, OR CONCRETE GROUTED RIPRAP-LINED CHUTE

Conclusions: Maximum acceptable normal depth is 1.58 feet. Chute design depth is 2.0 feet; therefore, 30 acres is the maximum allowable drainage area for a gabion or rock rip-rap lined chute on a 5% slope.
Maximum velocity is 44.24 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 2.01, 1996-2010).
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-2-21**.)	25
Rock Riprap (based on Sheet **IIIF-F-2-23** and a D_{50} of 12 inches. (If other riprap is used, it will meet the D_{50} requirements listed on Sheet **IIIF-F-2-23**.)	9
Gabion/Concrete Grouted Riprap (based on **Sheet IIIF-F-2-22** 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-2-22**. (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

FORT WORTH C&D LANDFILL
0771-356-11-35
EROSION CONTROL STRUCTURE DESIGN
GABION, TURF REINFORCEMENT MAT, ROCK RIPRAP, OR CONCRETE GROUTED RIPRAP-LINED CHUTE

Chute flow design for the concrete grouted riprap and turf reinforcement-lined chutes on 5% 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)
27.0	0.05	0.03	2	2	8	0.48	6.27	1.678	0.61	1.09	4.31	9.92
54.0	0.05	0.03	2	2	8	0.72	7.94	1.770	0.98	1.70	6.80	10.88
80.9	0.05	0.03	2	2	8	0.91	9.06	1.824	1.28	2.19	8.93	11.64
107.9	0.05	0.03	2	2	8	1.07	9.92	1.860	1.53	2.60	10.87	12.29
134.9	0.05	0.03	2	2	8	1.21	10.65	1.891	1.76	2.98	12.67	12.86
161.9	0.05	0.03	2	2	8	1.34	11.26	1.915	1.97	3.32	14.38	13.38

Chute flow design for the concrete grouted riprap and turf reinforcement-lined chutes on 33.3% 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)
27.0	0.333	0.03	2	2	8	0.27	11.49	3.987	2.05	2.33	2.35	9.10
54.0	0.333	0.03	2	2	8	0.41	14.79	4.242	3.40	3.81	3.65	9.65
80.9	0.333	0.03	2	2	8	0.52	17.04	4.382	4.51	5.04	4.75	10.10
107.9	0.333	0.03	2	2	8	0.62	18.80	4.481	5.50	6.12	5.74	10.48
134.9	0.333	0.03	2	2	8	0.71	20.27	4.558	6.38	7.09	6.66	10.83
161.9	0.333	0.03	2	2	8	0.79	21.53	4.620	7.20	7.99	7.52	11.14

Conclusions: Maximum normal depth is 1.34 feet. Chute design depth is 2.0 feet; therefore, design is acceptable.
Maximum velocity is 21.53 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 2.01, 1996-2010).
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-2-21**.)	25
Rock Riprap (based on Sheet **IIIF-F-2-23** and a D_{50} of 12 inches. If other riprap is used, it will meet the D_{50} requirements listed on Sheet **IIIF-F-2-23**.)	9
Gabion/Concrete Grouted Riprap (based on Sheet **IIIF-F-2-22** 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-2-22**. 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

Required: Design the riprap erosion protection at the downstream end of the open channel letdown.

Method: Use HEC-RAS to model the open channel geomembrane-lined letdown to determine the hydraulic characteristics of the hydraulic jump that will occur at the downstream end of the letdown. Based on the results, design the riprap erosion protection area.

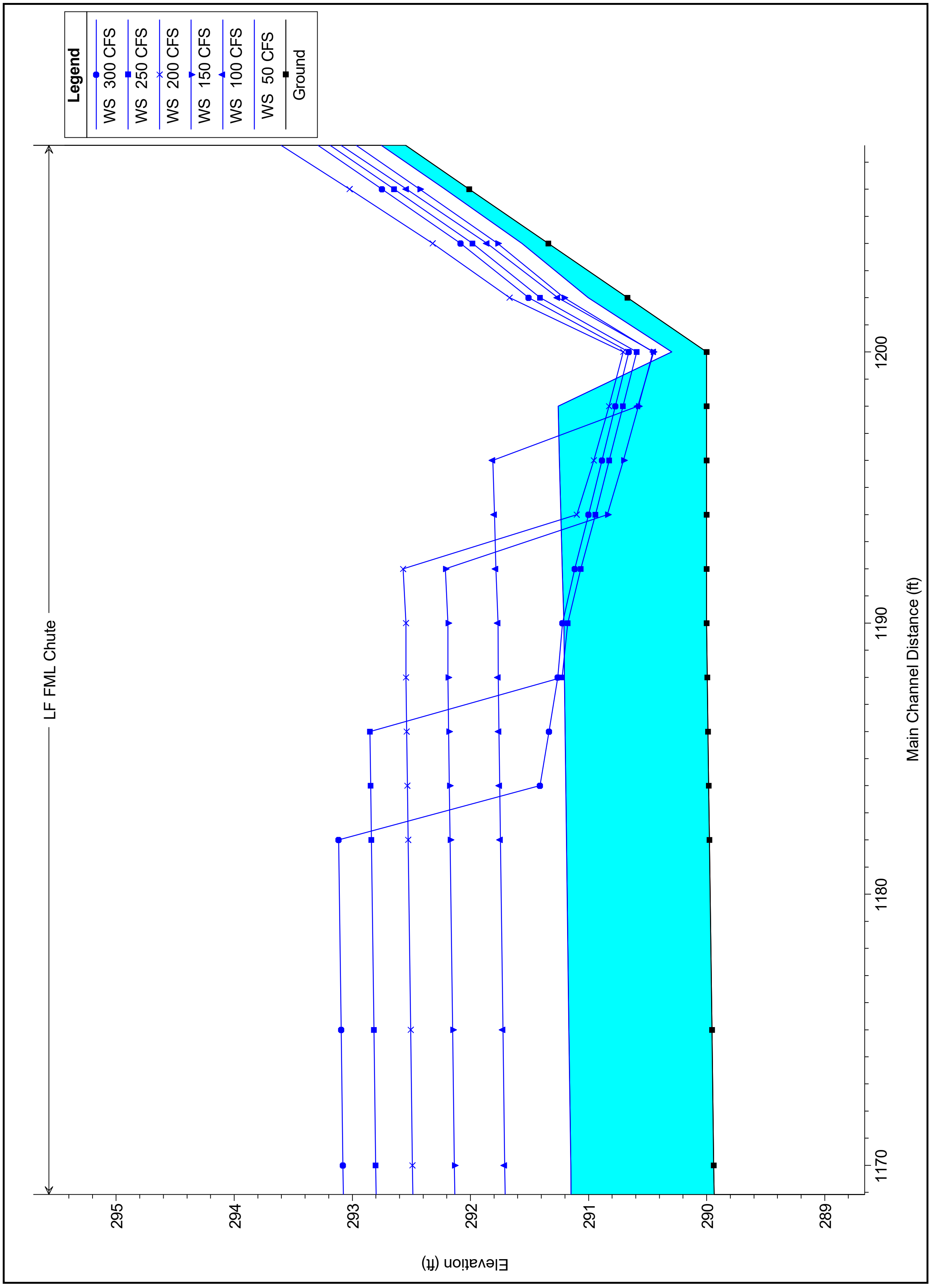
Note: This example calculation is shown for geomembrane-lined letdowns to conservatively estimate the length of riprap needed. As seen on pages IIIF-F-2-3 through IIIF-F-2-6, the geomembrane-lined letdowns have the highest velocities and represent the worst-case scenario. Therefore, this riprap design is applicable to all lined letdowns.

Solution: Page IIIF-F-2-9 shows the water surface profile for incremental flows up to 300 cfs for the geomembrane letdown into a channel, as modeled in HEC-RAS. The modeling output is presented on pages IIIF-F-2-10 through IIIF-F-2-20. The following table summarizes the erosion protection design for the various flows.

Flow (cfs)	Drainage Area* (ac)	Length of Hydraulic Jump (ft)	Specified Runout of Riprap (ft)
50	9.3	2	10
100	18.5	4	10
150	27.8	8	10
200	37.1	8	10
250	46.3	14	16
300	55.6	18	25

* Drainage areas are approximated based on the calculation methodology listed on page IIIF-F-2-2.

The values listed in the above table are specified riprap lengths for letdowns terminating into a perimeter channel. If the letdown terminates into a pond, 10 feet of riprap erosion control will be sufficient because the water in the pond will provide additional energy dissipation.


LF FML Chute
Legend
WS 300 CFS
WS 250 CFS
WS 200 CFS
WS 150 CFS
WS 100 CFS
WS 50 CFS
Ground
Elevation (ft)
295
294
293
292
291
290
289
Main Channel Distance (ft)
1170
1180
1190
1200

```
                   HEC-RAS HEC-RAS 6.2 March 2022
                    U.S. Army Corps of Engineers
                   Hydrologic Engineering Center
                          609 Second Street
                          Davis, California

   X     X  XXXXXX    XXXX        XXXX       XX      XXXX
   X     X  X        X    X       X   X     X  X    X
   X     X  X        X            X   X    X    X   X
   XXXXXXX  XXXX     X       XXX  XXXX     XXXXXX    XXXX
   X     X  X        X            X  X     X    X        X
   X     X  X        X    X       X   X    X    X        X
   X     X  XXXXXX    XXXX        X    X   X    X   XXXXX
```

```
PROJECT DATA
Project Title: Hydraulic Jump HEC-RAS
Project File : HydraulicJumpHEC-.prj
Run Date and Time: 6/3/2022 10:09:14 AM

Project in English units
PLAN DATA

Plan Title: Rio Grande Runup 0.3%
Plan File : p:\Solid waste\WC\FW C&D\Expansion 2022\Part III\IIIF\IIIF-F\Hydr Jump HEC-RAS\HydraulicJumpHEC-.p09

           Geometry Title: Rio Grande FML CHUTE with 4' RUNUP .003
           Geometry File : p:\Solid waste\WC\FW C&D\Expansion 2022\Part III\IIIF\IIIF-F\Hydr Jump HEC-RAS\HydraulicJumpHEC-.g08

           Flow Title    : Rio Grande FML CHUTE 0.3%
           Flow File     : p:\Solid waste\WC\FW C&D\Expansion 2022\Part III\IIIF\IIIF-F\Hydr Jump HEC-RAS\HydraulicJumpHEC-.f04

Plan Summary Information:
Number of:  Cross Sections =   36    Multiple Openings  =    0
            Culverts       =    0    Inline Structures  =    0
            Bridges        =    0    Lateral Structures =    0

Computational Information
    Water surface calculation tolerance  =  0.01
    Critical depth calculation tolerance =  0.01
    Maximum number of iterations         =  20
    Maximum difference tolerance         =  0.3
    Flow tolerance factor                =  0.001

Computation Options
    Critical depth computed only where necessary
    Conveyance Calculation Method: At breaks in n values only
    Friction Slope Method:         Average Conveyance
    Computational Flow Regime:     Mixed Flow

FLOW DATA

Flow Title: Rio Grande FML CHUTE 0.3%
Flow File : p:\Solid waste\WC\FW C&D\Expansion 2022\Part III\IIIF\IIIF-F\Hydr Jump HEC-RAS\HydraulicJumpHEC-.f04

Flow Data (cfs)

  River           Reach           RS                 50 CFS        100 CFS        150 CFS        200 CFS        250 CFS        300 CFS
  LF              FML Chute       5000                   50            100            150            200            250            300

Boundary Conditions

  River           Reach           Profile                     Upstream                 Downstream

  LF              FML Chute       50 CFS                   Normal S = 0.333         Normal S = 0.003
  LF              FML Chute       100 CFS                  Normal S = 0.333         Normal S = 0.003
  LF              FML Chute       150 CFS                  Normal S = 0.333         Normal S = 0.003
  LF              FML Chute       200 CFS                  Normal S = 0.333         Normal S = 0.003
  LF              FML Chute       250 CFS                  Normal S = 0.333         Normal S = 0.003
  LF              FML Chute       300 CFS                  Normal S = 0.333         Normal S = 0.003

GEOMETRY DATA

Geometry Title: Rio Grande FML CHUTE with 4' RUNUP .003
Geometry File : p:\Solid waste\WC\FW C&D\Expansion 2022\Part III\IIIF\IIIF-F\Hydr Jump HEC-RAS\HydraulicJumpHEC-.g08

CROSS SECTION
RIVER: LF
REACH: FML Chute          RS: 5000

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  566.41      20  556.41      28  556.41      48  566.41

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .01       0     .01      48     .01

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
             0      48              100     100     100             .1
```

```
CROSS SECTION
RIVER: LF
REACH: FML Chute          RS: 4900

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  533.11      20  523.11      28  523.11      48  533.11

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .01       0     .01      48     .01

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
             0      48              100     100     100             .1       .5

CROSS SECTION
RIVER: LF
REACH: FML Chute          RS: 4800

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  499.81      20  489.81      28  489.81      48  499.81

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .01       0     .01      48     .01

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
             0      48              100     100     100             .1       .5

CROSS SECTION
RIVER: LF
REACH: FML Chute          RS: 4700

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  466.51      20  456.51      28  456.51      48  466.51

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .01       0     .01      48     .01

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
             0      48              100     100     100             .1       .5

CROSS SECTION
RIVER: LF
REACH: FML Chute          RS: 4600

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  433.21      20  423.21      28  423.21      48  433.21

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .01       0     .01      48     .01

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
             0      48              100     100     100             .1       .5

CROSS SECTION
RIVER: LF
REACH: FML Chute          RS: 4500

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  399.91      20  389.91      28  389.91      48  399.91

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .01       0     .01      48     .01

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
             0      48              100     100     100             .1       .5

CROSS SECTION
RIVER: LF
REACH: FML Chute          RS: 4400

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  366.61      20  356.61      28  356.61      48  366.6
```

```
Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .01       0     .01      48     .01

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
             0      48              100     100     100             .1       .5
CROSS SECTION
RIVER: LF
REACH: FML Chute          RS: 4300

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  333.31      20  323.31      28  323.31      48  333.31

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .01       0     .01      48     .01

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
             0      48               75      75      75             .1       .5

CROSS SECTION
RIVER: LF
REACH: FML Chute          RS: 4225

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0 308.335      20 298.335      28 298.335      48 308.335

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .01       0     .01      48     .01

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
             0      48                5       5       5             .1       .5

CROSS SECTION
RIVER: LF
REACH: FML Chute          RS: 4220

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  306.67      20  296.67      28  296.67      48  306.67

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .01       0     .01      48     .01

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
             0      48                5       5       5             .1       .5

CROSS SECTION
RIVER: LF
REACH: FML Chute          RS: 4215

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  305.01      20  295.01      28  295.01      48  305.01

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .01       0     .01      48     .01

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
             0      48                5       5       5             .1       .5
CROSS SECTION
RIVER: LF
REACH: FML Chute          RS: 4210

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  303.34      20  293.34      28  293.34      48  303.34

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .01       0     .01      48     .01

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
             0      48                4       4       4             .1       .5

CROSS SECTION
RIVER: LF
REACH: FML Chute          RS: 4206

INPUT
```

```
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  302.01      20  292.01      28  292.01      48  302.01

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .01       0     .01      48     .01

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
             0      48                2       2       2             .1       .5

CROSS SECTION
RIVER: LF
REACH: FML Chute          RS: 4204

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  301.34      20  291.34      28  291.34      48  301.34

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .04       0     .04      48     .04

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
             0      48                2       2       2             .1       .5

CROSS SECTION
RIVER: LF
REACH: FML Chute          RS: 4202

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  300.67      20  290.67      28  290.67      48  300.67

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .04       0     .04      48     .04

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
             0      48                2       2       2             .1       .5
CROSS SECTION
RIVER: LF
REACH: FML Chute          RS: 4200

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0     300      30     290      42     290      72     300

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .04       0     .04      72     .04

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
             0      72                2       2       2             .1       .5

CROSS SECTION
RIVER: LF
REACH: FML Chute          RS: 4198

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0     300      30     290      42     290      72     300

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .04       0     .04      72     .04

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
             0      72                2       2       2             .1       .5

CROSS SECTION
RIVER: LF
REACH: FML Chute          RS: 4196

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0     300      30     290      42     290      72     300

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .04       0     .04      72     .04

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
             0      72                2       2       2             .1       .
```

```
CROSS SECTION
RIVER: LF
REACH: FML Chute          RS: 4194

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0     300      30     290      42     290      72     300

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .04       0     .04      72     .04

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
             0      72                2       2       2             .1       .5

CROSS SECTION
RIVER: LF
REACH: FML Chute          RS: 4192

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0     300      30     290      42     290      72     300

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .04       0     .04      72     .04

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
             0      72                2       2       2             .1       .5

CROSS SECTION
RIVER: LF
REACH: FML Chute          RS: 4190

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0     310      60     290      72     290     132     310

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .03       0     .03      72     .03

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
             0      72                2       2       2             .1       .5

CROSS SECTION
RIVER: LF
REACH: FML Chute          RS: 4188

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0 309.994      60 289.994      72 289.994     132 309.994

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .03       0     .03     132     .03

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
             0     132                2       2       2             .1       .5

CROSS SECTION
RIVER: LF
REACH: FML Chute          RS: 4186

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0 309.988      60 289.988      72 289.988     132 309.988

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .03       0     .03     132     .03

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
             0     132                2       2       2             .1       .5

CROSS SECTION
RIVER: LF
REACH: FML Chute          RS: 4184

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0 309.982      60 289.982      72 289.982     132 309.98
```

```
Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .03       0     .03     132     .03

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
             0     132                2       2       2             .1       .5

CROSS SECTION
RIVER: LF
REACH: FML Chute          RS: 4182

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0 309.976      60 289.976      72 289.976     132 309.976

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .03       0     .03     132     .03

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
             0     132                7       7       7             .1       .5

CROSS SECTION
RIVER: LF
REACH: FML Chute          RS: 4175

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0 309.955      60 289.955      72 289.955     132 309.955

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .03       0     .03     132     .03

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
             0     132                5       5       5             .1       .5

CROSS SECTION
RIVER: LF
REACH: FML Chute          RS: 4170

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  309.94      60  289.94      72  289.94     132  309.94

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .03       0     .03     132     .03

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
             0     132               10      10      10             .1       .5

CROSS SECTION
RIVER: LF
REACH: FML Chute          RS: 4160

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  309.91      60  289.91      72  289.91     132  309.91

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .03       0     .03     132     .03

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
             0     132               10      10      10             .1       .5

CROSS SECTION
RIVER: LF
REACH: FML Chute          RS: 4150

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  309.88      60  289.88      72  289.88     132  309.88

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .03       0     .03     132     .03

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
             0     132               10      10      10             .1       .5


CROSS SECTION
RIVER: LF
REACH: FML Chute          RS: 4140

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  309.85      60  289.85      72  289.85     132  309.85

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .03       0     .03     132     .03

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
             0     132               10      10      10             .1       .5

CROSS SECTION
RIVER: LF
REACH: FML Chute          RS: 413
```

```
INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  309.82      60  289.82      72  289.82     132  309.82

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .03       0     .03     132     .03

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
             0     132              10      10      10              .1       .5

CROSS SECTION
RIVER: LF
REACH: FML Chute         RS: 4120

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  309.79      60  289.79      72  289.79     132  309.79

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .03       0     .03     132     .03

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
             0     132              10      10      10              .1       .5

CROSS SECTION
RIVER: LF
REACH: FML Chute         RS: 4110

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  309.76      60  289.76      72  289.76     132  309.76

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .03       0     .03     132     .03

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
             0     132              10      10      10              .1       .5

CROSS SECTION
RIVER: LF
REACH: FML Chute         RS: 4100

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  309.73      60  289.73      72  289.73     132  309.73

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .03       0     .03     132     .03

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
             0     132             100     100     100              .1       .5

CROSS SECTION
RIVER: LF
REACH: FML Chute         RS: 4000

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  309.43      60  289.43      72  289.43     132  309.43

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .03       0     .03     132     .03

Bank Sta: Left   Right    Lengths: Left Channel   Right     Coeff Contr.   Expan.
             0     132            1000    1000    1000              .1       .5

CROSS SECTION
RIVER: LF
REACH: FML Chute         RS: 3000

INPUT
Description:
Station Elevation Data    num=       4
     Sta    Elev     Sta    Elev     Sta    Elev     Sta    Elev
       0  306.43      60  286.43      72  286.43     132  306.43

Manning's n Values        num=       3
     Sta   n Val     Sta   n Val     Sta   n Val
       0     .03       0     .03     132     .03

Bank Sta: Left   Right    Coeff Contr.   Expan.
             0     132             .1       .5
```

SUMMARY OF MANNING'S N VALUES

River:LF

Reach	River Sta.	n1	n2	n3
FML Chute	5000	.01	.01	.01
FML Chute	4900	.01	.01	.01
FML Chute	4800	.01	.01	.01
FML Chute	4700	.01	.01	.01
FML Chute	4600	.01	.01	.01
FML Chute	4500	.01	.01	.01
FML Chute	4400	.01	.01	.01
FML Chute	4300	.01	.01	.01
FML Chute	4225	.01	.01	.01

FML Chute	4220	.01	.01	.01
FML Chute	4215	.01	.01	.01
FML Chute	4210	.01	.01	.01
FML Chute	4206	.01	.01	.01
FML Chute	4204	.04	.04	.04
FML Chute	4202	.04	.04	.04
FML Chute	4200	.04	.04	.04
FML Chute	4198	.04	.04	.04
FML Chute	4196	.04	.04	.04
FML Chute	4194	.04	.04	.04
FML Chute	4192	.04	.04	.04
FML Chute	4190	.03	.03	.03
FML Chute	4188	.03	.03	.03
FML Chute	4186	.03	.03	.03
FML Chute	4184	.03	.03	.03
FML Chute	4182	.03	.03	.03
FML Chute	4175	.03	.03	.03
FML Chute	4170	.03	.03	.03
FML Chute	4160	.03	.03	.03
FML Chute	4150	.03	.03	.03
FML Chute	4140	.03	.03	.03
FML Chute	4130	.03	.03	.03
FML Chute	4120	.03	.03	.03
FML Chute	4110	.03	.03	.03
FML Chute	4100	.03	.03	.03
FML Chute	4000	.03	.03	.03
FML Chute	3000	.03	.03	.03

SUMMARY OF REACH LENGTHS

River: LF

Reach	River Sta.	Left	Channel	Right
FML Chute	5000	100	100	100
FML Chute	4900	100	100	100
FML Chute	4800	100	100	100
FML Chute	4700	100	100	100
FML Chute	4600	100	100	100
FML Chute	4500	100	100	100
FML Chute	4400	100	100	100
FML Chute	4300	75	75	75
FML Chute	4225	5	5	5
FML Chute	4220	5	5	5
FML Chute	4215	5	5	5
FML Chute	4210	4	4	4
FML Chute	4206	2	2	2
FML Chute	4204	2	2	2
FML Chute	4202	2	2	2
FML Chute	4200	2	2	2
FML Chute	4198	2	2	2
FML Chute	4196	2	2	2
FML Chute	4194	2	2	2
FML Chute	4192	2	2	2
FML Chute	4190	2	2	2
FML Chute	4188	2	2	2
FML Chute	4186	2	2	2
FML Chute	4184	2	2	2
FML Chute	4182	7	7	7
FML Chute	4175	5	5	5
FML Chute	4170	10	10	10
FML Chute	4160	10	10	10
FML Chute	4150	10	10	10
FML Chute	4140	10	10	10
FML Chute	4130	10	10	10
FML Chute	4120	10	10	10
FML Chute	4110	10	10	10
FML Chute	4100	100	100	100
FML Chute	4000	1000	1000	1000
FML Chute	3000			

SUMMARY OF CONTRACTION AND EXPANSION COEFFICIENTS
River: LF

Reach	River Sta.	Contr.	Expan.
FML Chute	5000	.1	.5
FML Chute	4900	.1	.5
FML Chute	4800	.1	.5
FML Chute	4700	.1	.5
FML Chute	4600	.1	.5
FML Chute	4500	.1	.5
FML Chute	4400	.1	.5
FML Chute	4300	.1	.5
FML Chute	4225	.1	.5
FML Chute	4220	.1	.5
FML Chute	4215	.1	.5
FML Chute	4210	.1	.5
FML Chute	4206	.1	.5
FML Chute	4204	.1	.5
FML Chute	4202	.1	.5
FML Chute	4200	.1	.5
FML Chute	4198	.1	.5
FML Chute	4196	.1	.5
FML Chute	4194	.1	.5
FML Chute	4192	.1	.5
FML Chute	4190	.1	.5
FML Chute	4188	.1	.5
FML Chute	4186	.1	.5
FML Chute	4184	.1	.5
FML Chute	4182	.1	.5
FML Chute	4175	.1	.5
FML Chute	4170	.1	.5
FML Chute	4160	.1	.5
FML Chute	4150	.1	.5
FML Chute	4140	.1	.5
FML Chute	4130	.1	.5
FML Chute	4120	.1	.5
FML Chute	4110	.1	.5
FML Chute	4100	.1	.5
FML Chute	4000	.1	.5

FML Chute 3000 .1 .5

Profile Output Table - Standard Table 1

Reach	River Sta	Profile	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	Crit W.S. (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Froude # Chl
FML Chute	5000	50 CFS	50.00	556.41	556.62	557.39	569.51	0.333234	28.80	1.74	8.83	11.45
FML Chute	5000	100 CFS	100.00	556.41	556.72	557.89	578.26	0.333110	37.23	2.69	9.25	12.18
FML Chute	5000	150 CFS	150.00	556.41	556.81	558.28	585.66	0.332972	43.09	3.48	9.58	12.61
FML Chute	5000	200 CFS	200.00	556.41	556.88	558.62	592.31	0.333648	47.75	4.19	9.87	12.92
FML Chute	5000	250 CFS	250.00	556.41	556.94	558.91	598.34	0.333628	51.61	4.84	10.14	13.16
FML Chute	5000	300 CFS	300.00	556.41	557.00	559.18	603.93	0.333631	54.95	5.46	10.38	13.36
FML Chute	4900	50 CFS	50.00	523.11	523.40	524.09	529.88	0.111748	20.43	2.45	9.14	6.96
FML Chute	4900	100 CFS	100.00	523.11	524.59	524.59	525.18	0.001491	6.15	16.25	13.93	1.00
FML Chute	4900	150 CFS	150.00	523.11	524.98	524.98	525.71	0.001415	6.81	22.03	15.50	1.01
FML Chute	4900	200 CFS	200.00	523.11	525.32	525.32	526.15	0.001361	7.29	27.43	16.83	1.01
FML Chute	4900	250 CFS	250.00	523.11	525.61	525.61	526.53	0.001320	7.68	32.56	18.01	1.01
FML Chute	4900	300 CFS	300.00	523.11	525.88	525.88	526.87	0.001289	8.01	37.47	19.07	1.01
FML Chute	4800	50 CFS	50.00	489.81	489.99	490.79	507.56	0.547894	33.63	1.49	8.71	14.35
FML Chute	4800	100 CFS	100.00	489.81	491.29	491.29	491.88	0.001489	6.15	16.26	13.93	1.00
FML Chute	4800	150 CFS	150.00	489.81	491.68	491.68	492.41	0.001415	6.81	22.03	15.50	1.01
FML Chute	4800	200 CFS	200.00	489.81	492.02	492.02	492.85	0.001361	7.29	27.43	16.84	1.01
FML Chute	4800	250 CFS	250.00	489.81	490.40	492.31	522.91	0.230832	45.74	5.47	10.38	11.11
FML Chute	4800	300 CFS	300.00	489.81	490.51	492.58	523.26	0.193736	45.91	6.53	10.78	10.40
FML Chute	4700	50 CFS	50.00	456.51	457.49	457.49	457.90	0.001646	5.14	9.73	11.91	1.00
FML Chute	4700	100 CFS	100.00	456.51	457.99	457.99	458.58	0.001489	6.15	16.26	13.93	1.00
FML Chute	4700	150 CFS	150.00	456.51	458.38	458.38	459.11	0.001415	6.81	22.03	15.50	1.01
FML Chute	4700	200 CFS	200.00	456.51	458.72	458.72	459.55	0.001361	7.29	27.43	16.84	1.01
FML Chute	4700	250 CFS	250.00	456.51	459.01	459.01	459.93	0.001320	7.68	32.56	18.01	1.01
FML Chute	4700	300 CFS	300.00	456.51	457.13	459.28	499.16	0.282220	52.00	5.77	10.50	12.37
FML Chute	4600	50 CFS	50.00	423.21	423.35	424.19	454.24	1.362689	44.58	1.12	8.54	21.69
FML Chute	4600	100 CFS	100.00	423.21	424.69	424.69	425.28	0.001489	6.15	16.26	13.93	1.00
FML Chute	4600	150 CFS	150.00	423.21	425.08	425.08	425.81	0.001415	6.81	22.03	15.50	1.01
FML Chute	4600	200 CFS	200.00	423.21	425.42	425.42	426.25	0.001361	7.29	27.43	16.84	1.01
FML Chute	4600	250 CFS	250.00	423.21	423.80	425.71	456.31	0.230832	45.74	5.47	10.38	11.11
FML Chute	4600	300 CFS	300.00	423.21	425.98	425.98	426.97	0.001289	8.01	37.47	19.07	1.01
FML Chute	4500	50 CFS	50.00	389.91	390.15	390.89	399.89	0.213100	25.04	2.00	8.94	9.34
FML Chute	4500	100 CFS	100.00	389.91	391.39	391.39	391.98	0.001489	6.15	16.26	13.93	1.00
FML Chute	4500	150 CFS	150.00	389.91	391.78	391.78	392.51	0.001415	6.81	22.03	15.50	1.01
FML Chute	4500	200 CFS	200.00	389.91	392.12	392.12	392.95	0.001361	7.29	27.43	16.84	1.01
FML Chute	4500	250 CFS	250.00	389.91	392.41	392.41	393.33	0.001320	7.68	32.56	18.01	1.01
FML Chute	4500	300 CFS	300.00	389.91	390.61	392.68	423.36	0.193736	45.91	6.53	10.78	10.40
FML Chute	4400	50 CFS	50.00	356.61	356.81	357.59	371.12	0.394203	30.35	1.65	8.79	12.36
FML Chute	4400	100 CFS	100.00	356.61	358.09	358.09	358.68	0.001489	6.15	16.26	13.93	1.00
FML Chute	4400	150 CFS	150.00	356.61	358.48	358.48	359.21	0.001415	6.81	22.03	15.50	1.01
FML Chute	4400	200 CFS	200.00	356.61	358.82	358.82	359.65	0.001361	7.29	27.43	16.84	1.01
FML Chute	4400	250 CFS	250.00	356.61	357.20	359.11	389.71	0.230832	45.74	5.47	10.38	11.11
FML Chute	4400	300 CFS	300.00	356.61	357.23	359.38	399.26	0.282220	52.00	5.77	10.50	12.37
FML Chute	4300	50 CFS	50.00	323.31	323.54	324.29	333.67	0.226621	25.53	1.96	8.93	9.61
FML Chute	4300	100 CFS	100.00	323.31	324.79	324.79	325.38	0.001489	6.15	16.26	13.93	1.00
FML Chute	4300	150 CFS	150.00	323.31	325.18	325.18	325.91	0.001415	6.81	22.03	15.50	1.01
FML Chute	4300	200 CFS	200.00	323.31	325.52	325.52	326.35	0.001361	7.29	27.43	16.84	1.01
FML Chute	4300	250 CFS	250.00	323.31	325.81	325.81	326.73	0.001320	7.68	32.56	18.01	1.01
FML Chute	4300	300 CFS	300.00	323.31	326.08	326.08	327.07	0.001289	8.01	37.47	19.07	1.01
FML Chute	4225	50 CFS	50.00	298.34	298.54	299.31	312.12	0.362431	29.57	1.69	8.81	11.89
FML Chute	4225	100 CFS	100.00	298.34	299.82	299.82	300.41	0.001489	6.15	16.26	13.93	1.00
FML Chute	4225	150 CFS	150.00	298.34	298.76	300.21	323.17	0.256779	39.63	3.78	9.71	11.19
FML Chute	4225	200 CFS	200.00	298.34	298.89	300.54	323.61	0.192150	39.89	5.01	10.20	10.03
FML Chute	4225	250 CFS	250.00	298.34	299.00	300.84	323.99	0.155014	40.10	6.23	10.67	9.25
FML Chute	4225	300 CFS	300.00	298.34	299.11	301.10	324.33	0.130940	40.28	7.45	11.12	8.68
FML Chute	4220	50 CFS	50.00	296.67	296.87	297.65	310.21	0.351928	29.30	1.71	8.81	11.74
FML Chute	4220	100 CFS	100.00	296.67	297.46	298.15	300.18	0.013884	13.22	7.56	11.16	2.83
FML Chute	4220	150 CFS	150.00	296.67	297.09	298.54	321.84	0.262273	39.90	3.76	9.70	11.30
FML Chute	4220	200 CFS	200.00	296.67	298.88	298.88	299.71	0.001360	7.29	27.43	16.84	1.01
FML Chute	4220	250 CFS	250.00	296.67	297.33	299.17	323.12	0.162567	40.74	6.14	10.63	9.45
FML Chute	4220	300 CFS	300.00	296.67	297.44	299.44	323.57	0.138088	41.01	7.32	11.07	8.89
FML Chute	4215	50 CFS	50.00	295.01	295.21	295.99	308.39	0.344914	29.11	1.72	8.82	11.63
FML Chute	4215	100 CFS	100.00	295.01	295.66	296.49	299.92	0.027305	16.56	6.04	10.60	3.87
FML Chute	4215	150 CFS	150.00	295.01	295.43	296.88	320.49	0.267394	40.15	3.74	9.69	11.40
FML Chute	4215	200 CFS	200.00	295.01	296.33	297.22	299.47	0.008981	14.20	14.09	13.29	2.43
FML Chute	4215	250 CFS	250.00	295.01	295.66	297.51	322.22	0.169937	41.34	6.05	10.60	9.65
FML Chute	4215	300 CFS	300.00	295.01	295.77	297.78	322.78	0.145089	41.69	7.20	11.03	9.10
FML Chute	4210	50 CFS	50.00	293.34	293.55	294.32	306.62	0.340963	29.01	1.72	8.82	11.57
FML Chute	4210	100 CFS	100.00	293.34	293.91	294.82	299.61	0.042424	19.15	5.22	10.29	4.74
FML Chute	4210	150 CFS	150.00	293.34	293.76	295.21	319.11	0.272308	40.39	3.71	9.68	11.50
FML Chute	4210	200 CFS	200.00	293.34	294.45	295.55	299.24	0.016618	17.56	11.39	12.46	3.24
FML Chute	4210	250 CFS	250.00	293.34	293.98	295.84	321.28	0.177130	41.91	5.97	10.57	9.84
FML Chute	4210	300 CFS	300.00	293.34	294.09	296.11	321.95	0.151989	42.34	7.09	10.99	9.30
FML Chute	4206	50 CFS	50.00	292.01	292.22	292.99	305.25	0.339094	28.96	1.73	8.82	11.54
FML Chute	4206	100 CFS	100.00	292.01	292.54	293.49	299.31	0.055170	20.87	4.79	10.12	5.35
FML Chute	4206	150 CFS	150.00	292.01	292.43	293.88	318.00	0.275899	40.56	3.70	9.67	11.57
FML Chute	4206	200 CFS	200.00	292.01	293.02	294.22	299.04	0.023202	19.68	10.16	12.05	3.78
FML Chute	4206	250 CFS	250.00	292.01	292.65	294.51	320.50	0.182654	42.34	5.90	10.55	9.98
FML Chute	4206	300 CFS	300.00	292.01	292.75	294.78	321.27	0.157383	42.84	7.00	10.96	9.45
FML Chute	4204	50 CFS	50.00	291.34	291.56	292.32	302.62	4.172140	26.67	1.87	8.89	10.24
FML Chute	4204	100 CFS	100.00	291.34	291.86	292.82	298.98	0.955054	21.41	4.67	10.07	5.54
FML Chute	4204	150 CFS	150.00	291.34	291.77	293.21	315.88	4.030682	39.39	3.81	9.72	11.09
FML Chute	4204	200 CFS	200.00	291.34	292.32	293.55	298.87	0.420194	20.53	9.74	11.91	4.00
FML Chute	4204	250 CFS	250.00	291.34	291.98	293.84	319.31	2.839183	41.94	5.96	10.57	9.84
FML Chute	4204	300 CFS	300.00	291.34	292.08	294.11	320.33	2.481728	42.63	7.04	10.97	9.38
FML Chute	4202	50 CFS	50.00	290.67	291.00	291.65	295.67	1.064640	17.33	2.88	9.33	5.50
FML Chute	4202	100 CFS	100.00	290.67	291.26	292.15	296.57	0.609397	18.48	5.41	10.36	4.51
FML Chute	4202	150 CFS	150.00	290.67	291.21	292.54	305.85	1.871139	30.70	4.89	10.15	7.80
FML Chute	4202	200 CFS	200.00	290.67	291.67	292.88	297.89	0.389814	20.01	9.99	12.00	3.87
FML Chute	4202	250 CFS	250.00	290.67	291.41	293.17	311.13	1.737490	35.62	7.02	10.96	7.85
FML Chute	4202	300 CFS	300.00	290.67	291.51	293.44	312.91	1.641477	37.11	8.08	11.34	7.75
FML Chute	4200	50 CFS	50.00	290.00	290.30	290.76	292.97	0.697387	13.11	3.81	13.78	4.39

FML Chute	4200	100 CFS	100.00	290.00	290.45	291.17	294.79	0.679885	16.72	5.98	14.69	4.62
FML Chute	4200	150 CFS	150.00	290.00	290.45	291.48	299.98	1.470453	24.76	6.06	14.72	6.80
FML Chute	4200	200 CFS	200.00	290.00	290.71	291.75	296.96	0.568827	20.05	9.97	16.24	4.51
FML Chute	4200	250 CFS	250.00	290.00	290.59	291.99	305.21	1.639404	30.67	8.15	15.55	7.47
FML Chute	4200	300 CFS	300.00	290.00	290.66	292.21	307.20	1.635594	32.62	9.20	15.95	7.57
FML Chute	4198	50 CFS	50.00	290.00	291.26	290.76	291.36	0.004652	2.52	19.82	19.54	0.44
FML Chute	4198	100 CFS	100.00	290.00	290.58	291.17	293.01	0.277130	12.49	8.01	15.49	3.06
FML Chute	4198	150 CFS	150.00	290.00	290.58	291.48	296.13	0.641417	18.91	7.93	15.47	4.66
FML Chute	4198	200 CFS	200.00	290.00	290.83	291.75	295.16	0.327645	16.69	11.98	16.96	3.50
FML Chute	4198	250 CFS	250.00	290.00	290.71	291.99	300.41	0.879390	24.98	10.01	16.25	5.61
FML Chute	4198	300 CFS	300.00	290.00	290.77	292.21	302.22	0.939075	27.14	11.05	16.63	5.87
FML Chute	4196	50 CFS	50.00	290.00	291.25		291.35	0.004810	2.55	19.59	19.47	0.45
FML Chute	4196	100 CFS	100.00	290.00	291.81	291.17	291.97	0.004913	3.17	31.52	22.85	0.48
FML Chute	4196	150 CFS	150.00	290.00	290.70	291.48	294.24	0.322813	15.09	9.94	16.23	3.40
FML Chute	4196	200 CFS	200.00	290.00	290.96	291.75	294.04	0.197716	14.08	14.20	17.73	2.77
FML Chute	4196	250 CFS	250.00	290.00	290.82	291.99	297.65	0.519279	20.96	11.93	16.94	4.40
FML Chute	4196	300 CFS	300.00	290.00	290.88	292.21	299.21	0.583126	23.14	12.96	17.31	4.71
FML Chute	4194	50 CFS	50.00	290.00	291.23		291.34	0.004983	2.58	19.36	19.40	0.46
FML Chute	4194	100 CFS	100.00	290.00	291.80		291.96	0.005042	3.20	31.24	22.78	0.48
FML Chute	4194	150 CFS	150.00	290.00	290.84	291.48	293.19	0.174955	12.30	12.19	17.04	2.56
FML Chute	4194	200 CFS	200.00	290.00	291.10	291.75	293.30	0.120037	11.89	16.83	18.60	2.20
FML Chute	4194	250 CFS	250.00	290.00	290.94	291.99	295.93	0.325085	17.91	13.96	17.65	3.55
FML Chute	4194	300 CFS	300.00	290.00	291.00	292.21	297.24	0.380880	20.04	14.97	17.99	3.87
FML Chute	4192	50 CFS	50.00	290.00	291.22		291.33	0.005160	2.61	19.13	19.33	0.46
FML Chute	4192	100 CFS	100.00	290.00	291.78		291.95	0.005172	3.23	30.96	22.70	0.49
FML Chute	4192	150 CFS	150.00	290.00	292.21	291.48	292.42	0.005181	3.64	41.24	25.28	0.50
FML Chute	4192	200 CFS	200.00	290.00	292.57	291.75	292.81	0.005190	3.95	50.64	27.42	0.51
FML Chute	4192	250 CFS	250.00	290.00	291.06	291.99	294.78	0.210569	15.46	16.17	18.39	2.91
FML Chute	4192	300 CFS	300.00	290.00	291.12	292.21	295.89	0.257086	17.53	17.11	18.69	3.23
FML Chute	4190	50 CFS	50.00	290.00	291.21		291.32	0.002884	2.76	18.87	19.25	0.47
FML Chute	4190	100 CFS	100.00	290.00	291.77		291.94	0.002881	3.44	30.55	22.60	0.50
FML Chute	4190	150 CFS	150.00	290.00	292.19		292.41	0.002882	3.89	40.71	25.15	0.51
FML Chute	4190	200 CFS	200.00	290.00	292.55		292.80	0.002887	4.24	50.00	27.28	0.52
FML Chute	4190	250 CFS	250.00	290.00	291.18	292.01	294.18	0.079253	14.22	18.27	19.06	2.46
FML Chute	4190	300 CFS	300.00	290.00	291.22	292.23	295.18	0.100281	16.34	19.10	19.32	2.78
FML Chute	4188	50 CFS	50.00	289.99	291.20		291.31	0.002999	2.64	18.91	19.26	0.47
FML Chute	4188	100 CFS	100.00	289.99	291.76		291.93	0.003001	3.27	30.62	22.62	0.49
FML Chute	4188	150 CFS	150.00	289.99	292.19		292.40	0.003000	3.68	40.82	25.18	0.51
FML Chute	4188	200 CFS	200.00	289.99	292.55		292.79	0.003001	3.99	50.14	27.31	0.52
FML Chute	4188	250 CFS	250.00	289.99	291.22	291.99	293.83	0.070807	12.96	19.29	19.38	2.29
FML Chute	4188	300 CFS	300.00	289.99	291.26	292.21	294.76	0.091842	15.00	19.99	19.59	2.62
FML Chute	4186	50 CFS	50.00	289.99	291.20		291.31	0.003000	2.64	18.91	19.26	0.47
FML Chute	4186	100 CFS	100.00	289.99	291.76		291.92	0.003001	3.27	30.62	22.62	0.49
FML Chute	4186	150 CFS	150.00	289.99	292.18		292.39	0.003000	3.68	40.82	25.17	0.51
FML Chute	4186	200 CFS	200.00	289.99	292.54		292.79	0.003002	3.99	50.14	27.31	0.52
FML Chute	4186	250 CFS	250.00	289.99	292.85	291.98	293.13	0.002997	4.24	58.92	29.17	0.53
FML Chute	4186	300 CFS	300.00	289.99	291.33	292.20	294.34	0.073608	13.90	21.58	20.07	2.36
FML Chute	4184	50 CFS	50.00	289.98	291.19		291.30	0.003000	2.64	18.91	19.26	0.47
FML Chute	4184	100 CFS	100.00	289.98	291.75		291.92	0.003001	3.27	30.62	22.62	0.49
FML Chute	4184	150 CFS	150.00	289.98	292.18		292.39	0.003000	3.68	40.82	25.17	0.51
FML Chute	4184	200 CFS	200.00	289.98	292.53		292.78	0.003002	3.99	50.14	27.31	0.52
FML Chute	4184	250 CFS	250.00	289.98	292.84		293.12	0.002996	4.24	58.93	29.17	0.53
FML Chute	4184	300 CFS	300.00	289.98	291.41	292.19	293.99	0.059238	12.89	23.27	20.57	2.14
FML Chute	4182	50 CFS	50.00	289.98	291.19		291.29	0.003000	2.64	18.91	19.26	0.47
FML Chute	4182	100 CFS	100.00	289.98	291.75		291.91	0.003001	3.27	30.62	22.61	0.49
FML Chute	4182	150 CFS	150.00	289.98	292.17		292.38	0.003000	3.68	40.81	25.17	0.51
FML Chute	4182	200 CFS	200.00	289.98	292.53		292.77	0.003002	3.99	50.14	27.31	0.52
FML Chute	4182	250 CFS	250.00	289.98	292.84		293.12	0.002996	4.24	58.93	29.17	0.53
FML Chute	4182	300 CFS	300.00	289.98	293.12	292.19	293.42	0.002998	4.46	67.23	30.83	0.53
FML Chute	4175	50 CFS	50.00	289.96	291.16		291.27	0.003000	2.64	18.91	19.26	0.47
FML Chute	4175	100 CFS	100.00	289.96	291.72		291.89	0.003001	3.27	30.62	22.62	0.49
FML Chute	4175	150 CFS	150.00	289.96	292.15		292.36	0.003000	3.68	40.82	25.17	0.51
FML Chute	4175	200 CFS	200.00	289.96	292.51		292.75	0.003002	3.99	50.14	27.31	0.52
FML Chute	4175	250 CFS	250.00	289.96	292.82		293.10	0.002996	4.24	58.93	29.17	0.53
FML Chute	4175	300 CFS	300.00	289.96	293.09		293.40	0.002998	4.46	67.23	30.83	0.53
FML Chute	4170	50 CFS	50.00	289.94	291.15		291.26	0.003000	2.64	18.91	19.26	0.47
FML Chute	4170	100 CFS	100.00	289.94	291.71		291.87	0.003001	3.27	30.62	22.61	0.49
FML Chute	4170	150 CFS	150.00	289.94	292.14		292.35	0.003000	3.68	40.81	25.17	0.51
FML Chute	4170	200 CFS	200.00	289.94	292.49		292.74	0.003002	3.99	50.14	27.31	0.52
FML Chute	4170	250 CFS	250.00	289.94	292.80		293.08	0.002996	4.24	58.93	29.17	0.53
FML Chute	4170	300 CFS	300.00	289.94	293.08		293.39	0.002998	4.46	67.23	30.83	0.53
FML Chute	4160	50 CFS	50.00	289.91	291.12		291.23	0.003000	2.64	18.91	19.26	0.47
FML Chute	4160	100 CFS	100.00	289.91	291.68		291.84	0.003001	3.27	30.62	22.61	0.49
FML Chute	4160	150 CFS	150.00	289.91	292.11		292.32	0.003000	3.68	40.81	25.17	0.51
FML Chute	4160	200 CFS	200.00	289.91	292.46		292.71	0.003002	3.99	50.14	27.31	0.52
FML Chute	4160	250 CFS	250.00	289.91	292.77		293.05	0.002996	4.24	58.93	29.17	0.53
FML Chute	4160	300 CFS	300.00	289.91	293.05		293.36	0.002998	4.46	67.23	30.83	0.53
FML Chute	4150	50 CFS	50.00	289.88	291.09		291.20	0.003000	2.64	18.91	19.26	0.47
FML Chute	4150	100 CFS	100.00	289.88	291.65		291.81	0.003001	3.27	30.62	22.61	0.49
FML Chute	4150	150 CFS	150.00	289.88	292.08		292.29	0.003000	3.68	40.81	25.17	0.51
FML Chute	4150	200 CFS	200.00	289.88	292.43		292.68	0.003002	3.99	50.14	27.31	0.52
FML Chute	4150	250 CFS	250.00	289.88	292.74		293.02	0.002996	4.24	58.93	29.17	0.53
FML Chute	4150	300 CFS	300.00	289.88	293.02		293.33	0.002998	4.46	67.23	30.83	0.53
FML Chute	4140	50 CFS	50.00	289.85	291.06		291.17	0.003000	2.64	18.91	19.26	0.47
FML Chute	4140	100 CFS	100.00	289.85	291.62		291.78	0.003001	3.27	30.62	22.61	0.49
FML Chute	4140	150 CFS	150.00	289.85	292.05		292.26	0.003000	3.68	40.81	25.17	0.51
FML Chute	4140	200 CFS	200.00	289.85	292.40		292.65	0.003003	3.99	50.13	27.31	0.52
FML Chute	4140	250 CFS	250.00	289.85	292.71		292.99	0.002996	4.24	58.93	29.17	0.53
FML Chute	4140	300 CFS	300.00	289.85	292.99		293.30	0.002998	4.46	67.23	30.83	0.53
FML Chute	4130	50 CFS	50.00	289.82	291.03		291.14	0.003000	2.64	18.91	19.26	0.47
FML Chute	4130	100 CFS	100.00	289.82	291.59		291.75	0.003001	3.27	30.62	22.61	0.49
FML Chute	4130	150 CFS	150.00	289.82	292.02		292.23	0.003000	3.68	40.81	25.17	0.51
FML Chute	4130	200 CFS	200.00	289.82	292.37		292.62	0.003003	3.99	50.13	27.31	0.52
FML Chute	4130	250 CFS	250.00	289.82	292.68		292.96	0.002996	4.24	58.93	29.17	0.53
FML Chute	4130	300 CFS	300.00	289.82	292.96		293.27	0.002997	4.46	67.23	30.84	0.53
FML Chute	4120	50 CFS	50.00	289.79	291.00		291.11	0.003000	2.64	18.91	19.26	0.47
FML Chute	4120	100 CFS	100.00	289.79	291.56		291.72	0.003001	3.27	30.62	22.61	0.49
FML Chute	4120	150 CFS	150.00	289.79	291.99		292.20	0.003000	3.68	40.81	25.17	0.51

FML Chute	4120	200 CFS	200.00	289.79	292.34		292.59	0.003003	3.99	50.13	27.31	0.52
FML Chute	4120	250 CFS	250.00	289.79	292.65		292.93	0.002996	4.24	58.93	29.17	0.53
FML Chute	4120	300 CFS	300.00	289.79	292.93		293.24	0.002997	4.46	67.23	30.84	0.53
FML Chute	4110	50 CFS	50.00	289.76	290.97		291.08	0.003000	2.64	18.91	19.26	0.47
FML Chute	4110	100 CFS	100.00	289.76	291.53		291.69	0.003002	3.27	30.62	22.61	0.49
FML Chute	4110	150 CFS	150.00	289.76	291.96		292.17	0.003000	3.68	40.81	25.17	0.51
FML Chute	4110	200 CFS	200.00	289.76	292.31		292.56	0.003003	3.99	50.13	27.31	0.52
FML Chute	4110	250 CFS	250.00	289.76	292.62		292.90	0.002995	4.24	58.93	29.18	0.53
FML Chute	4110	300 CFS	300.00	289.76	292.90		293.21	0.002997	4.46	67.24	30.84	0.53
FML Chute	4100	50 CFS	50.00	289.73	290.94		291.05	0.003000	2.64	18.91	19.26	0.47
FML Chute	4100	100 CFS	100.00	289.73	291.50		291.66	0.003002	3.27	30.62	22.61	0.49
FML Chute	4100	150 CFS	150.00	289.73	291.93		292.14	0.003000	3.68	40.81	25.17	0.51
FML Chute	4100	200 CFS	200.00	289.73	292.28		292.53	0.003003	3.99	50.13	27.30	0.52
FML Chute	4100	250 CFS	250.00	289.73	292.59		292.87	0.002995	4.24	58.93	29.18	0.53
FML Chute	4100	300 CFS	300.00	289.73	292.87		293.18	0.002997	4.46	67.24	30.84	0.53
FML Chute	4000	50 CFS	50.00	289.43	290.64		290.75	0.002994	2.64	18.92	19.26	0.47
FML Chute	4000	100 CFS	100.00	289.43	291.20	290.60	291.36	0.003004	3.27	30.61	22.61	0.50
FML Chute	4000	150 CFS	150.00	289.43	291.63		291.84	0.003001	3.68	40.81	25.17	0.51
FML Chute	4000	200 CFS	200.00	289.43	291.98	291.18	292.23	0.003006	3.99	50.11	27.30	0.52
FML Chute	4000	250 CFS	250.00	289.43	292.29		292.57	0.002992	4.24	58.96	29.18	0.53
FML Chute	4000	300 CFS	300.00	289.43	292.57	291.64	292.88	0.002995	4.46	67.25	30.84	0.53
FML Chute	3000	50 CFS	50.00	286.43	287.64	287.19	287.75	0.003001	2.64	18.91	19.26	0.47
FML Chute	3000	100 CFS	100.00	286.43	288.20	287.60	288.36	0.003002	3.27	30.62	22.61	0.49
FML Chute	3000	150 CFS	150.00	286.43	288.63	287.91	288.84	0.003004	3.68	40.80	25.17	0.51
FML Chute	3000	200 CFS	200.00	286.43	288.98	288.18	289.23	0.003001	3.99	50.14	27.31	0.52
FML Chute	3000	250 CFS	250.00	286.43	289.29	288.42	289.57	0.003000	4.24	58.90	29.17	0.53
FML Chute	3000	300 CFS	300.00	286.43	289.57	288.64	289.88	0.003004	4.47	67.18	30.82	0.53

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 08/2005 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 at 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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www.ivindustries.com

IVI-EC-3442-0401-0142

Type	Thickness m ft	Size mm	d_{50} m ft	velocity m/s ft/sec	velocity m/s ft/sec
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 - 250	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 performances 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 mattress covered, the lining will be completely impervious. The 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].

SIZE OF RIPRAP TO BE USED DOWNSTREAM FROM STILLING BASINS

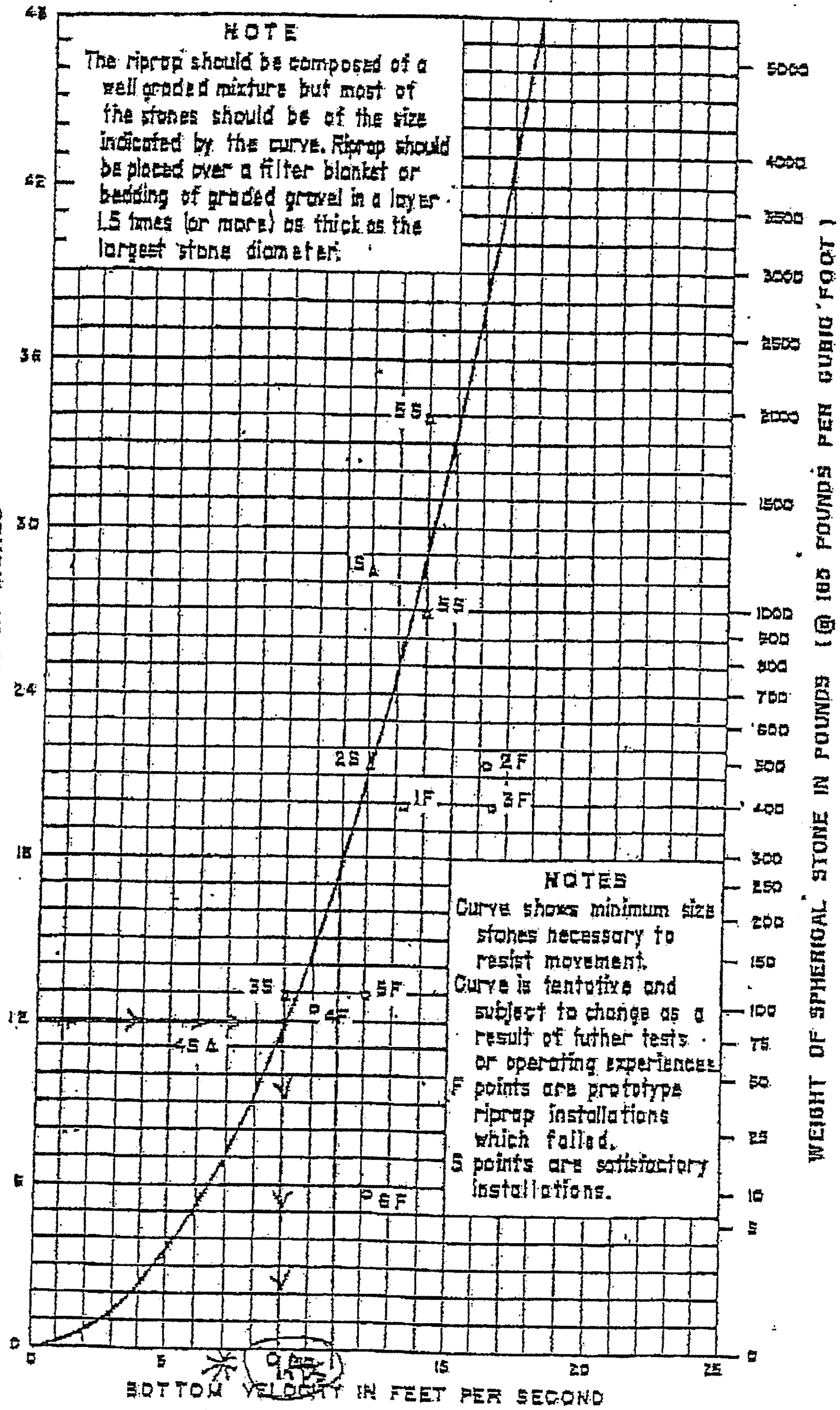


FIGURE 165.—Curve to determine minimum stone size in riprap mixture.

SOURCE: HYDRAULIC DESIGN OF STILLING BASINS AND ENERGY DISSIPATORS. US DEPT OF THE INTERIOR - BUREAU OF LAND RECLAMATION, 1964.

Required: Determine the maximum drainage area for 24-inch and 36-inch diameter letdown pipes using the BCAP computer program.

Method:
1. Determine the maximum flow for 24-inch and 36-inch diameter letdown pipes on the 33% 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, September 2019.

Note: The pipe letdown analysis has been performed using "Broken-Back" Culvert Analysis Program (BCAP) which is available from the Federal Highway Administration Web Page:

http://www.dor.state.ne.us/roadway-design/ [follow link to downloadable files and info]

The program was developed to analyze culverts with changing slopes.

Solution:
1. Determine the maximum flow for 24-inch and 36-inch diameter letdown pipes on the 33% side slope.

The following pages include the program outputs for the 24-in dia culvert and 36-in diameter culvert. Pages IIIF-F-2-26 and IIIF-F-2-31 include rating tables that show if the hydraulic jump occurs within the pipe or not [YES/NO]. 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 flow ratings are used to calculate the maximum allowable top dome drainage area for each pipe size analyzed (Step 2). The maximum flow rate that has hydraulic jump within the culvert is used for allowable drainage area calculations on page IIIF-F-2-35. The computer program does not have corrugated plastic pipe option; therefore, the corrugated metal pipe option has been used with a Manning's Coefficient of 0.024.

Results:

Q24 =	22.6	cfs	maximum allowable flow in 24-in-dia pipe
Q36 =	32.0	cfs	maximum allowable flow in 36-in-dia pipe

NEBRASKA DEPARTMENT OF ROADS
Broken-Back Culvert Analysis Program (BCAP)

PROJECT INFO

Project:	FW C&D LANDFILL EXPANSION
Station or Location:	TARRANT COUNTY, TEXAS
Date:	07 / 21 / 2022

DISCHARGE DATA

Minimum:	1.00 cfs
Design Discharge:	20.00 cfs
Maximum:	25.00 cfs
Number of Barrels:	1

TAILWATER DATA

Type:	Downstream
Channel Shape:	Trapezoid
Left Side Slope:	3 H:1V
Right Side Slope:	3 H:1V
Bottom Width:	10 ft
Bottom Slope:	0.005 ft/ft
Roughness Coefficient:	0.04

CULVERT DATA

Type:	Circular Pipe
Pipe Diameter:	2 ft
Culvert Material:	Corr. Metal Pipe
Inlet Type:	Mitered to Conform to Slope
Roughness Coefficient:	0.024
Outlet Section Roughness Coeff.:	0.024
Inlet Section Slope:	0 ft/ft
Steep Section Slope:	0.3333 ft/ft
Outlet Section Slope:	0 ft/ft

CULVERT PROFILE DATA

Type:	Double Broken-Back
Inlet Station:	100.00 ft
Inlet Elevation:	860.00 ft
Upper Break Station:	110.00 ft
Upper Break Elevation:	860.00 ft
Lower Break Station:	860.00 ft
Lower Break Elevation:	610.00 ft
Outlet Station:	897.50 ft
Outlet Elevation:	610.00 ft

NEBRASKA DEPARTMENT OF ROADS
Broken-Back Culvert Analysis Program (BCAP)

Project: FW C&D LANDFILL EXPANSION
Station or Location: TARRANT COUNTY, TEXAS
Date: 07/21/2022

Discharge (cfs)	Headwater Depth (ft)	Inlet Control Elevation (ft)	Break Control Elevation (ft)	Critical Depth (ft)	Outlet Depth (ft)	Outlet Velocity (ft/s)	Outlet Froude Number	Tailwater Depth (ft)	Tailwater Velocity (ft/s)	Hydraulic Jump
3.4	1.05	860.90	861.05	.65	.65	3.83	1.0	.29	1.08	YES
5.8	1.40	861.24	861.40	.85	.85	4.56	1.0	.39	1.33	YES
8.2	1.70	861.51	861.70	1.01	1.01	5.15	1.0	.49	1.46	YES
10.6	1.97	861.78	861.97	1.15	1.15	5.67	1.0	.56	1.62	YES
13.0	2.24	862.08	862.24	1.27	1.27	6.16	1.0	.63	1.74	YES
15.4	2.51	862.43	862.51	1.39	1.39	6.63	1.0	.69	1.85	YES
20.0	3.27	863.27	863.15	1.58	1.58	7.52	1.0	.81	1.99	YES
20.2	3.31	863.31	863.18	1.59	1.59	7.56	1.0	.81	2.01	YES
22.6	3.84	863.84	863.57	1.68	2.00	7.19	.7	.87	2.06	YES
25.0	4.43	864.43	863.98	1.77	2.00	7.96	.8	.91	2.16	NO

NEBRASKA DEPARTMENT OF ROADS
Broken-Back Culvert Analysis Program (BCAP)

PROJECT INFO

Project:	FW C&D LANDFILL EXPANSION
Station or Location:	TARRANT COUNTY, TEXAS
Date:	07/21/2022

CULVERT DATA

Discharge:	20.0 cfs
Shape:	Circular
Material:	Corr. Metal Pipe
Size:	1-2.0 ft x 2.0 ft
Inlet Type:	Mitered to Conform to Slope

WATER SURFACE PROFILE

Inlet Depth:	2.00 ft
Inlet Velocity:	6.37 ft/s
Upper Break Depth:	1.58 ft
Upper Break Velocity:	7.52 ft/s
Lower Break Depth:	0.72 ft
Lower Break Velocity:	19.64 ft/s
Depth at End of Hydraulic Jump:	2.00 ft
Velocity at End of Hydraulic Jump:	6.37 ft/s
Depth at End of Hydraulic Jump:	0.81 ft
Velocity at End of Hydraulic Jump:	1.99 ft/s

OUTPUT DATA

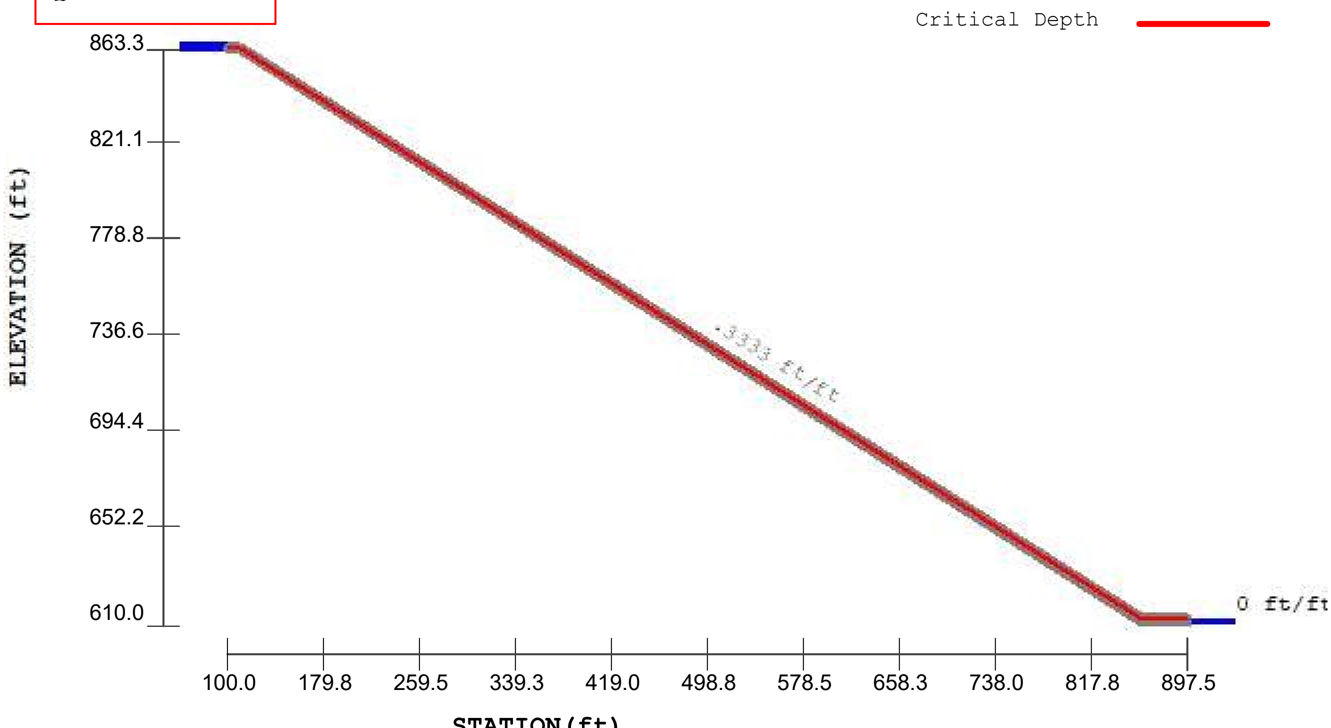
Head Water Depth:	3.27 ft
Inlet Control Elevation:	863.27 ft
Break Control Elevation:	863.15 ft
Critical Depth:	1.58 ft
Tailwater Depth:	0.81 ft
Hydraulic Jump?	YES
Jump Station:	884.84 ft
Jump Length:	12.31 ft
Outlet Depth:	1.58 ft
Outlet Velocity:	7.52 ft/s
Outlet Froude No.:	1.0

Project: FW C&D LANDFILL EXPANSION   Location: TARRANT COUNTY, TEXAS

Circle Pipe Culvert
Diameter=2 ft
Culvert Material: Corr. Metal Pipe

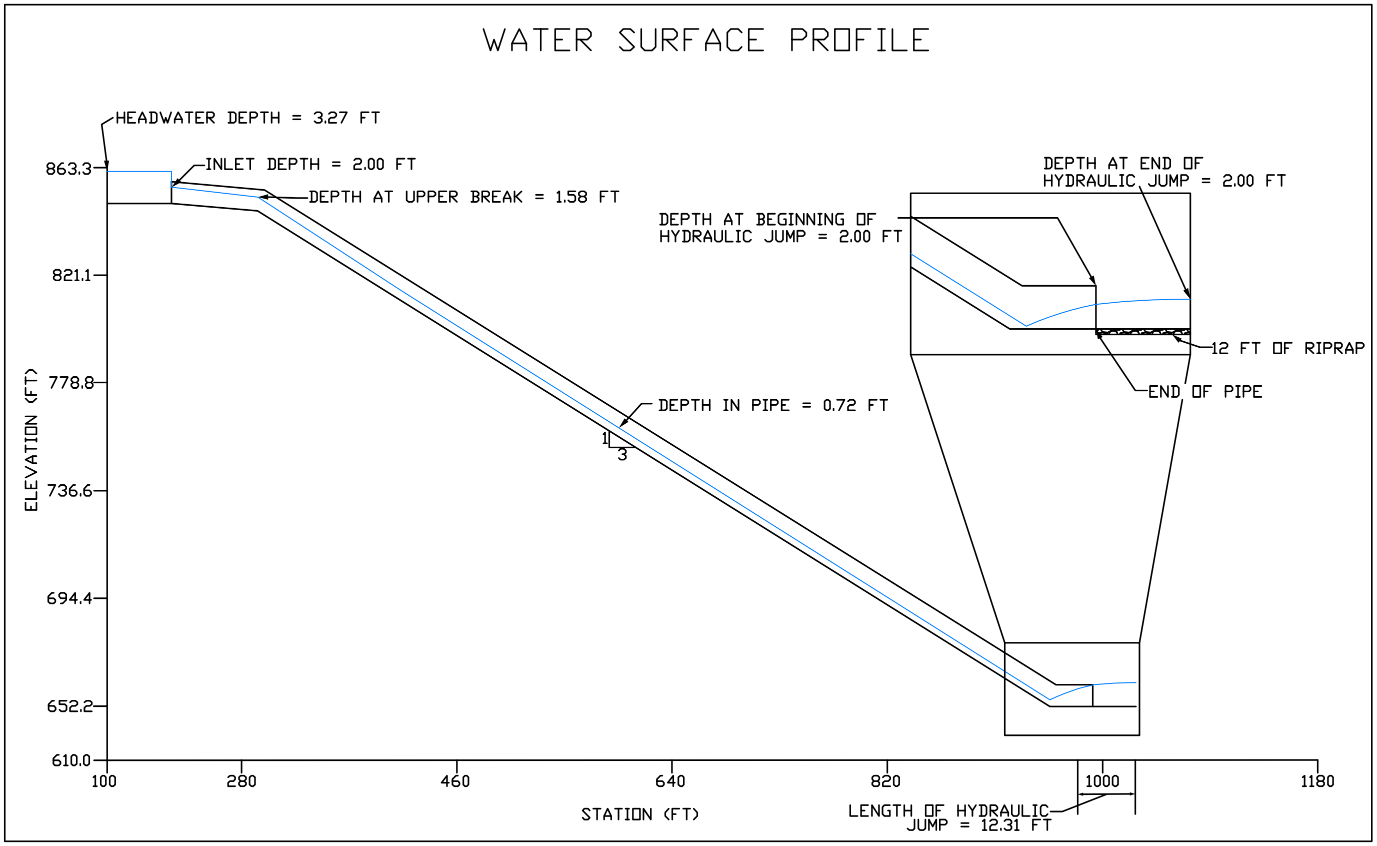
Inlet Type
Mitered to Conform to Slope
Rough. Coeff.= 0.024
Outlet Sec. Rough. Coeff.= 0.024

Q = 20 cfs



Press to Return to Output Form

Source: P:\SOLID WASTE\WC\FW CD\EXPANSION 2022\PART III\I CAP 2FT



IIIF-F2-29

P:\Solid waste\WC\FW C&D\Expansion 2022\Part III\IIIF\IIIF-F\Pipe Water Profile 2 ft.dwg, eroulon, 1:1

NEBRASKA DEPARTMENT OF ROADS
Broken-Back Culvert Analysis Program (BCAP)

PROJECT INFO

Project:	FORT WORTH C&D LANDFILL EXPANSION
Station or Location:	TARRANT COUNTY, TEXAS
Date:	07 / 21 / 2022

DISCHARGE DATA

Minimum:	5.00 cfs
Design Discharge:	25.00 cfs
Maximum:	35.00 cfs
Number of Barrels:	1

TAILWATER DATA

Type:	Downstream
Channel Shape:	Trapezoid
Left Side Slope:	3 H:1V
Right Side Slope:	3 H:1V
Bottom Width:	10 ft
Bottom Slope:	0.005 ft/ft
Roughness Coefficient:	0.04

CULVERT DATA

Type:	Circular Pipe
Pipe Diameter:	3 ft
Culvert Material:	Corr. Metal Pipe
Inlet Type:	Mitered to Conform to Slope
Roughness Coefficient:	0.024
Outlet Section Roughness Coeff.:	0.024
Inlet Section Slope:	0 ft/ft
Steep Section Slope:	0.3333 ft/ft
Outlet Section Slope:	0 ft/ft

CULVERT PROFILE DATA

Type:	Double Broken-Back
Inlet Station:	100.00 ft
Inlet Elevation:	860.00 ft
Upper Break Station:	110.00 ft
Upper Break Elevation:	860.00 ft
Lower Break Station:	860.00 ft
Lower Break Elevation:	610.00 ft
Outlet Station:	910.00 ft
Outlet Elevation:	610.00 ft

NEBRASKA DEPARTMENT OF ROADS
Broken-Back Culvert Analysis Program (BCAP)

Project:	FORT WORTH C&D LANDFILL EXPANSION
Station or Location:	TARRANT COUNTY, TEXAS
Date:	07/21/2022

Discharge cfs	Headwater Depth ft	Inlet Control Elevation ft	Break Control Elevation ft	Critical Depth ft	Outlet Depth ft	Outlet Velocity ft/s	Outlet Froude Number	Tailwater Depth ft	Tailwater Velocity ft/s	Hydraulic Jump
8.0	1.43	861.21	861.43	.90	.90	4.47	1.0	.47	1.49	YES
11.0	1.69	861.50	861.69	1.06	1.06	4.94	1.0	.57	1.65	YES
14.0	1.93	861.73	861.93	1.19	1.19	5.34	1.0	.65	1.80	YES
17.0	2.15	861.93	862.15	1.32	1.32	5.70	1.0	.73	1.91	YES
20.0	2.36	862.11	862.36	1.43	1.43	6.04	1.0	.81	1.99	YES
25.0	2.68	862.41	862.68	1.59	1.59	6.55	1.0	.91	2.16	YES
26.0	2.74	862.47	862.74	1.63	1.63	6.64	1.0	.93	2.19	YES
29.0	2.92	862.66	862.92	1.72	1.72	6.93	1.0	.99	2.26	YES
32.0	3.10	862.85	863.10	1.80	1.99	6.45	.8	1.05	2.32	YES
35.0	3.28	863.06	863.28	1.89	2.02	6.93	.9	1.11	2.37	NO

NEBRASKA DEPARTMENT OF ROADS
Broken-Back Culvert Analysis Program (BCAP)

PROJECT INFO

Project:	FORT WORTH C&D LANDFILL EXPANSION
Station or Location:	TARRANT COUNTY, TEXAS
Date:	07/21/2022

CULVERT DATA

Discharge:	25.0 cfs
Shape:	Circular
Material:	Corr. Metal Pipe
Size:	1-3.0 ft x 3.0 ft
Inlet Type:	Mitered to Conform to Slope

WATER SURFACE PROFILE

Inlet Depth:	1.89 ft
Inlet Velocity:	5.31 ft/s
Upper Break Depth:	1.59 ft
Upper Break Velocity:	6.55 ft/s
Lower Break Depth:	0.70 ft
Lower Break Velocity:	19.94 ft/s
Depth at End of Hydraulic Jump:	2.08 ft
Velocity at End of Hydraulic Jump:	4.79 ft/s
Depth at End of Hydraulic Jump:	0.91 ft
Velocity at End of Hydraulic Jump:	2.16 ft/s

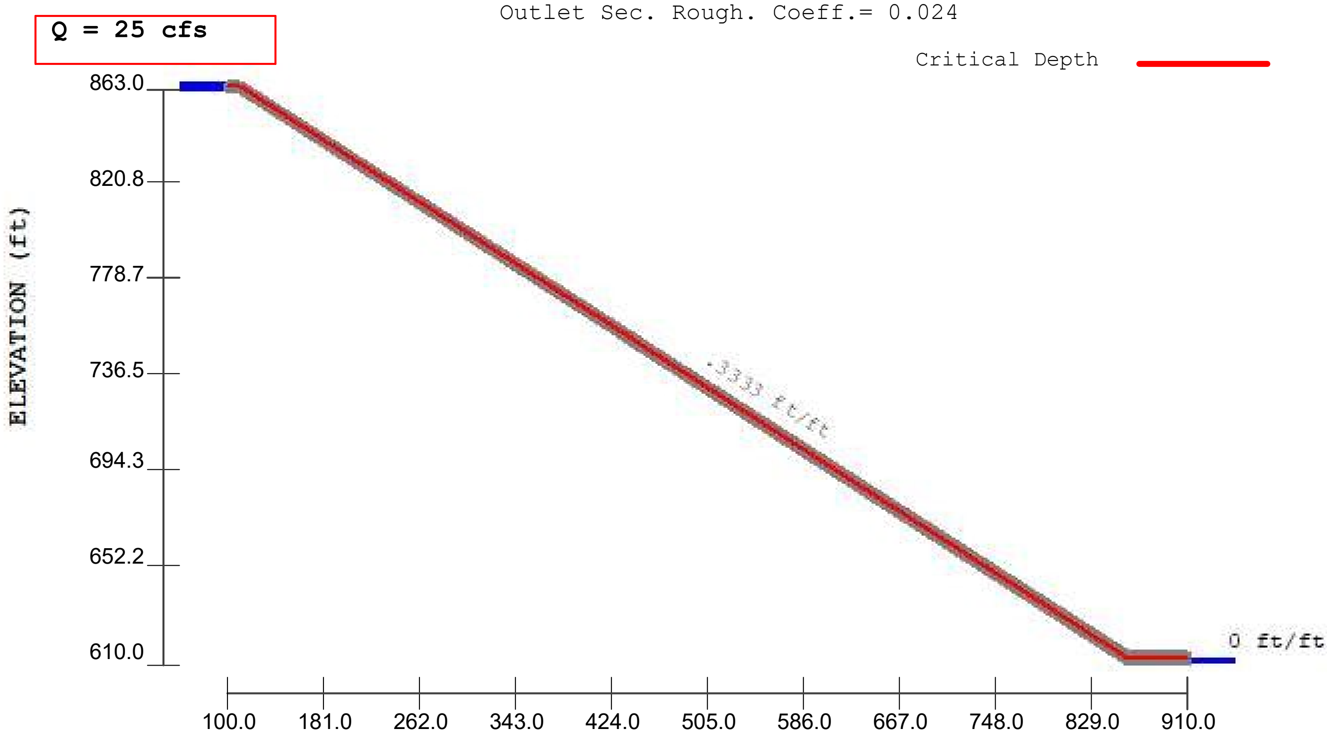
OUTPUT DATA

Head Water Depth:	2.68 ft
Inlet Control Elevation:	862.41 ft
Break Control Elevation:	862.68 ft
Critical Depth:	1.59 ft
Tailwater Depth:	0.91 ft
Hydraulic Jump?	YES
Jump Station:	894.76 ft
Jump Length:	12.46 ft
Outlet Depth:	1.59 ft
Outlet Velocity:	6.55 ft/s
Outlet Froude No.:	1.0

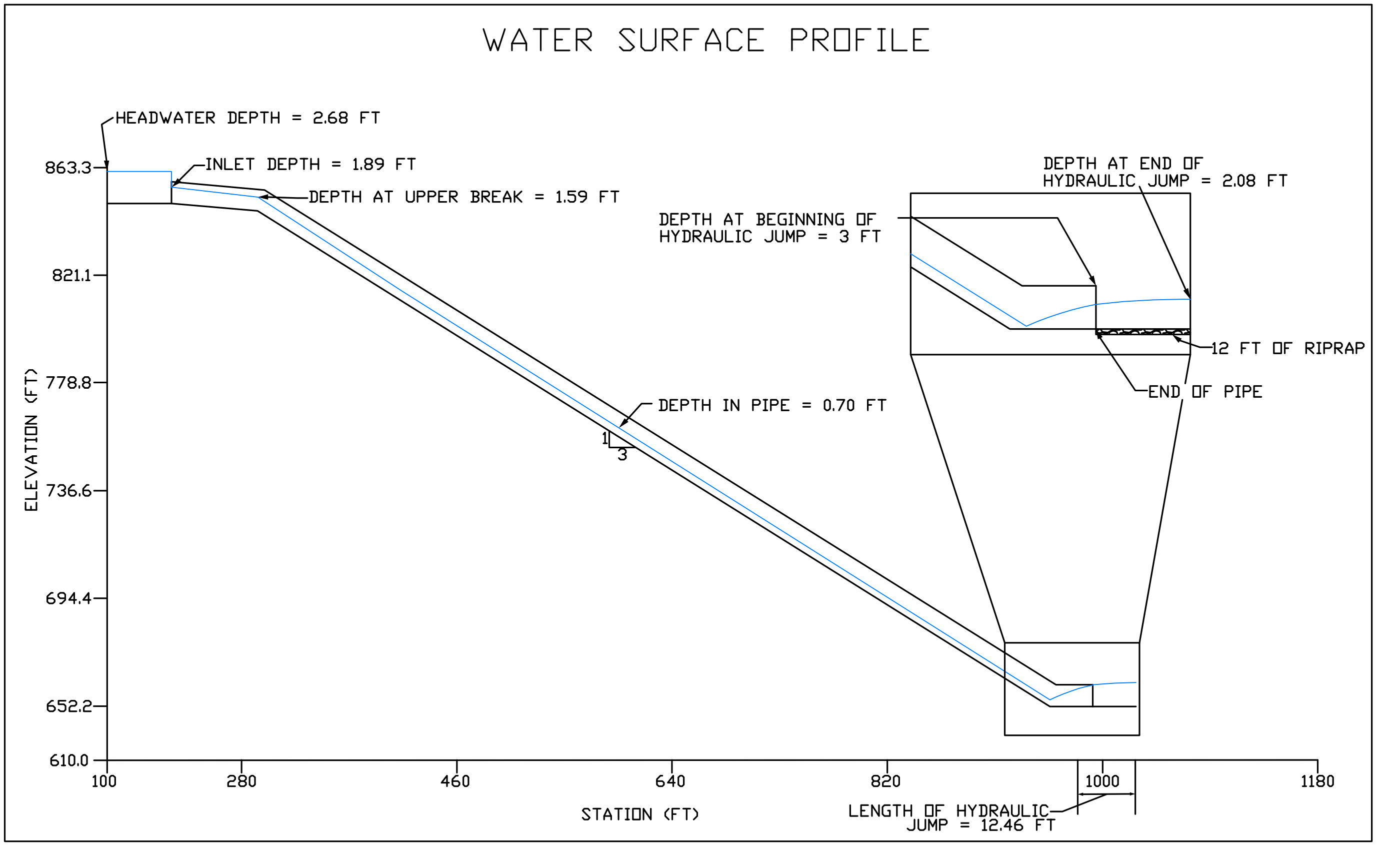
Project: FORT WORTH C&D LANDFILL EXPANSION **Location: TARRANT COUNT**

Circle Pipe Culvert
Diameter=3 ft
Culvert Material: Corr. Metal Pipe

Inlet Type
Mitered to Conform to Slope
Rough. Coeff.= 0.024
Outlet Sec. Rough. Coeff.= 0.024



Source: P:\SOLID WASTE\WC\FW CD\EXPANSION 2022\PART III\I Press to Return to Output Form CAP 3FT



IIIF-F2-34

P:\Solid waste\WC\FW C&D\Expansion 2022\Part III\IIIF\IIIF-F\Pipe Water Profile 3 ft.dwg, eroulon, 1:1

2. Determine the maximum drainage areas for the flows calculated in Step 1.

$$Q = CIA$$

Where: C= 0.7 (runoff coefficient, Ref 1.)
I = intensity, in/hr
A= drainage area, ac

$$I = \frac{b}{(t_c + d)^e}$$

b = 79.18 From Ref. 1, for Tarrant County
d = 10.44 25-year storm event
e = 0.772

t_c is assumed to be 10 min.

I = 7.71 in/hr

$$A = Q / (CI)$$

Pipe Diameter (in)	Flow (cfs)	Area (ac)
24	22.6	4.2
36	32.0	5.9

Conclusion: The maximum allowable drainage area for a 24-inch diameter letdown pipe is 4.2 acres for each inlet and for a 36-inch diameter letdown pipe is 5.9 acres for each inlet. The minimum berm height is 3 feet for a 24-inch diameter pipe and 4 feet for 36-inch diameter pipe. (Figure 3 details indicate 1 foot berm above the pipe).

Required: Determine the Riprap size and Dimensions for 24-inch and 36-inch diameter letdown pipes using Riprap Apron Design provided by the Reference 1.

Method:

1. Determine the hydraulic conditions at the outlet of 24-inch and 36-inch diameter letdown pipes using the hydraulic design developed using the BCAP computer simulation.
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-2-27 (pipe diameter 24-inches) and IIIF-F-2-32 (pipe diameter 36-inches):

Parameter	Symbol	24-inch Dia. Culvert	36-inch Dia. Culvert
Design flow rates, cfs	Q=	22.6	32.0
Pipe Diameters, ft	D=	2.00	3.00
Depth at the pipe outlet, ft	y_n=	2.00	2.08
Adjusted culvert rise, ft	D'=	2.00	3.12
Tailwater Depth[1], ft	TW=	0.81	0.91

[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)

$$D' = \frac{D \times y_n}{2}$$ 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	24-inch Dia. Culvert	36-inch Dia. Culvert
D_{50}, calculated, inches =	7.4	6.1
D_{50}, selected, inches =	10	12
Apron Length, calculated, feet =	10	15
Apron Length, selected, feet =	12	18
Apron Depth, calculated, inches =	24.0	28.8
Apron Depth, selected, inches =	30	30

Conclusion:

Riprap sizes for pipe diameters of 24-inches and 36-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 dissipator calculations are acceptable and channels at the pipe outlets will be stable.

APPENDIX IIIF-F-3

SEDIMENT CONTROL POND DESIGN

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 Sheet IIIF-F-13 (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/acres/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. 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 required 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 discharge of eroded soil to less than the allowable amount for external side slopes (i.e., 50 tons/acre/year) if the required percent vegetation coverage is not obtained soil loss is greater than 50 tons/acre/year. 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 0.5-acre detention pond is capable of reducing the discharge of 60 tons/acre/year of soil to less than 50 tons/acre/year of soil 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 Sheet IIIF-F-13 – Sediment Control Pond Plan as well as the table on page IIIF-F-3-7.

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/yea or less.

Reference:

1. State of Texas, Department of Transportation, Bridge Division, Hydraulic Manual, 3rd Edition, September 2019.
2. NOAA Atlas 14 - Precipitation-Frequency Atlas of the United States, Volume 11, Version 2.0: Texas (U.S. Department of Commerce, National Oceanic and Atmospheric Administration and National Weather Service, 2018)
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 = 0.7 (runoff coefficient, Ref. 1)
I = intensity (in/hr)
A = upstream drainage area (ac)

Note: A runoff coefficient of 0.7 is used for all areas regardless of slope

$$I = \frac{b}{(t_c + d)^e}$$

b = 79.18
d = 10.44
e = 0.772

From Ref. 2, for Tarrant County 25-year frequency storm event

t_c is assumed to be 10 min.

I = 7.71 in/hr

A = 20.0 acres

Q = 107.91 cfs

2. Calculate the settling velocity, V_s (ft/hr), of sediment particles using Stokes equation.

$$V_s = \frac{\alpha (\rho_s/\rho_w - 1) g\phi^2}{18 v_w} \quad \text{(Ref. 3)}$$

Where:

α = factor that measures the effect of particle shape (assume spherical, α = 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)

α = 1
ρ_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. 4)

Where:

V_c = overflow rate
A_p = area of sediment control pond (ac)

Q = 107.91 cfs (from Step 2)
A_p = 0.50 acre

V_c = 17.84 ft/hr

b. Determine the fraction of sediment removed.

$F = 1 - (1 + 1/\beta * V_s/V_c)^{-\beta}$ (Ref. 4)

Where:

F = single-storm trapping of sediment
β = turbulence or short-circuiting parameter reflecting non-ideal performance of pond (assume good performance, β = 3)

β = 3

$D_R = L_F\,[(1/CV_Q^2) / (1/CV_Q^2 - \ln(E_m/L_F))]^{(1/CVQ\hat{}2)+1}$ (Ref. 4)

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 = 1.74 (from Table 9B.1, p. 570, Ref. 4)

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	0.027	0.27
2	20	1.97E-05	0.38	0.021	0.034	0.68
3	30	2.62E-05	0.68	0.037	0.041	1.24
4	20	3.28E-05	1.06	0.057	0.049	0.98
5	20	3.94E-05	1.52	0.081	0.057	1.14
Total	**100**					**4.3**

[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, 24-hour storm (ft)
A = upstream drainage area (ac)

A_Q = 21,780 ft^2 (assume equal to A_p)
T_{IA} = 108 hrs (from Table 9B.1, p. 570 of Ref. 4)
R = 0.61 ft (Ref. 5)
A = 20.0 ac (from Step 1)

V_R = 529,980 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]	Removed Under Quiescent Conditions[2]	Fraction Captured Under Quiescent Conditions, E_Q
1	10	0.17	0.75	0.120	0.12	1.20
2	20	0.38	1.69	0.130	0.12	2.40
3	30	0.68	3.00	0.140	0.13	3.90
4	20	1.06	4.68	0.145	0.14	2.80
5	20	1.52	6.74	0.150	0.15	3.00
Total	**100**					**13.3**

[1] Based on Figure 9.29 from Ref. 4, using RR and V_B/V_R.
V_B = reservoir volume = 87,120 ft^3, assuming a 0.5-acre pond with an average depth of 4 feet.
V_B/V_R = 0.164
[2] Based on Figure 9.30 from Ref. 4 with CV_R = 1.74.

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 = 17.0 %

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 =	17.0	%	(from Step 5)

Net Soil Loss = Total Soil Loss x (1 - E_T/100)

Net Soil Loss = 49.8 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 =	0.5	acres	(from Step 3)

Required Pond Size / Unit Drainage Area Factor = 0.025

This factor was calculated using a drainage area of 20 acres and a pond area of 0.5 acres. If a 40-acre drainage area drains to the pond, then a 1.0-acre pond will be required to achieve the above efficiency and net soil loss estimate (40 acres x 0.025 = 1.0 acre). Refer to page IIIF-F-3-7 for the required pond size/unit drainage area factor for different soil loss reduction amounts.

Conclusion:

A 0.5-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.

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	4.3	13.3	17.0	49.8	0.025	YES
70	5.1	25.5	29.3	49.5	0.040	YES
80	6.1	34.0	38.1	49.6	0.060	YES
90	6.9	41.5	45.5	49.0	0.075	YES
100	8.4	46.4	50.9	49.1	0.110	YES
200	16.4	71.2	75.9	48.2	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.

APPENDIX IIIF-G

EXCERPTS FROM APPROVED CLOMR

STATE OF TEXAS

CHARLES R. MARSH

105073

LICENSED

PROFESSIONAL ENGINEER

02/09/2023

CONTENTS

STATE OF TEXAS
CHARLES R. MARSH
105073
LICENSED
PROFESSIONAL ENGINEER
02/09/2023

FLOODPLAIN SUMMARY

This appendix addresses §330.61(m).

As discussed in Parts I/II in Section 11, Parts I/II-Appendix I/IIC, and Part III-Appendix IIIF, the floodplain for Fort Worth C&D Landfill is located west of the landfill area. A Conditional Letter of Map Revision (CLOMR) was developed for the proposed expansion to revise the floodplain limits as a part of the proposed landfill development.

Excerpts from the CLOMR are included in Appendix IIIF-G-A. As shown in Appendix IIIF-G-A, the proposed solid waste fill areas will not be located within the limits of the post-development 100-year floodplain in the approved CLOMR.

APPENDIX IIIF-G-A

EXCERPTS FROM THE APPROVED CLOMR APPLICATION

Federal Emergency Management Agency

Washington, D.C. 20472

September 23, 2022

CERTIFIED MAIL
RETURN RECEIPT REQUESTED

IN REPLY REFER TO:
Case No.: 22-06-1554R
Community Name: City of Fort Worth, TX
Community No.: 480596

104

The Honorable Mattie Parker
Mayor, City of Fort Worth
200 Texas Street
Fort Worth, TX 76102

Dear: Mayor Parker:

We are providing our comments with the enclosed Conditional Letter of Map Revision (CLOMR) on a proposed project within your community that, if constructed as proposed, could revise the effective Flood Insurance Study report and Flood Insurance Rate Map (FIRM) for your community.

If you have any questions regarding the floodplain management regulations for your community, the National Flood Insurance Program (NFIP) in general, or technical questions regarding this CLOMR, please contact the Director, Mitigation Division of the Federal Emergency Management Agency (FEMA) Regional Office in Denton, Texas, at (940) 898-5127, or the FEMA Mapping and Insurance eXchange (FMIX) toll free at 1-877-336-2627 (1-877-FEMA MAP). Additional information about the NFIP is available on our website at https://www.fema.gov/flood-insurance.

Sincerely,

Patrick "Rick" F. Sacbibit, P.E., Branch Chief
Engineering Services Branch
Federal Insurance and Mitigation Administration

Enclosure:
Conditional Letter of Map Revision Comment Document

cc: The Honorable B. Glen Whitley
Tarrant County Judge

Joseph Jackson, P.E., CFM
Floodplain Administrator
Tarrant County

Clair C. Davis, P.E., CFM
Floodplain Administrator
City of Fort Worth

Charles R. Marsh, P.E.
Project Director
Weaver Consultants Group, LLC

 | **Issue Date: September 23, 2022** | **Case No.: 22-06-1554R** | CLOMR-APP

Federal Emergency Management Agency
Washington, D.C. 20472

CONDITIONAL LETTER OF MAP REVISION
COMMENT DOCUMENT

COMMUNITY INFORMATION		PROPOSED PROJECT DESCRIPTION	BASIS OF CONDITIONAL REQUEST
COMMUNITY	**City of Fort Worth** **Tarrant County** **Texas** **COMMUNITY NO.: 480596**	FILL EXCAVATION	1D HYDRAULIC ANALYSIS FLOODWAY UPDATED TOPOGRAPHIC DATA
IDENTIFIER	Fort Worth C&D Landfill	**APPROXIMATE LATITUDE AND LONGITUDE:** 32.633, -97.243 **SOURCE:** OTHER **DATUM:** NAD 83	

AFFECTED MAP PANELS			
TYPE: FIRM*	NO.: 48439C0340K	DATE: 9/25/2009	
TYPE: FIRM*	NO.: 48439C0320L	DATE: 3/21/2019	* FIRM - Flood Insurance Rate Map

FLOODING SOURCE AND REACH DESCRIPTION

Village Creek – From the downstream side of County Road 2051 to approximately 1,550 feet downstream of Shelby Road

PROPOSED PROJECT DESCRIPTION

Flooding Source	Proposed Project	Location of Proposed Project
Village Creek	Fill Placement	From approximately 2,260 feet upstream of County Road 2051 to approximately 5,930 feet upstream of County Road 2051
	Excavation	From approximately 2,260 feet upstream of County Road 2051 to approximately 5,930 feet upstream of County Road 2051

SUMMARY OF IMPACTS TO FLOOD HAZARD DATA

Flooding Source	Effective Flooding	Proposed Flooding	Increases	Decreases
Village Creek	Floodway	Floodway	Yes	Yes
	Zone AE	Zone AE	Yes	Yes
	Zone X (shaded)	Zone X (shaded)	Yes	Yes
	BFEs*	BFEs	Yes	Yes

* BFEs - Base (1-percent-annual-chance) Flood Elevations

COMMENT

This document provides the Federal Emergency Management Agency's (FEMA's) comment regarding a request for a CLOMR for the project described above. This document is not a final determination; it only provides our comment on the proposed project in relation to the flood hazard information shown on the effective National Flood Insurance Program (NFIP) map. We reviewed the submitted data and the data used to prepare the effective flood hazard information for your community and determined that the proposed project meets the minimum floodplain management criteria of the NFIP. Your community is responsible for approving all floodplain development and for ensuring that all permits required by Federal or State/Commonwealth law have been received. State/Commonwealth, county, and community officials, based on their knowledge of local conditions and in the interest of safety, may set higher standards for construction in the Special Flood Hazard Area (SFHA), the area subject to inundation by the base flood). If the State/Commonwealth, county, or community has adopted more restrictive or comprehensive floodplain management criteria, these criteria take precedence over the minimum NFIP criteria.

This comment is based on the flood data presently available. If you have any questions about this document, please contact the FEMA Mapping and Insurance eXchange (FMIX) toll free at 1-877-336-2627 (1-877-FEMA MAP) or by letter addressed to the LOMC Clearinghouse, 3601 Eisenhower Avenue, Suite 500, Alexandria, VA 22304-6426. Additional Information about the NFIP is available on the FEMA website at https://www.fema.gov/flood-insurance.

Patrick "Rick" F. Sacbibit, P.E., Branch Chief
Engineering Services Branch
Federal Insurance and Mitigation Administration

Federal Emergency Management Agency
Washington, D.C. 20472

CONDITIONAL LETTER OF MAP REVISION COMMENT DOCUMENT (CONTINUED)

OTHER COMMUNITIES AFFECTED BY THIS CONDITIONAL REQUEST

CID Number: 480582 **Name:** Unincorporated Areas of Tarrant County, Texas

AFFECTED MAP PANELS

TYPE: FIRM* NO.: 48439C0340K DATE: 9/25/2009

TYPE: FIRM* NO.: 48439C0320L DATE: 3/21/2019

This comment is based on the flood data presently available. If you have any questions about this document, please contact the FEMA Mapping and Insurance eXchange (FMIX) toll free at 1-877-336-2627 (1-877-FEMA MAP) or by letter addressed to the LOMC Clearinghouse, 3601 Eisenhower Avenue, Suite 500, Alexandria, VA 22304-6426. Additional Information about the NFIP is available on the FEMA website at https://www.fema.gov/flood-insurance.

Patrick "Rick" F. Sacbibit, P.E., Branch Chief
Engineering Services Branch
Federal Insurance and Mitigation Administration

Federal Emergency Management Agency

Washington, D.C. 20472

CONDITIONAL LETTER OF MAP REVISION
COMMENT DOCUMENT (CONTINUED)

COMMUNITY INFORMATION

To determine the changes in flood hazards that will be caused by the proposed project, we compared the hydraulic modeling reflecting the proposed project (referred to as the proposed conditions model) to the hydraulic modeling used to prepare the Flood Insurance Study (FIS) (referred to as the effective model). If the effective model does not provide enough detail to evaluate the effects of the proposed project, an existing conditions model must be developed to provide this detail. This existing conditions model is then compared to the effective model and the proposed conditions model to differentiate the increases or decreases in flood hazards caused by more detailed modeling from the increases or decreases in flood hazards that will be caused by the proposed project.

The table below shows the changes in the BFEs:

BFE Comparison Table

Flooding Source: Village Creek		BFE Change (feet)	Location of maximum change
Existing vs. Effective	Maximum increase	3.1	Approximately 4,500 feet upstream of County Road 2051
	Maximum decrease	0.5	Approximately 2,260 feet upstream of County Road 2051
Proposed vs. Existing	Maximum increase	None	N/A
	Maximum decrease	2.9	Approximately 4,000 feet upstream of County Road 2051
Proposed vs. Effective	Maximum increase	1.8	Approximately 5,000 feet upstream of County Road 2051
	Maximum decrease	0.5	Approximately 2,260 feet upstream of County Road 2051

NFIP regulations Subparagraph 60.3(b)(7) requires communities to ensure that the flood-carrying capacity within the altered or relocated portion of any watercourse is maintained. This provision is incorporated into your community's existing floodplain management ordinances; therefore, responsibility for maintenance of the altered or relocated watercourse, including any related appurtenances such as bridges, culverts, and other drainage structures, rests with your community. We may request that your community submit a description and schedule of maintenance activities necessary to ensure this requirement.

This comment is based on the flood data presently available. If you have any questions about this document, please contact the FEMA Mapping and Insurance eXchange (FMIX) toll free at 1-877-336-2627 (1-877-FEMA MAP) or by letter addressed to the LOMC Clearinghouse, 3601 Eisenhower Avenue, Suite 500, Alexandria, VA 22304-6426. Additional Information about the NFIP is available on the FEMA website at https://www.fema.gov/flood-insurance.

Patrick "Rick" F. Sacbibit, P.E., Branch Chief
Engineering Services Branch
Federal Insurance and Mitigation Administration

Federal Emergency Management Agency

Washington, D.C. 20472

CONDITIONAL LETTER OF MAP REVISION COMMENT DOCUMENT (CONTINUED)

COMMUNITY INFORMATION (CONTINUED)

DATA REQUIRED FOR FOLLOW-UP LOMR

Upon completion of the project, your community must submit the data listed below and request that we make a final determination on revising the effective FIRM and FIS report. If the project is built as proposed and the data below are received, a revision to the FIRM and FIS report would be warranted.

• Detailed application and certification forms must be used for requesting final revisions to the maps. Therefore, when the map revision request for the area covered by this letter is submitted, Form 1, entitled "Overview and Concurrence Form," must be included. A copy of this form may be accessed at https://www.fema.gov/flood-maps/change-your-flood-zone/paper-application-forms/mt-2.

• The detailed application and certification forms listed below may be required if as-built conditions differ from the proposed plans. If required, please submit new forms, which may be accessed at https://www.fema.gov/flood-maps/change-your-flood-zone/paper-application-forms/mt-2, or annotated copies of the previously submitted forms showing the revised information.

Form 2, entitled "Riverine Hydrology and Hydraulics Form." Hydraulic analyses for as-built conditions of the base flood, the 10-percent, 2-percent, and 0.2-percent-annual-chance floods, and the regulatory floodway, must be submitted with Form 2.

• A certified topographic work map showing the revised and effective base and 0.2-percent-annual-chance floodplain and floodway boundaries. Please ensure that the revised information ties in with the current effective information at the downstream and upstream ends of the revised reach.

• An annotated copy of the FIRM, at the scale of the effective FIRM, that shows the revised base and 0.2-percent-annual-chance floodplain and floodway boundary delineations shown on the submitted work map and how they tie-in to the base and 0.2-percent-annual-chance floodplain and floodway boundary delineations shown on the current effective FIRM at the downstream and upstream ends of the revised reach.

• As-built plans, certified by a registered Professional Engineer, of all proposed project elements.

• A copy of the public notice distributed by your community stating its intent to revise the regulatory floodway, or a signed statement by your community that it has notified all affected property owners and affected adjacent jurisdiction.

• Documentation of the individual legal notices sent to property owners who will be affected by any widening or shifting of the base floodplain and/or any BFE increases along Village Creek.

This comment is based on the flood data presently available. If you have any questions about this document, please contact the FEMA Mapping and Insurance eXchange (FMIX) toll free at 1-877-336-2627 (1-877-FEMA MAP) or by letter addressed to the LOMC Clearinghouse, 3601 Eisenhower Avenue, Suite 500, Alexandria, VA 22304-6426. Additional Information about the NFIP is available on the FEMA website at https://www.fema.gov/flood-insurance.

Patrick "Rick" F. Sacbibit, P.E., Branch Chief
Engineering Services Branch
Federal Insurance and Mitigation Administration

Federal Emergency Management Agency

Washington, D.C. 20472

CONDITIONAL LETTER OF MAP REVISION
COMMENT DOCUMENT (CONTINUED)

COMMUNITY INFORMATION (CONTINUED)

DATA REQUIRED FOR FOLLOW-UP LOMR (continued)

• FEMA's fee schedule for reviewing and processing requests for conditional and final modifications to published flood information and maps may be accessed at https://www.fema.gov/flood-maps/change-your-flood-zone/status/flood-map-related-fees. The fee at the time of the map revision submittal must be received before we can begin processing the request. Payment of this fee can be made through a check or money order, made payable in U.S. funds to the National Flood Insurance Program, or by credit card (Visa or MasterCard only). Please either forward the payment, along with the revision application, to the following address:

LOMC Clearinghouse
Attention: LOMR Manager
3601 Eisenhower Avenue, Suite 500
Alexandria, VA 22304-6426

or submit the LOMR using the Online LOMC portal at: https://hazards.fema.gov/femaportal/onlinelomc/signin

After receiving appropriate documentation to show that the project has been completed, FEMA will initiate a revision to the FIRM and FIS report. Because the flood hazard information (i.e., base flood elevations, base flood depths, SFHAs, zone designations, and/or regulatory floodways) will change as a result of the project, a 90-day appeal period will be initiated for the revision, during which community officials and interested persons may appeal the revised flood hazard information based on scientific or technical data.

This comment is based on the flood data presently available. If you have any questions about this document, please contact the FEMA Mapping and Insurance eXchange (FMIX) toll free at 1-877-336-2627 (1-877-FEMA MAP) or by letter addressed to the LOMC Clearinghouse, 3601 Eisenhower Avenue, Suite 500, Alexandria, VA 22304-6426. Additional Information about the NFIP is available on the FEMA website at https://www.fema.gov/flood-insurance.

Patrick "Rick" F. Sacbibit, P.E., Branch Chief
Engineering Services Branch
Federal Insurance and Mitigation Administration

Federal Emergency Management Agency

Washington, D.C. 20472

CONDITIONAL LETTER OF MAP REVISION
COMMENT DOCUMENT (CONTINUED)

COMMUNITY INFORMATION (CONTINUED)

COMMUNITY REMINDERS

We have designated a Consultation Coordination Officer (CCO) to assist your community. The CCO will be the primary liaison between your community and FEMA. For information regarding your CCO, please contact:

Sandy Keefe
Director, Mitigation Division
Federal Emergency Management Agency, Region VI
Federal Regional Center, Room 202
800 North Loop 288
Denton, TX 76209
(940) 898-5127

A preliminary study is being conducted for the City of Fort Worth. Preliminary copies of the revised FIRM and FIS report were submitted to your community for review on November 13, 2020, and may become effective before the revision request following this CLOMR is submitted. Please ensure that the data submitted for the revision ties into the data effective at the time of the submittal.

This comment is based on the flood data presently available. If you have any questions about this document, please contact the FEMA Mapping and Insurance eXchange (FMIX) toll free at 1-877-336-2627 (1-877-FEMA MAP) or by letter addressed to the LOMC Clearinghouse, 3601 Eisenhower Avenue, Suite 500, Alexandria, VA 22304-6426. Additional Information about the NFIP is available on the FEMA website at https://www.fema.gov/flood-insurance.

Patrick "Rick" F. Sacbibit, P.E., Branch Chief
Engineering Services Branch
Federal Insurance and Mitigation Administration

Federal Emergency Management Agency
Washington, D.C. 20472

September 23, 2022

CERTIFIED MAIL
RETURN RECEIPT REQUESTED

The Honorable B. Glen Whitley
Tarrant County Judge
100 East Weatherford Street
Fort Worth, TX 76196

IN REPLY REFER TO:

Case No.:	22-06-1554R
Community Name:	Tarrant County, TX
Community No.:	480582

104

Dear Judge Whitley:

We are providing our comments with the enclosed Conditional Letter of Map Revision (CLOMR) on a proposed project within your community that, if constructed as proposed, could revise the effective Flood Insurance Study report and Flood Insurance Rate Map (FIRM) for your community.

If you have any questions regarding the floodplain management regulations for your community, the National Flood Insurance Program (NFIP) in general, or technical questions regarding this CLOMR, please contact the Director, Mitigation Division of the Federal Emergency Management Agency (FEMA) Regional Office in Denton, Texas, at (940) 898-5127, or the FEMA Mapping and Insurance eXchange (FMIX) toll free at 1-877-336-2627 (1-877-FEMA MAP). Additional information about the NFIP is available on our website at https://www.fema.gov/flood-insurance.

Sincerely,

Patrick "Rick" F. Sacbibit, P.E., Branch Chief
Engineering Services Branch
Federal Insurance and Mitigation Administration

Enclosure:
Conditional Letter of Map Revision Comment Document

cc: The Honorable Mattie Parker
Mayor, City of Fort Worth

Clair C. Davis, P.E., CFM
Floodplain Administrator
City of Fort Worth

Joseph Jackson, P.E., CFM
Floodplain Administrator
Tarrant County

Charles R. Marsh, P.E.
Project Director
Weaver Consultants Group, LLC

 | Issue Date: September 23, 2022 | Case No.: 22-06-1554R | CLOMR-APP

Federal Emergency Management Agency

Washington, D.C. 20472

CONDITIONAL LETTER OF MAP REVISION COMMENT DOCUMENT

COMMUNITY INFORMATION		PROPOSED PROJECT DESCRIPTION	BASIS OF CONDITIONAL REQUEST
COMMUNITY	Tarrant County Texas (Unincorporated Areas) COMMUNITY NO.: 480582	FILL EXCAVATION	1D HYDRAULIC ANALYSIS FLOODWAY UPDATED TOPOGRAPHIC DATA
IDENTIFIER	Fort Worth C&D Landfill	APPROXIMATE LATITUDE AND LONGITUDE: 32.633, -97.243 SOURCE: OTHER DATUM: NAD 83	

AFFECTED MAP PANELS

TYPE	NO.	DATE
FIRM*	48439C0340K	9/25/2009
FIRM*	48439C0320L	3/21/2019

* FIRM - Flood Insurance Rate Map

FLOODING SOURCE AND REACH DESCRIPTION

Village Creek – From the downstream side of County Road 2051 to approximately 1,550 feet downstream of Shelby Road

PROPOSED PROJECT DESCRIPTION

Flooding Source	Proposed Project	Location of Proposed Project
Village Creek	Fill Placement	From approximately 2,260 feet upstream of County Road 2051 to approximately 5,930 feet upstream of County Road 2051
	Excavation	From approximately 2,260 feet upstream of County Road 2051 to approximately 5,930 feet upstream of County Road 2051

SUMMARY OF IMPACTS TO FLOOD HAZARD DATA

Flooding Source	Effective Flooding	Proposed Flooding	Increases	Decreases
Village Creek	Floodway	Floodway	Yes	Yes
	Zone AE	Zone AE	Yes	Yes
	Zone X (shaded)	Zone X (shaded)	Yes	Yes
	BFEs*	BFEs	Yes	Yes

* BFEs - Base (1-percent-annual-chance) Flood Elevations

COMMENT

This document provides the Federal Emergency Management Agency's (FEMA's) comment regarding a request for a CLOMR for the project described above. This document is not a final determination; it only provides our comment on the proposed project in relation to the flood hazard information shown on the effective National Flood Insurance Program (NFIP) map. We reviewed the submitted data and the data used to prepare the effective flood hazard information for your community and determined that the proposed project meets the minimum floodplain management criteria of the NFIP. Your community is responsible for approving all floodplain development and for ensuring that all permits required by Federal or State/Commonwealth law have been received. State/Commonwealth, county, and community officials, based on their knowledge of local conditions and in the interest of safety, may set higher standards for construction in the Special Flood Hazard Area (SFHA), the area subject to inundation by the base flood). If the State/Commonwealth, county, or community has adopted more restrictive or comprehensive floodplain management criteria, these criteria take precedence over the minimum NFIP criteria.

This comment is based on the flood data presently available. If you have any questions about this document, please contact the FEMA Mapping and Insurance eXchange (FMIX) toll free at 1-877-336-2627 (1-877-FEMA MAP) or by letter addressed to the LOMC Clearinghouse, 3601 Eisenhower Avenue, Suite 500, Alexandria, VA 22304-6426. Additional Information about the NFIP is available on the FEMA website at https://www.fema.gov/flood-insurance.

Patrick "Rick" F. Sacbibit, P.E., Branch Chief
Engineering Services Branch
Federal Insurance and Mitigation Administration

Federal Emergency Management Agency

Washington, D.C. 20472

CONDITIONAL LETTER OF MAP REVISION
COMMENT DOCUMENT (CONTINUED)

OTHER COMMUNITIES AFFECTED BY THIS CONDITIONAL REQUEST

CID Number: 480596 **Name:** City of Fort Worth, Texas

AFFECTED MAP PANELS

TYPE: FIRM*	NO.: 48439C0340K	DATE: 9/25/2009
TYPE: FIRM*	NO.: 48439C0320L	DATE: 3/21/2019

This comment is based on the flood data presently available. If you have any questions about this document, please contact the FEMA Mapping and Insurance eXchange (FMIX) toll free at 1-877-336-2627 (1-877-FEMA MAP) or by letter addressed to the LOMC Clearinghouse, 3601 Eisenhower Avenue, Suite 500, Alexandria, VA 22304-6426. Additional Information about the NFIP is available on the FEMA website at https://www.fema.gov/flood-insurance.

Patrick "Rick" F. Sacbibit, P.E., Branch Chief
Engineering Services Branch
Federal Insurance and Mitigation Administration

Federal Emergency Management Agency

Washington, D.C. 20472

CONDITIONAL LETTER OF MAP REVISION COMMENT DOCUMENT (CONTINUED)

COMMUNITY INFORMATION

To determine the changes in flood hazards that will be caused by the proposed project, we compared the hydraulic modeling reflecting the proposed project (referred to as the proposed conditions model) to the hydraulic modeling used to prepare the Flood Insurance Study (FIS) (referred to as the effective model). If the effective model does not provide enough detail to evaluate the effects of the proposed project, an existing conditions model must be developed to provide this detail. This existing conditions model is then compared to the effective model and the proposed conditions model to differentiate the increases or decreases in flood hazards caused by more detailed modeling from the increases or decreases in flood hazards that will be caused by the proposed project.

The table below shows the changes in the BFEs:

BFE Comparison Table

Flooding Source: Village Creek		BFE Change (feet)	Location of maximum change
Existing vs. Effective	Maximum increase	3.1	Approximately 4,500 feet upstream of County Road 2051
	Maximum decrease	0.5	Approximately 2,760 feet upstream of County Road 2051
Proposed vs. Existing	Maximum increase	None	N/A
	Maximum decrease	2.9	Approximately 4,000 feet upstream of County Road 2051
Proposed vs. Effective	Maximum increase	1.8	Approximately 5,000 feet upstream of County Road 2051
	Maximum decrease	1.1	Approximately 3,220 feet upstream of County Road 2051

NFIP regulations Subparagraph 60.3(b)(7) requires communities to ensure that the flood-carrying capacity within the altered or relocated portion of any watercourse is maintained. This provision is incorporated into your community's existing floodplain management ordinances; therefore, responsibility for maintenance of the altered or relocated watercourse, including any related appurtenances such as bridges, culverts, and other drainage structures, rests with your community. We may request that your community submit a description and schedule of maintenance activities necessary to ensure this requirement.

This comment is based on the flood data presently available. If you have any questions about this document, please contact the FEMA Mapping and Insurance eXchange (FMIX) toll free at 1-877-336-2627 (1-877-FEMA MAP) or by letter addressed to the LOMC Clearinghouse, 3601 Eisenhower Avenue, Suite 500, Alexandria, VA 22304-6426. Additional Information about the NFIP is available on the FEMA website at https://www.fema.gov/flood-insurance.

Patrick "Rick" F. Sacbibit, P.E., Branch Chief
Engineering Services Branch
Federal Insurance and Mitigation Administration

Federal Emergency Management Agency

Washington, D.C. 20472

CONDITIONAL LETTER OF MAP REVISION COMMENT DOCUMENT (CONTINUED)

COMMUNITY INFORMATION (CONTINUED)

DATA REQUIRED FOR FOLLOW-UP LOMR

Upon completion of the project, your community must submit the data listed below and request that we make a final determination on revising the effective FIRM and FIS report. If the project is built as proposed and the data below are received, a revision to the FIRM and FIS report would be warranted.

• Detailed application and certification forms must be used for requesting final revisions to the maps. Therefore, when the map revision request for the area covered by this letter is submitted, Form 1, entitled "Overview and Concurrence Form," must be included. A copy of this form may be accessed at https://www.fema.gov/flood-maps/change-your-flood-zone/paper-application-forms/mt-2.

• The detailed application and certification forms listed below may be required if as-built conditions differ from the proposed plans. If required, please submit new forms, which may be accessed at https://www.fema.gov/flood-maps/change-your-flood-zone/paper-application-forms/mt-2, or annotated copies of the previously submitted forms showing the revised information.

Form 2, entitled "Riverine Hydrology and Hydraulics Form." Hydraulic analyses for as-built conditions of the base flood, the 10-percent, 2-percent, and 0.2-percent-annual-chance floods, and the regulatory floodway, must be submitted with Form 2.

• A certified topographic work map showing the revised and effective base and 0.2-percent-annual-chance floodplain and floodway boundaries. Please ensure that the revised information ties in with the current effective information at the downstream and upstream ends of the revised reach.

• An annotated copy of the FIRM, at the scale of the effective FIRM, that shows the revised base and 0.2-percent-annual-chance floodplain and floodway boundary delineations shown on the submitted work map and how they tie-in to the base and 0.2-percent-annual-chance floodplain and floodway boundary delineations shown on the current effective FIRM at the downstream and upstream ends of the revised reach.

• As-built plans, certified by a registered Professional Engineer, of all proposed project elements.
• A copy of the public notice distributed by your community stating its intent to revise the regulatory floodway, or a signed statement by your community that it has notified all affected property owners and affected adjacent jurisdiction.

• Documentation of the individual legal notices sent to property owners who will be affected by any widening or shifting of the base floodplain and/or any BFE increases along Village Creek.

This comment is based on the flood data presently available. If you have any questions about this document, please contact the FEMA Mapping and Insurance eXchange (FMIX) toll free at 1-877-336-2627 (1-877-FEMA MAP) or by letter addressed to the LOMC Clearinghouse, 3601 Eisenhower Avenue, Suite 500, Alexandria, VA 22304-6426. Additional Information about the NFIP is available on the FEMA website at https://www.fema.gov/flood-insurance.

Patrick "Rick" F. Sacbibit, P.E., Branch Chief
Engineering Services Branch
Federal Insurance and Mitigation Administration

Federal Emergency Management Agency

Washington, D.C. 20472

CONDITIONAL LETTER OF MAP REVISION COMMENT DOCUMENT (CONTINUED)

COMMUNITY INFORMATION (CONTINUED)

COMMUNITY REMINDERS

We have designated a Consultation Coordination Officer (CCO) to assist your community. The CCO will be the primary liaison between your community and FEMA. For information regarding your CCO, please contact:

Sandy Keefe
Director, Mitigation Division
Federal Emergency Management Agency, Region VI
Federal Regional Center, Room 202
800 North Loop 288
Denton, TX 76209
(940) 898-5127

A preliminary study is being conducted for Tarrant County. Preliminary copies of the revised FIRM and FIS report were submitted to your community for review on November 13, 2020, and may become effective before the revision request following this CLOMR is submitted. Please ensure that the data submitted for the revision ties into the data effective at the time of the submittal

This comment is based on the flood data presently available. If you have any questions about this document, please contact the FEMA Mapping and Insurance eXchange (FMIX) toll free at 1-877-336-2627 (1-877-FEMA MAP) or by letter addressed to the LOMC Clearinghouse, 3601 Eisenhower Avenue, Suite 500, Alexandria, VA 22304-6426. Additional Information about the NFIP is available on the FEMA website at https://www.fema.gov/flood-insurance.

Patrick "Rick" F. Sacbibit, P.E., Branch Chief
Engineering Services Branch
Federal Insurance and Mitigation Administration

Weaver Consultants Group

June 24, 2022
Project No. 0771-356-11-35

Mr. Benjamin Kaiser, P.E., CFM
LOMC Clearinghouse
3601 Eisenhower Avenue, Suite 500
Alexandria, VA 22304-6426

Re: Conditional Letter of Map Revision (CLOMR) – Comment Response
Case No. 22-06-1554R
Fort Worth C&D Landfill
Tarrant County, Texas

Dear Mr. Kaiser:

The purpose of this letter, submitted on behalf of Texas Regional Landfill Company, LP, is to respond to the Federal Emergency Management Agency (FEMA) letter dated May 13, 2022. This response letter contains each comment item identified in the FEMA comment letter (in bold) and a detailed response to each item in the same order listed within the FEMA comment letter.

To facilitate your review the Revised Complete CLOMR is included as Attachment 1. The pages include all the text and drawings that have been updated to address the comments below.

1. **From our review of the submitted annotated Flood Insurance Rate Map (FIRM), it appears that City of Fort Worth is affected by this CLOMR. Please submit a copy of MT-2 Application/Certification Form 1, entitled "Overview and Concurrence Form," where the second signature block has been signed by a City of Fort Worth official (preferably the Floodplain Administer). Alternatively, please provide documentation that the corporate limits shown on the FIRM map are not accurate and City of Fort Worth is not actually affected by this revision. Acceptable documentation includes a current corporate limits map provided by the community along with an annexation agreement, if applicable.**

Response:

A copy of a Overview and Concurrence MT-2 Form 1 signed by the City of Fort Worth has been incorporated into Appendix B of the CLOMR. A copy has also been uploaded to the FEMA portal.

2. **Our review of the submitted hydraulic analyses revealed the following issues. Please update the models to resolve these issues and resubmit the executable**

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files for review. If any changes to the model are made other than those septically requested below, please describe those changes made.

a. **Our review revealed that the duplicate effective hydraulic analysis for Village Creek does not yield results that match the effective Flood Insurance Rate Map and Flood Insurance Study (FIS) report. Please revise the submitted duplicate effective model so that it is an exact reproduction of the effective model and matches the information in the effective FIRM and FIS report within 0.5 feet. Please provide paper and digital copies of the input and output files for this model.**

Response:

This comment has been disregarded per the May 20, 2022 email.

b. **Typically, a contraction coefficient of 0.1 and an expansion coefficient of 0.3 should be used at cross sections that are not at structures. Please revise the submitted hydraulic model so that the contraction and expansion loss coefficients are equal to 0.1 and 0.3, respectively, at Cross Section 84520.**

Response:

The hydraulic model has been revised to include contraction coefficients of 0.1 and expansion coefficients of 0.3 at cross sections within the project area that are not at structures, including cross section 84520.

c. **Our review of the submitted floodway model revealed that the left and/or right encroachment stations are located outside the 1-percent-annual-chance (base) floodplain at Cross Sections 79070, 79190, and 80190. Please revise the proposed and existing conditions models so that the encroachment stations are located at the limit of or within the base floodplain at all cross sections.**

Response:

All encroachment stations within the project area have been reviewed to ensure they are not located outside the 1-percent-annual-chance (base) floodplain. Right encroachment stations at cross sections 79070 and 79190 and the left encroachment station at cross section 80190 have been updated to coincide with the base floodplain limit.

d. **Our review of the submitted proposed conditions hydraulic model revealed that Cross Section 81190 and 81690 are not completely modeled within the 0.2-percent-annual-chance (500-year) floodplain. The use of vertically extended cross sections might both overestimate the water surface elevation**

(WESL) and underestimate the width of the proposed 0.2-percent-annual-chance floodplain. Please revise Cross Sections 81190 and 81690 so that they cross the entire width of the 0.2-percent-annual-chance floodplain.

Response:

Cross sections 81190 and 81690 have been extended so that they completely model the entire width of the 0.2-percent-annual-chance floodplain. The hydraulic model and topographic work map have been updated to reflect this.

e. **Our review revealed that there are interpolated cross sections in the HEC-RAS hydraulic model along the left banks at Cross Sections 81690, 81190, 80690, 80190, 79690, 79190, 78790, 78290, 77790, 77290, and 76790. Please revise the model to include only field run cross sections and/or cross sections cut from topographic data, or explain why interpolated cross sections are appropriate at these locations. Please note that interpolated cross sections are not appropriate at locations where the bounding cross sections have greatly varied geometry.**

Response:

This comment has been disregarded per the May 26, 2020 email.

3. **Please provide a topographic work map for the entire requested area of revision that reflects all applicable items listed on page 2, Section C, of Application/Certification Form 2, entitled "Riverine Hydrology & Hydraulics Form," including those items listed below. Please show this information on a map of suitable scale and topographic definition to provide reasonable accuracy. All items should be labeled for easy cross-referencing to the submitted existing conditions hydraulic model. Please ensure that the topographic maps reference the vertical datum such as the National Geodetic Vertical Datum of 1929 (NGVD29) or the North America Vertical Datum of 1988 (NAVD88).**

   a. **Please add the following cross section included in the proposed conditions hydraulic model to the work map. Cross Section 84520.**

   Response:

   Cross Section 84520 has been added to the topographic work map.

   b. **Please ensure the following cross sections have visible labels that are not overrun by lines: Cross Sections 75825, 76790, 77790, 78290, 80690, 81690, and 82090.**

Response:

Cross section labels have been updated to be more visible and not overrun by lines.

c. **Please revise the work map to only show the boundaries of the effective and proposed conditions base floodplain, 0.2-percent-annual-chance floodplain, and regulatory. Please remove information related to the corrected effective conditions from the work map. For clarity, please show the effective and proposed delineations in different line types and color. Please show smooth graphical tie-in between the proposed and effective floodway at the downstream end of the revised reach. Please ensure that the proposed delineations tie-in directly to the effective delineations and that the tie-ins occur a short distance downstream of the downstream most revised cross section.**

Response:

The corrected effective condition has been removed from the topographic work map. The effective and proposed delineations are shown in different line types and colors. A key is included in the topographic work map for clarity. Tie in call outs are located at the upstream and downstream ends of the floodplains.

d. **Our review revealed that the revised floodway has abrupt expansions and contractions and do not reflect the natural path of the regulatory floodway between Cross Sections 82090 to 81190, 80690 to 79690, and 76790 to 75705. Please revise the submitted proposed conditions HEC-RAS hydraulic model and topographic work map so that the expansion and contraction of the floodway is more gradual. In addition, please revise the floodway hydraulic model to reflect this floodway revision, and submit the executable files of the model.**

Response:

The floodway limits have been revised to more closely match the natural path of Village Creek. The hydraulic work map has been updated to reflect these revisions and is included in the FEMA portal upload. Please note that these abrupt expansions are consistent with the HEC-RAS encroachment model.

e. **Please reference the vertical datum.**

Response:

The vertical datum is NAVD88. A datum certification is included in Appendix B of the CLOMR and is included in the key for the topographic work map.

4. **The topwidth in the proposed conditions hydraulic analysis at Cross Section 75825, 2,971 feet, does not match the approximate topwidth shown on the topographic work map, entitled "CLOMR REQUEST TOPOGRAPHIC WORK MAP," prepared by Weaver Consultants Group, dated February 2022, which is 3,103 feet. Please submit revised hydraulic analyses or revised work maps as appropriate.**

   Response:

   As part of this comment response all topwidths have been checked against the hydraulic model. The top width at Cross Section 75825 has been revised to match the topwidth in the hydraulic model.

5. **Our review revealed that the reach intersects Cross Section 75825 more than once and causes discrepancies between the reach lengths shown on the topographic work map and the reach lengths used in the submitted hydraulic HEC-RAS models at the same cross section. Please resolve these discrepancies and submit revised topographic work maps or revised models as appropriate so the cross section does not intersect the streamline more than once. Please ensure that the reach lengths between cross sections shown on the work map match the reach length given in the submitted hydraulic models.**

   Response:

   Cross Section 75825 has been revised in the corrected effective and post-project hydraulic models to only intersect Village Creek once. Reach lengths have also been revised to reflect the updated cross section.

6. **The submitted annotated FIRM does not include all the effective FIRM panels affected by your revision request. Our review revealed multiple panels of the FIRM for Tarrant County and Incorporated Areas and FIRM panel 48439C0320L dated March 21, 2019, is also affected. Please submit annotated copies of all affected FIRM panels, at the scales of the effective FIRM, that clearly show the revised boundary, delineations of the base floodplain and 0.2-percent-annual-chance floodplain shown on the above-referenced topographic work map and how they tie-in to the boundary delineation shown on the effective FIRM at the downstream and upstream ends of the revised reach.**

   Response:

   Drawing A.2 – Flood Insurance Rate Map (FIRM) and A.10 – Revised Flood Insurance Rate Map (FIRM) have been revised to include all FIRM panels affected by the revision request.

7. Our review indicates that the proposed project encroaches upon a regulatory floodway and will cause increases in base flood elevations (BFEs). Please provide evidence that the proposed project satisfies the requirements of Section 65.12 of the National Flood Insurance Program (NFIP) regulations, including the items stated below. A copy of Part 65 of the regulations can be accessed at https://www.ecfr.gov/cgi-bin/text-idx?c=ecfr&tpl=/ecfrbrowse/Title44/44cfr65_ main_02.tpl.

   a. **Evaluation of alternatives which would not result in any increase in BFEs and an explanation why these alternatives are not feasible.**

   b. **Documentation that individual legal notices have been sent to all property owners affected by the increases in BFEs due to the proposed project. Documentation of legal notice may take the form of a signed copy of the letter sent and either a mailing list or certified mailing receipts. The attached Combined CLOMR Notice template may be used to prepare the legal notice. Prior to distribution, please submit a draft copy of the notice for verification of content.**

   c. **Certification by a registered Professional Engineer (P.E.) that no structures are located in areas which would be impacted by the increased BFEs due to the project.**

Response:

Per email correspondence on May 20, 2022, this comment has been disregarded.

During your review, if you need additional information or have any questions, please call.

Sincerely,
Weaver Consultants Group, LLC

Charles R. Marsh, P.E.
Project Director

Attachments: Attachment 1 – Revised Complete CLOMR Application

Weaver Consultants Group

February 11, 2022
Project No: 0771-356-11-31

Joseph Jackson
County Engineer
100 E. Weatherford
Fort Worth, TX 76196

Re: Conditional Letter of Map Revision Request
Fort Worth C&D Landfill
Tarrant County, Texas

Dear Mr. Jackson:

On behalf of Texas Regional Landfill Company, LP, please find enclosed a Conditional Letter of Map Revision (CLOMR) request for the referenced facility. The purpose of this study is to request a CLOMR from Tarrant County and the Federal Emergency Management Agency (FEMA) for proposed revisions to Flood Hazard Zones within a 184-acre track of land in Tarrant County, Texas. This property is owned by Texas Regional Landfill Company, LP. All proposed revisions are within the property and associated with the existing Fort Worth C&D Landfill which is operated within its boundaries. The proposed revisions to the landfill design are necessary to increase the landfill capacity as part of ongoing efforts to address long term waste disposal needs of the communities in Tarrant and surrounding counties.

Upon completion of your review, please sign the Overview and Concurrence Forms included in Appendix B. Please send a copy of the signed form back to Weaver Consultants Group, LLC for our files. Weaver Consultants Group will then submit the CLOMR document, including hydraulic models, electronically to FEMA. The processing and review fees will be submitted to the FEMA Revisions Fee Collection System Administrator after we receive your overview and concurrence forms.

If you have any questions or require further information, please call.

Sincerely,
Weaver Consultants Group, LLC

Charles R. Marsh, P.E.
Project Director

Attachments: Attachment 1 – Conditional Letter of Map Revision Request
Attachment 2 – Hydraulic Models (Thumb Drive)

cc: Gary Bartels, Texas Regional Landfill Company, LP

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ATTACHMENT 1

CONDITIONAL LETTER OF MAP REVISION REQUEST

CONDITIONAL LETTER OF MAP REVISION REQUEST

FORT WORTH C&D LANDFILL

TARRANT COUNTY, TEXAS

Prepared for

Texas Regional Landfill Company, LP

February 2022

Revised June 2022

STATE OF TEXAS
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105073
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6.24.22

Prepared by

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WCG Project No. 0771-356-11-31
IIIF-G-A-22

CONTENTS

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CONTENTS (CONTINUED)

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1 INTRODUCTION

1.1 Purpose

The purpose of this study is to request a Conditional Letter of Map Revision (CLOMR) from Tarrant County and the Federal Emergency Management Agency (FEMA) for proposed revisions to Flood Hazard Zones within a 184-acre area in Tarrant County, Texas. This property is owned by Texas Regional Landfill Company, LP, and the Fort Worth C&D Landfill is operated within its boundaries. Figure 1.1 – Site Location Map shows the location of the site and the 184 acre property boundary.

This CLOMR request has been developed to obtain approval to revise the Flood Insurance Rate Map (FIRM) to facilitate an expansion of the landfill. The scope of this study is limited to portions of Village Creek near the landfill. This waterway is shown on the aerial photo included in Figure 1.2. This CLOMR will allow for the continued development of the existing landfill operation, as demonstrated on Figure 1.3. Figures 1.4 and 1.5 demonstrate comparisons of the current and proposed conditions of Village Creek. Figures 1.6, 1.7, and 1.8 include the duplicate effective, corrected effective, and post-project site conditions, respectively. A summary of the proposed map revisions is provided below.

- **Modification of the floodplain in the western portion of the current landfill.** This area is proposed to be removed from the 100-year floodplain to provide for the continued development of the solid waste disposal area. The modified floodplain is shown on Figure 1.5.
- **Detention pond development along the western portion of the site.** A detention pond is proposed to be constructed along the eastern side of the proposed expansion area. The detention pond will collect runoff from the landfill and convey it into Village Creek. The detention pond is shown on Figure 1.5.
- **Southern area development to allow for the development of a borrow area facilities pad to support borrowing and stockpiling of soil.** The pad will be constructed on a separate property owned by Texas Regional Landfill Company, LP that is used as the landfill's borrow area. The pad is shown on Figure 1.5.

The results of this study will be used to revise floodplain and floodway boundaries and to provide FEMA with the required technical data to issue a CLOMR for the proposed project.

1.2 Project Background

The Fort Worth C&D Landfill is an existing Type IV municipal solid waste (MSW) landfill operating under Texas Commission on Environmental Quality (TCEQ) Permit No. MSW-1983D. Per TCEQ MSW rules, a Type IV landfill is a disposal facility that accept construction and demolition (C&D) and other authorized waste as included in Title 30 §TAC 330. The existing landfill currently provides C&D waste disposal services for residences and businesses in Tarrant, Johnson, Parker, Collin, Dallas, and Denton Counties and surrounding areas. With this expansion, the existing landfill refuse footprint will increase from approximately 100.3 to 124.3 acres. The Fort Worth C&D Landfill is owned and operated by Texas Regional Landfill Company, LP. The site is located approximately 15 miles southeast of downtown Fort Worth, adjacent to the City of Kennedale. As shown on Figure 1.1, the facility is located approximately 2.4 miles south of IH-20 and 5 miles east of IH-35W. Figure 1.2 shows the existing landfill development, as well as the proposed landfill expansion area, on a recent aerial photograph.

1.3 Proposed Site Development

As demonstrated on Figure 1.3, relative to the currently permitted landfill configuration, the landfill is proposed to develop on the west side of the existing disposal area. Figures 1.6, 1.7, and 1.8 show the duplicate effective, corrected effective, and post-project floodplain delineations for the different site conditions of the Village Creek model discussed in this CLOMR. The specifics of each condition are discussed in Sections 1.5.1, 1.5.2, and 1.5.3, respectively. The post-project condition includes proposed revisions to the floodplain and floodway, as discussed in Section 1.5.3 and shown in detail on Figure 1.8.

1.4 Scope

The scope of this study is limited to Village Creek near the project site as shown on Figure 1.3. The proposed landfill development included in this CLOMR request is detailed on Figure 1.8 and the drawings presented in Appendix A.

The following conditions are included in this CLOMR request and shown on Figures 1.6 through 1.8.

- Duplicate Effective Condition – The duplicate effective hydraulic model was provided by FEMA in April 2021. Floodplain and floodway limits were reproduced from effective firm panel 48439C0340K.
- Corrected Effective Condition – The corrected effective condition was modeled by adding eleven (11) additional cross sections to the duplicate effective condition. These cross sections are shown on Figure 1.7. This condition supplements the duplicate effective condition by modeling the existing site condition including the existing landfill development.
- Post-Project Condition – The post-project condition represents the proposed condition of the landfill development after the landfill expansion and related site improvements have been made, as shown on Figure 1.8. Refer to Drawing A.10 for tie-in locations between proposed and effective floodplain limits.

1.5 Scenarios Investigated

The analysis for all scenarios investigated in this CLOMR are discussed in detail below. The HEC-RAS output files and hydraulic models for each condition are provided electronically.

1.5.1 Duplicate Effective Condition

The existing condition of Village Creek (Figure 1.6 and Drawing A.3) contains Zone AE flood hazard areas as shown on the effective FIRM panel for the area. As noted in Section 1.4, the Village Creek duplicate effective model for this CLOMR was provided by FEMA. The 100-year and 500-year floodplains and the floodway were reproduced from the effective FIRM panel and are shown on Drawing A.4.

1.5.2 Corrected Effective Condition

The corrected effective condition was developed by adding eleven (11) additional cross sections to the Village Creek duplicate effective condition hydraulic model (cross sections 76790 through 78790 and 79190 through 81690). These cross sections, shown on Figure 1.7, were added to model the proposed development of the landfill. The cross section geometries were interpolated from the nearest upstream effective cross section using an average slope and the distance between the cross sections on the west side of Village Creek. The geometry for the areas east of Village Creek were obtained from 2021 aerial photography from Firmatek. The floodplains and floodway for the corrected effective condition are shown on Drawing A.5 in Appendix A.

The inclusion of additional cross sections resulted in minor changes in the corrected effective floodplain elevations relative to the duplicate effective condition. The water surfaces at cross sections 85830 through 79070 increased for the 100-year storm. Cross sections 88660 through 79070 had an increase in water surface elevations for the 500-year storm when compared to the duplicate effective model, as shown in Table 2-3.

1.5.3 Post-Project Condition

The post-project condition hydraulic model for Village Creek incorporates the proposed landfill development, shown on Figure 1.8 and Drawing A.6 (Post-Project Condition Map). The proposed landfill development includes the proposed landfill expansion, detention pond, and borrow area facilities pad.

Cross sections 77290 through 79190 were updated to include the required geometric revisions to reflect the construction of the proposed landfill expansion and detention pond. Cross sections 80190 and 80690 were updated to reflect the proposed borrow area facilities pad. The addition of the detention pond affects the 500-year floodplain in the area due to the additional storage created in these sections. The 500-year floodplain is not significantly impacted within the limits of the project as a result of this change.

1.6 Concepts and Methods

The hydraulic methods employed in this study are employed in accordance with the requirements of FEMA and they are consistent with the Tarrant County and the TCEQ requirements. Peak flow rates were determined from the effective Flood Insurance Study (FIS) for the project area. The USACE HEC-RAS (Ref. 2) computer program, Version 5.0.7, was used to determine water surface profiles, floodplain limits, and floodways. The floodplains and floodways presented in this study represent duplicate effective, corrected effective, and post-project conditions (after completion of proposed development). Analyses of the peak flow rates, floodplains, floodplain limits, and floodways for these conditions proceeded in the following sequence:

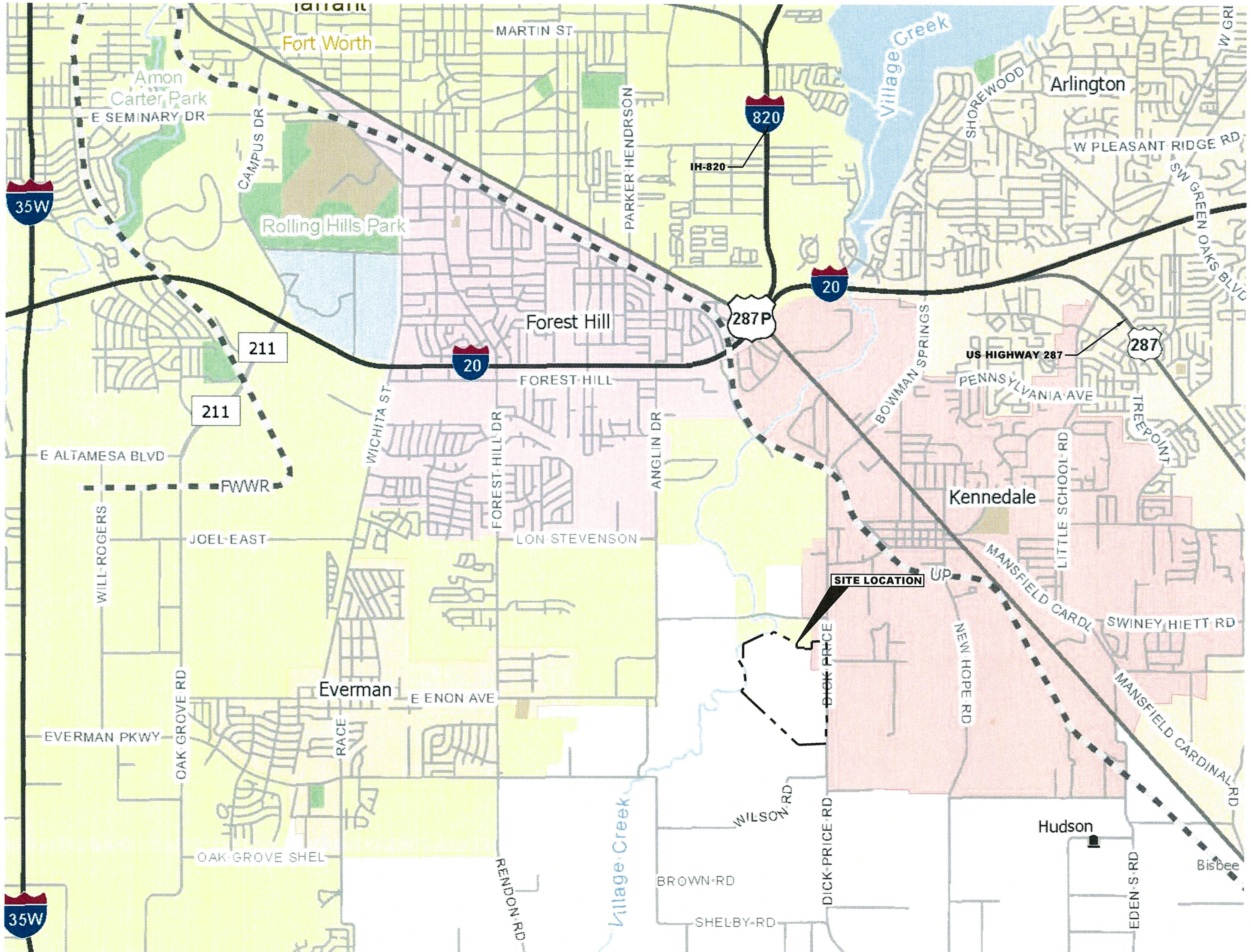
(1) Peak flow rates were assigned using FIS peak flows for the study area.

(2) Hydraulic models were developed to evaluate flood elevations for the streams under peak flow conditions using HEC-RAS.

(3) Hydraulic models were developed to delineate the limits of encroachment where the base flood elevation increased by no more than 1 foot.

(4) The floodplains and floodways were delineated using the results of the hydraulic modeling.

Peak flow rates, water surface elevations, and floodway boundaries are based on 10-, 50-, 100-, and 500-year storm return frequency.



LEGEND

- PERMIT BOUNDARY
- Unincorporated Community
- County Seat
- Border Crossing
- Cemetery
- Cemetery (Inside City)
- Deep Draft Port
- Shallow Draft Port
- Railroad
- Dam
- River or Stream
- TXDOT District
- Lakes
- Education
- Military
- Airport Runway
- Airport
- Prison
- Parks and Other Public Land
- CITY OF FORT WORTH CITY LIMITS

NOTES:

1. ADAPTED FROM TEXAS DEPARTMENT OF TRANSPORTATION HIGHWAY MAY, 2018.

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☐ DRAFT ☒ FOR PERMITTING PURPOSES ONLY ☐ ISSUED FOR CONSTRUCTION	PREPARED FOR TEXAS REGIONAL LANDFILL COMPANY, LP	CLOMR REQUEST SITE LOCATION MAP
DATE: 02/2022 FILE: 0771-356-11 CAD: FIG 1.1-SITE LOCATION MAP.DWG DRAWN BY: RAA DESIGN BY: JBP REVIEWED BY: CRM	REVISIONS NO. / DATE / DESCRIPTION	FORT WORTH C&D LANDFILL TARRANT COUNTY, TEXAS
Weaver Consultants Group TBPE REGISTRATION NO. F-3727		WWW.WCGRP.COM / FIGURE 1.1

O:\0771\356\CLOMR (2021)\FIG 1.1-SITE LOCATION MAP.dwg, rarrington, 1:2

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N

0 200 400
SCALE IN FEET

LEGEND

- PERMIT BOUNDARY
- PERMITTED LIMIT OF WASTE
- PROPOSED LIMIT OF WASTE

NOTES:

1. AERIAL IMAGERY PROVIDED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN 01-08-2021.

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O:\0771\356\CLOMR (2021)\FIG 1.2-AERIAL PHOTOGRAPH.dwg, rarrington, 1:2

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DESIGN BY: JBP
REVIEWED BY: CRM

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CLOMR REQUEST
AERIAL PHOTOGRAPH

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TARRANT COUNTY, TEXAS

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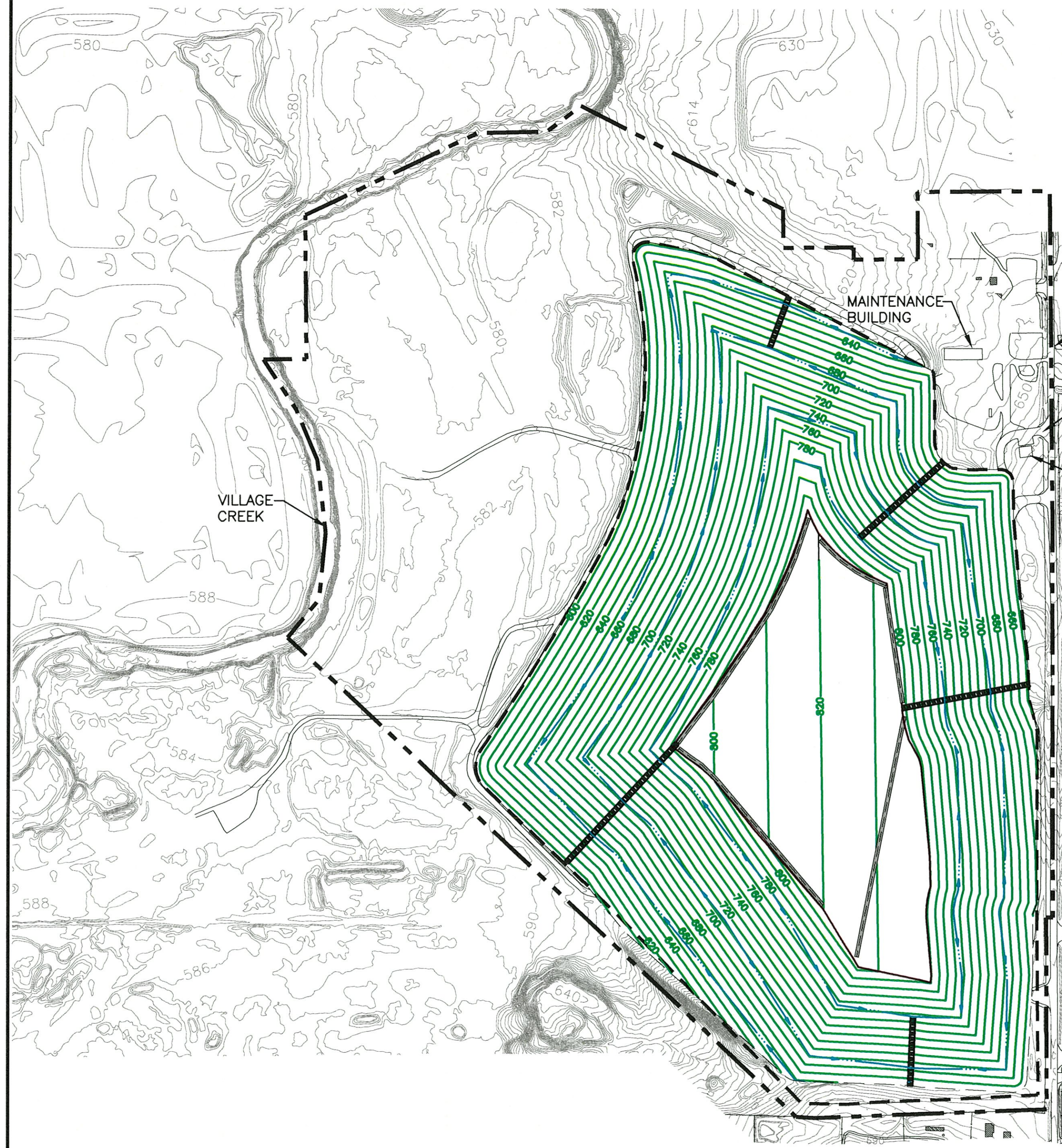
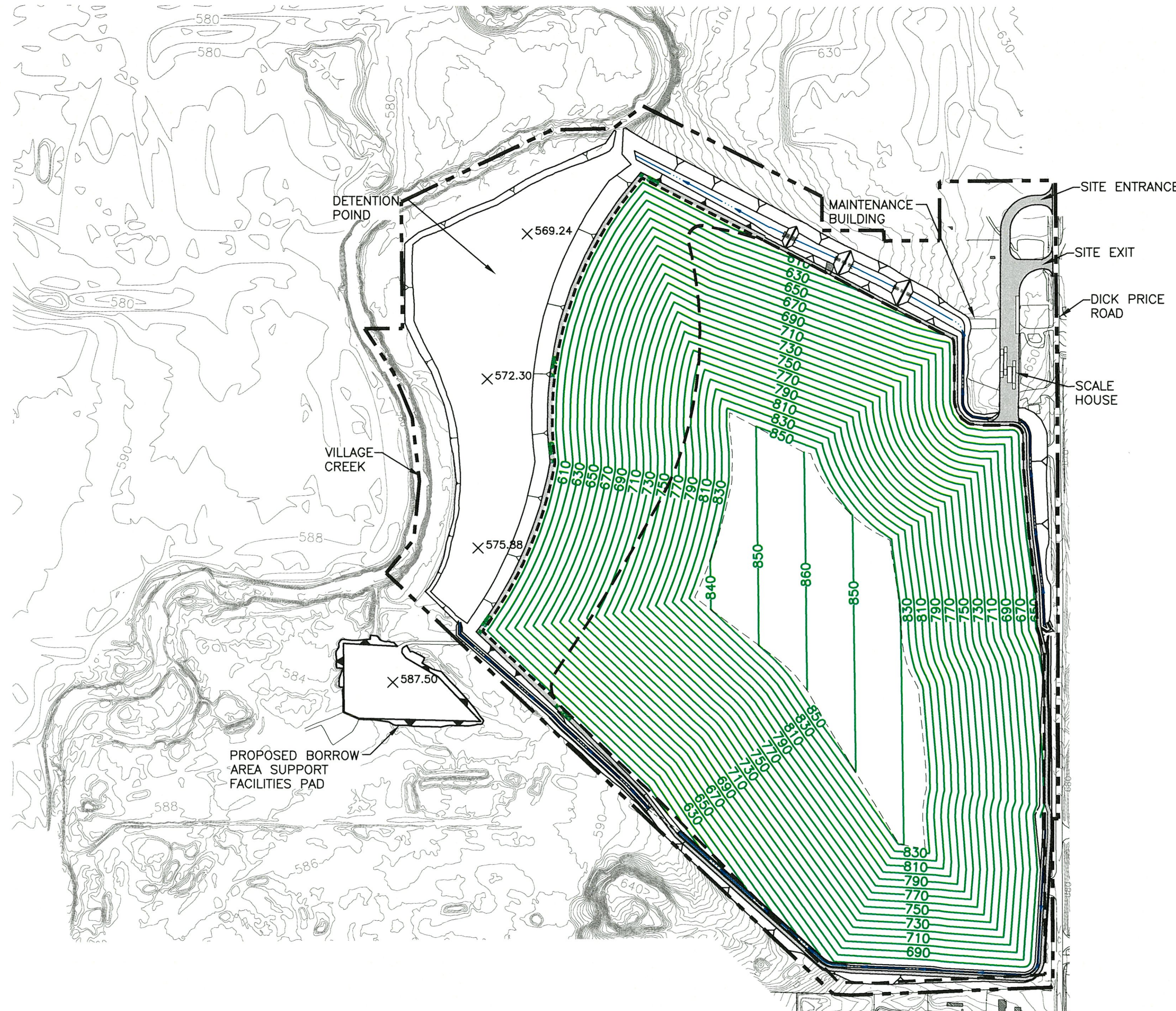
FIGURE 1.2



PERMITTED LANDFILL COMPLETION PLAN



PROPOSED LANDFILL COMPLETION PLAN

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LEGEND

- PERMIT BOUNDARY
- PERMITTED LIMIT OF WASTE
- PROPOSED LIMIT OF WASTE
- 580 COMPOSITE TOPOGRAPHY (SEE NOTE 1)
- 700 PROPOSED FINAL COVER CONTOUR

N

0 300 600
SCALE IN FEET

NOTES:

1. COMPOSITE TOPOGRAPHY PROVIDED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN 01-08-2021 AND BY NORTH CENTRAL TEXAS COUNCIL OF GOVERNMENTS TOPOGRAPHY TAKEN 2017.

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DATE: 02/2022 FILE: 0771-356-11 CAD: FIG 1.3-EXPANSION COMP. SUMMARY.DWG	DRAWN BY: RAA DESIGN BY: JBP REVIEWED BY: CRM	REVISIONS NO. | DATE | DESCRIPTION	FORT WORTH C&D LANDFILL TARRANT COUNTY, TEXAS
Weaver Consultants Group			WWW.WCGRP.COM | FIGURE 1.3

O:\0771\356\CLOMR (2021)\FIG 1.3-EXPANSION COMPARISON SUMMARY.dwg, rarrington, 1:2

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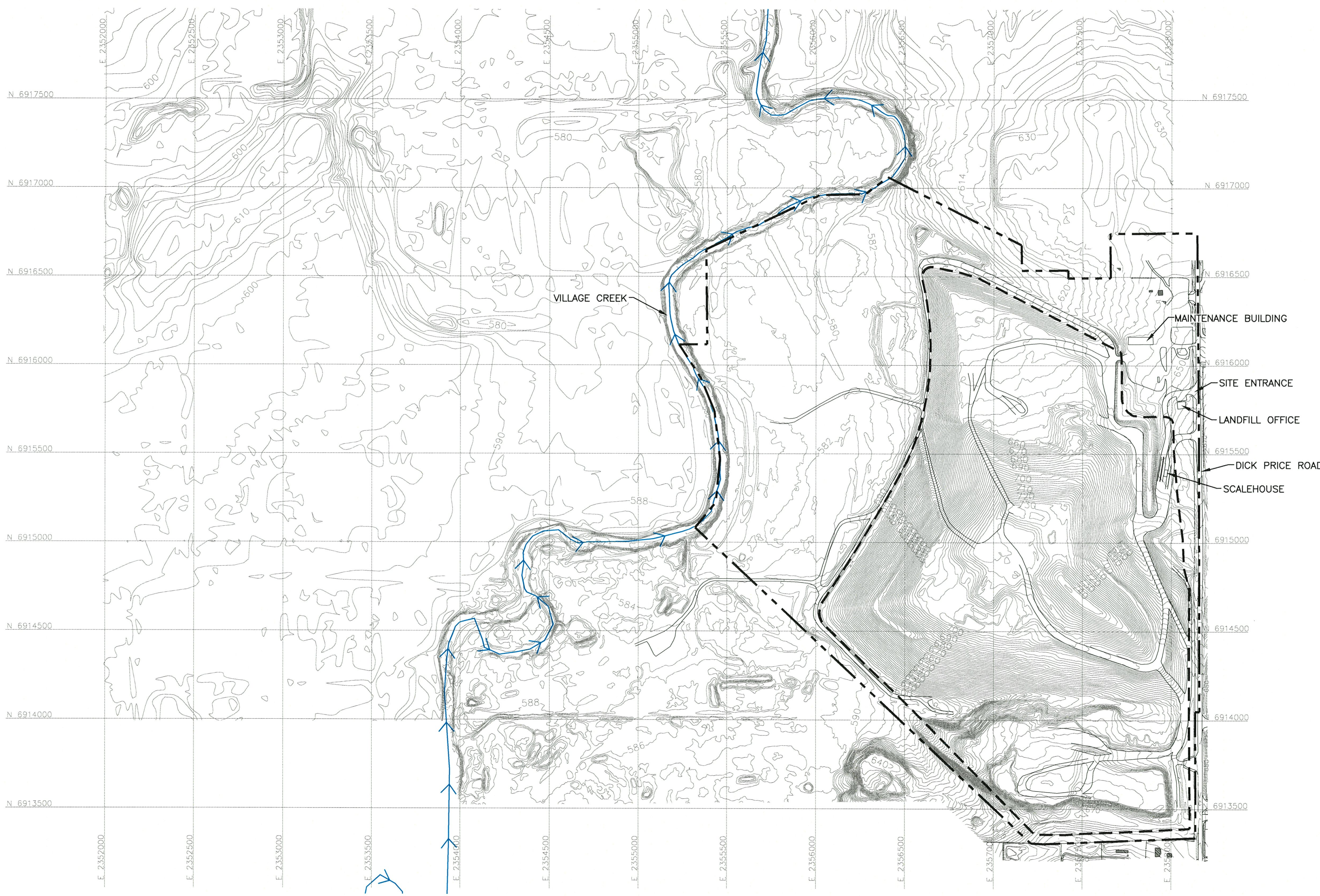

0 300 600
SCALE IN FEET

LEGEND

- PERMIT BOUNDARY
- PERMITTED LIMIT OF WASTE
- 580 COMPOSITE TOPOGRAPHY (SEE NOTE 1)
- N 6915000 STATE PLANE COORDINATE
- HYDRAULIC MODELING FLOW LINE

NOTES:

1. COMPOSITE TOPOGRAPHY PROVIDED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN 01-08-2021 AND BY NORTH CENTRAL TEXAS COUNCIL OF GOVERNMENTS TOPOGRAPHY TAKEN 2017.

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DESIGN BY: JBP
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EXISTING SITE PLAN

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FIGURE 1.4

O:\0771\356\CLOMR (2021)\FIG 1.4-EXISTING SITE PLAN.dwg, rarrington, 1:2

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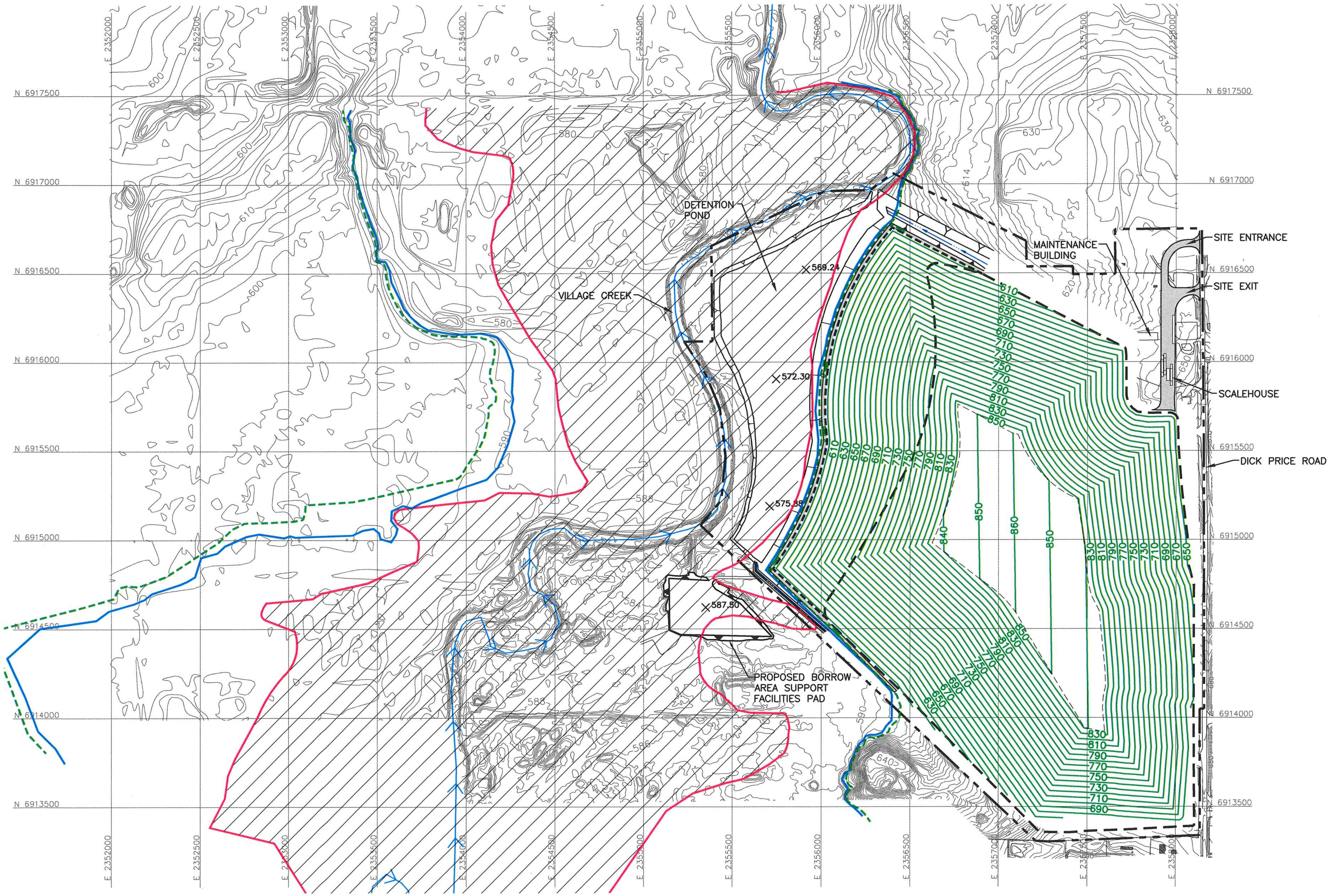

0 300 600
SCALE IN FEET

LEGEND

PERMIT BOUNDARY
PERMITTED LIMIT OF WASTE
PROPOSED LIMIT OF WASTE
580 COMPOSITE TOPOGRAPHY (SEE NOTE 1)
N 6915000 STATE PLANE COORDINATE
HYDRAULIC MODELING FLOW LINE
POST–PROJECT 100–YEAR FLOODPLAIN
POST–PROJECT 500–YEAR FLOODPLAIN
POST–PROJECT FLOODWAY
750 PROPOSED FINAL COVER CONTOUR

NOTES:

1. COMPOSITE TOPOGRAPHY PROVIDED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN 01–08–2021 AND BY NORTH CENTRAL TEXAS COUNCIL OF GOVERNMENTS TOPOGRAPHY TAKEN 2017.

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CAD: FIG 1.5–POST–PROJECT SITE PLAN.DWG
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CLOMR REQUEST
POST–PROJECT SITE PLAN

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FIGURE 1.5

O:\0771\356\CLOMR (2021)\FIG 1.5–POST–PROJECT SITE PLAN.dwg, erouton, 1:2

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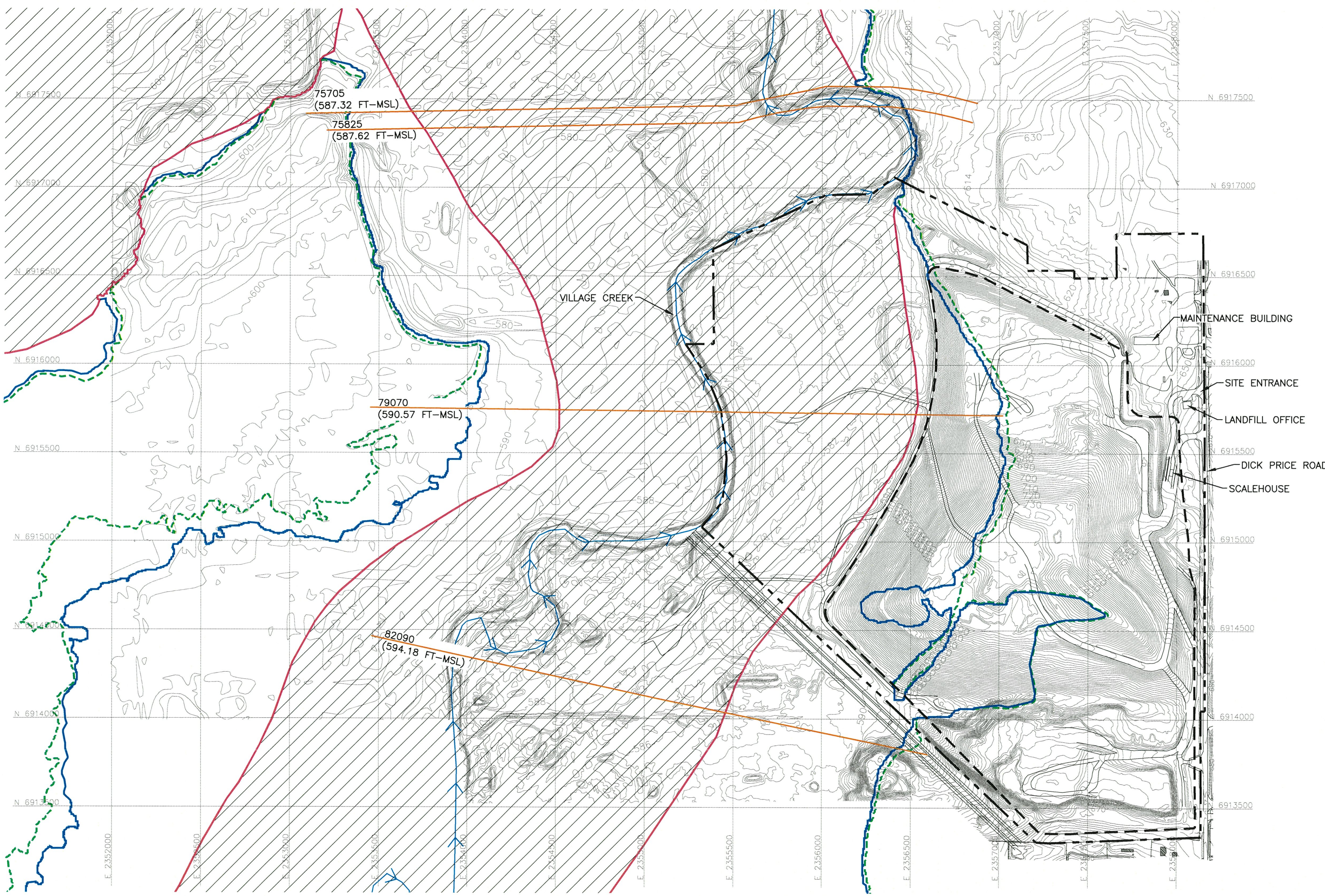

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0 300 600

SCALE IN FEET

LEGEND

PERMIT BOUNDARY

PERMITTED LIMIT OF WASTE

580 COMPOSITE TOPOGRAPHY (SEE NOTE 1)

N 6915000 STATE PLANE COORDINATE

HYDRAULIC MODELING FLOW LINE

DUPLICATE EFFECTIVE 100-YEAR FLOODPLAIN (SEE NOTE 2)

DUPLICATE EFFECTIVE 500-YEAR FLOODPLAIN (SEE NOTE 2)

DUPLICATE EFFECTIVE FLOODWAY (SEE NOTE 2)

79070 (590.57 FT-MSL) HEC-RAS CROSS SECTION AND 100-YEAR FLOODPLAIN ELEVATION

NOTES:

1. COMPOSITE TOPOGRAPHY PROVIDED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN 01-08-2021 AND BY NORTH CENTRAL TEXAS COUNCIL OF GOVERNMENTS TOPOGRAPHY TAKEN 2017.
2. FLOODPLAIN AND FLOODWAY INFORMATION REPRODUCED FROM FEMA FIRM PANEL 48439C0340K.

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O:\0771\356\CLOMR (2021)\FIG 1.6-DUPLICATE EFFECTIVE CONDITION MAP.dwg, rarrington, 1:2

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CAD: FIG 1.6-DUPLICATE EFFECTIVE MAP.DWG

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DUPLICATE EFFECTIVE CONDITION MAP

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FIGURE 1.6

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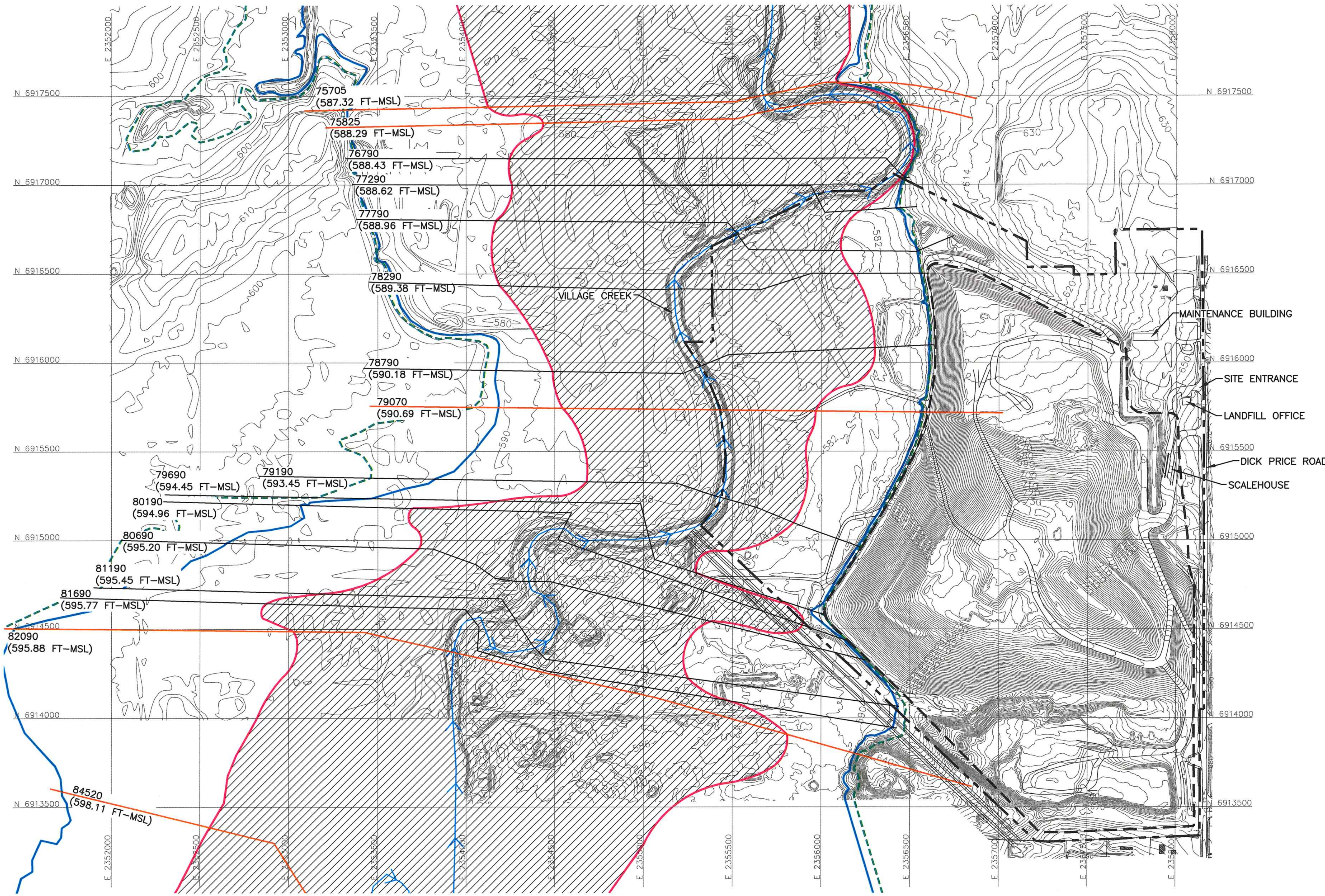

N

0 300 600
SCALE IN FEET

LEGEND

PERMIT BOUNDARY
PERMITTED LIMIT OF WASTE
580 COMPOSITE TOPOGRAPHY (SEE NOTE 1)
N 6915000 STATE PLANE COORDINATE
HYDRAULIC MODELING FLOW LINE
CORRECTED EFFECTIVE 100-YEAR FLOODPLAIN
CORRECTED EFFECTIVE 500-YEAR FLOODPLAIN
CORRECTED EFFECTIVE FLOODWAY
79070 (590.69 FT-MSL) HEC-RAS CROSS SECTION AND BASE FLOOD ELEVATION
79190 (593.45 FT-MSL) CORRECTED EFFECTIVE HEC-RAS CROSS SECTION AND BASE FLOOD ELEVATION

NOTES:

1. COMPOSITE TOPOGRAPHY PROVIDED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN 01-08-2021 AND BY NORTH CENTRAL TEXAS COUNCIL OF GOVERNMENTS TOPOGRAPHY TAKEN 2017.

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CAD: FIG 1.7-CORRECTED EFFECTIVE MAP.DWG

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1	06/2022	REVISED FLOODWAY AND FLOODPLAINS
2	06/2022	ADDED CROSS SECTION 84520

CLOMR REQUEST
CORRECTED EFFECTIVE CONDITION MAP

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FIGURE 1.7

O:\0771\356\CLOMR (2021)\FIG 1.7-CORRECTED EFFECTIVE CONDITION MAP.dwg, erouton, 1:2

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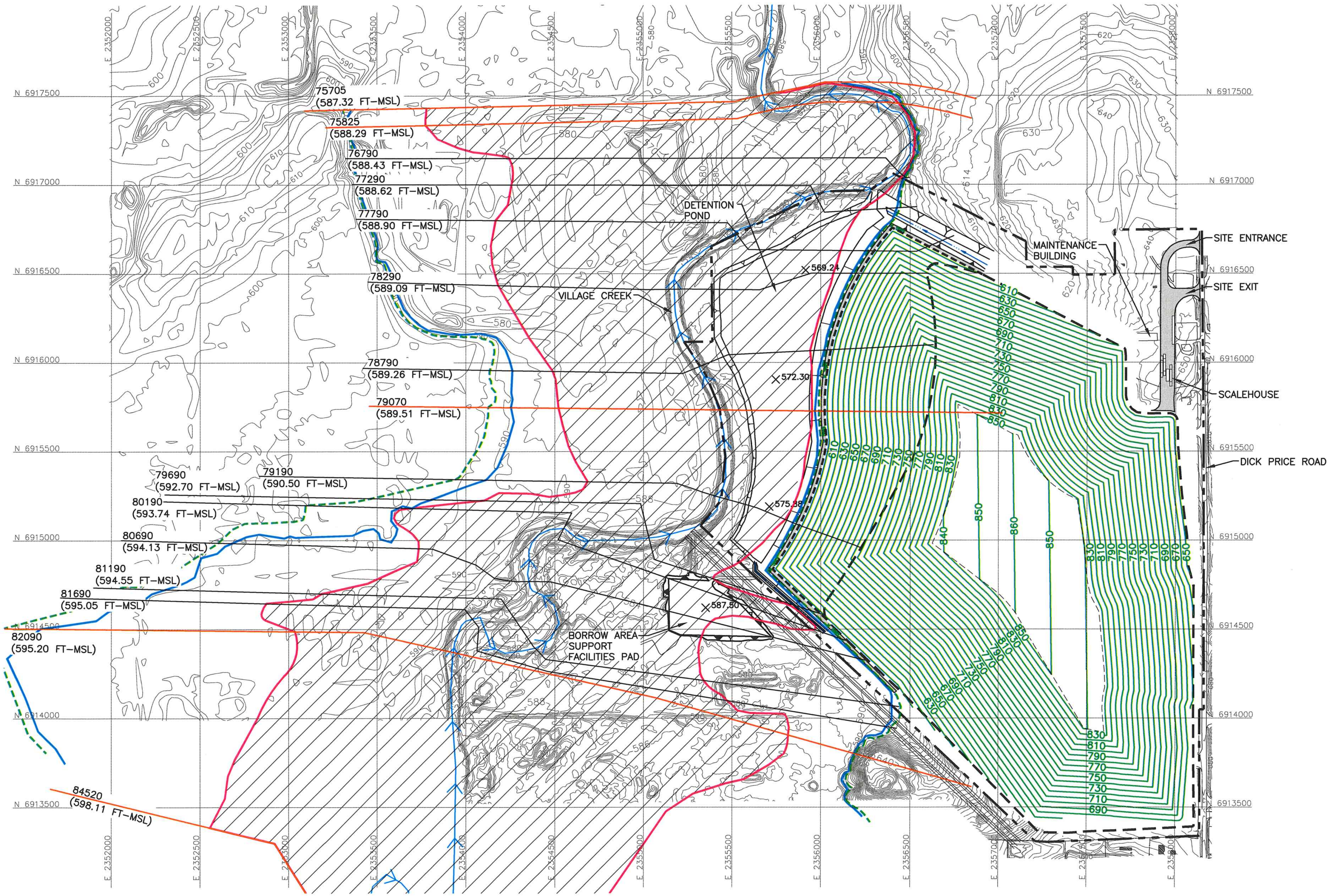

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0 300 600
SCALE IN FEET

LEGEND

— · · —	PERMIT BOUNDARY
— — —	PERMITTED LIMIT OF WASTE
- - - -	PROPOSED LIMIT OF WASTE
580	COMPOSITE TOPOGRAPHY (SEE NOTE 1)
N 6915000	STATE PLANE COORDINATE
→ →	HYDRAULIC MODELING FLOW LINE
——	POST-PROJECT 100-YEAR FLOODPLAIN
- - - -	POST-PROJECT 500-YEAR FLOODPLAIN
▨	POST-PROJECT FLOODWAY
750	PROPOSED FINAL COVER CONTOUR
79070 (590.69 FT-MSL)	HEC-RAS CROSS SECTION AND BASE FLOOD ELEVATION
79190 (593.45 FT-MSL)	POST-PROJECT HEC-RAS CROSS SECTION AND BASE FLOOD ELEVATION

NOTES:

1. COMPOSITE TOPOGRAPHY PROVIDED BY FIRMATEK FROM AERIAL PHOTOGRAPHY FLOWN 01-08-2021 AND BY NORTH CENTRAL TEXAS COUNCIL OF GOVERNMENTS TOPOGRAPHY TAKEN 2017.

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POST-PROJECT CONDITION MAP

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FIGURE 1.8

O:\0771\356\CLOMR (2021)\FIG 1.8-POST-PROJECT CONDITION MAP.dwg, erouton, 1:2

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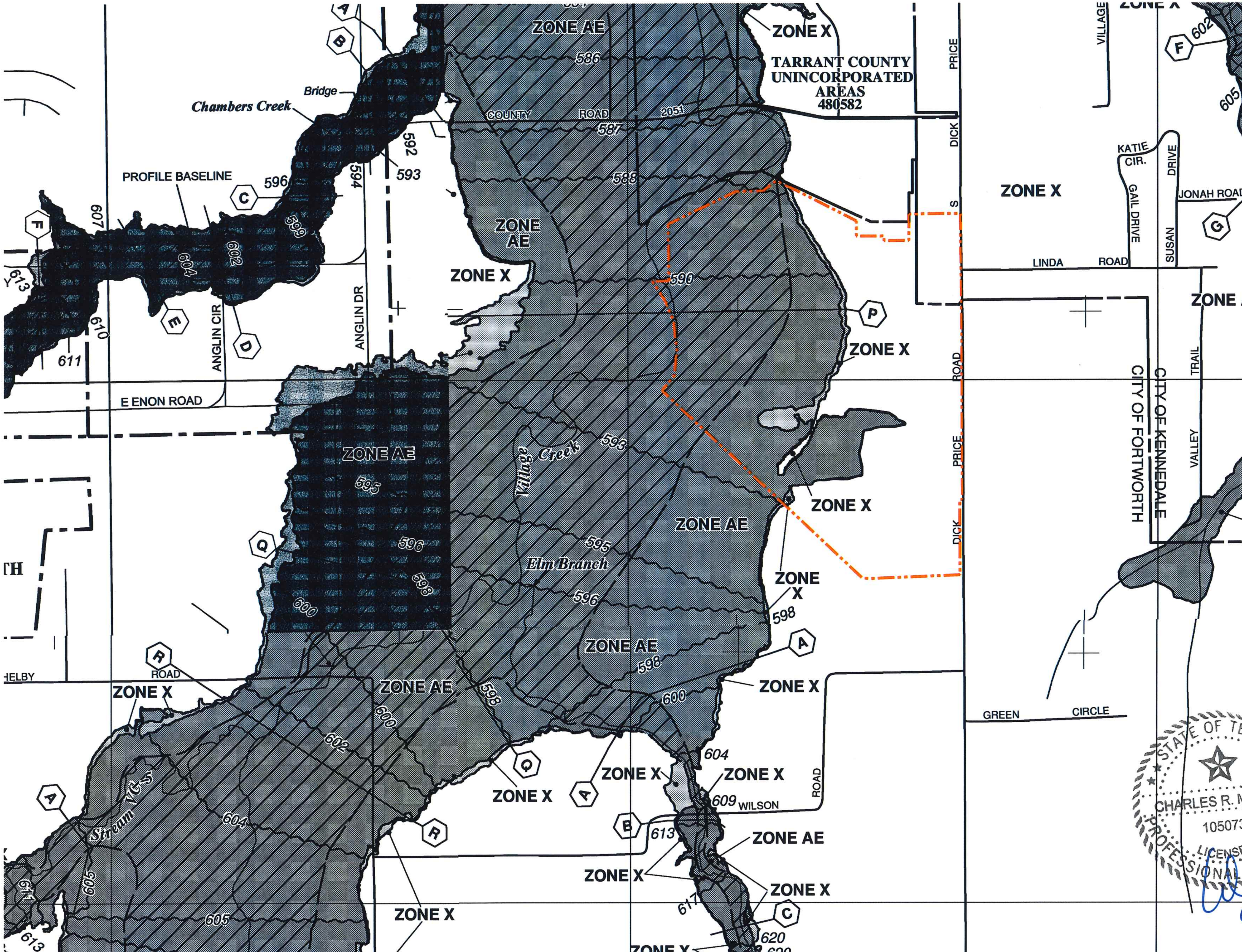

N

0 500 1000

SCALE IN FEET

LEGEND

PROPERTY BOUNDARY

ZONE A	No Base Flood Elevations determined.
ZONE AE	Base Flood Elevations determined.
ZONE AH	Flood depths of 1 to 3 feet (usually areas of ponding); Base Flood Elevations determined.
ZONE AO	Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain); average depths determined. For areas of alluvial fan flooding, velocities also determined.
ZONE AR	Special Flood Hazard Area formerly protected from the 1% annual chance flood by a flood control system that was subsequently decertified. Zone AR indicates that the former flood control system is being restored to provide protection from the 1% annual chance or greater flood.
ZONE A99	Area to be protected from 1% annual chance flood by a Federal flood protection system under construction; no Base Flood Elevations determined.
ZONE V	Coastal flood zone with velocity hazard (wave action); no Base Flood Elevations determined.
ZONE VE	Coastal flood zone with velocity hazard (wave action); Base Flood Elevations determined.

FLOODWAY AREAS IN ZONE AE

The floodway is the channel of a stream plus any adjacent floodplain areas that must be kept free of encroachment so that the 1% annual chance flood can be carried without substantial increases in flood heights.

OTHER FLOOD AREAS

ZONE X	Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood.

OTHER AREAS

ZONE X	Areas determined to be outside the 0.2% annual chance floodplain.
ZONE D	Areas in which flood hazards are undetermined, but possible.

COASTAL BARRIER RESOURCES SYSTEM (CBRS) AREAS

OTHERWISE PROTECTED AREAS (OPAs)

CBRS areas and OPAs are normally located within or adjacent to Special Flood Hazard Areas.

	Floodplain boundary
	Floodway boundary
	Zone D boundary
	CBRS and OPA boundary
	Boundary dividing Special Flood Hazard Areas of different Base Flood Elevations, flood depths or flood velocities.
513	Base Flood Elevation line and value; elevation in feet*
(EL 987)	Base Flood Elevation value where uniform within zone; elevation in feet*

* Referenced to the North American Vertical Datum of 1988 (NAVD 88)

(A)——(A)	Cross section line
(23)- - -(23)	Transect line
97°07'30", 32°22'30"	Geographic coordinates referenced to the North American Datum of 1983 (NAD 83)
$^{42}75^{000m}$N	1000-meter Universal Transverse Mercator grid ticks, zone 14
6000000 FT	5000-foot grid values: Texas State Plane coordinate system, north central zone (FIPSZONE 4202), Lambert Conformal Conic
DX5510 ×	Bench mark (see explanation in Notes to Users section of this FIRM panel)
• M1.5	River Mile

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NOTES:

1. REPRODUCED FROM FEMA FIRM PANELS 48439C0340K, 48439C0320L, 48439C0435K, 48439C0455K.

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DATE: 02/2022 FILE: 0771-356-11 CAD: A.2-FIRM MAP.DWG	DRAWN BY: RAA DESIGN BY: JBP REVIEWED BY: CRM	REVISIONS NO. 1 — DATE 06/2022 — DESCRIPTION: REVISED FLOODWAY AND FLOODPLAINS		
Weaver Consultants Group TBPE REGISTRATION NO. F-3727				WWW.WCGRP.COM — DRAWING A.2

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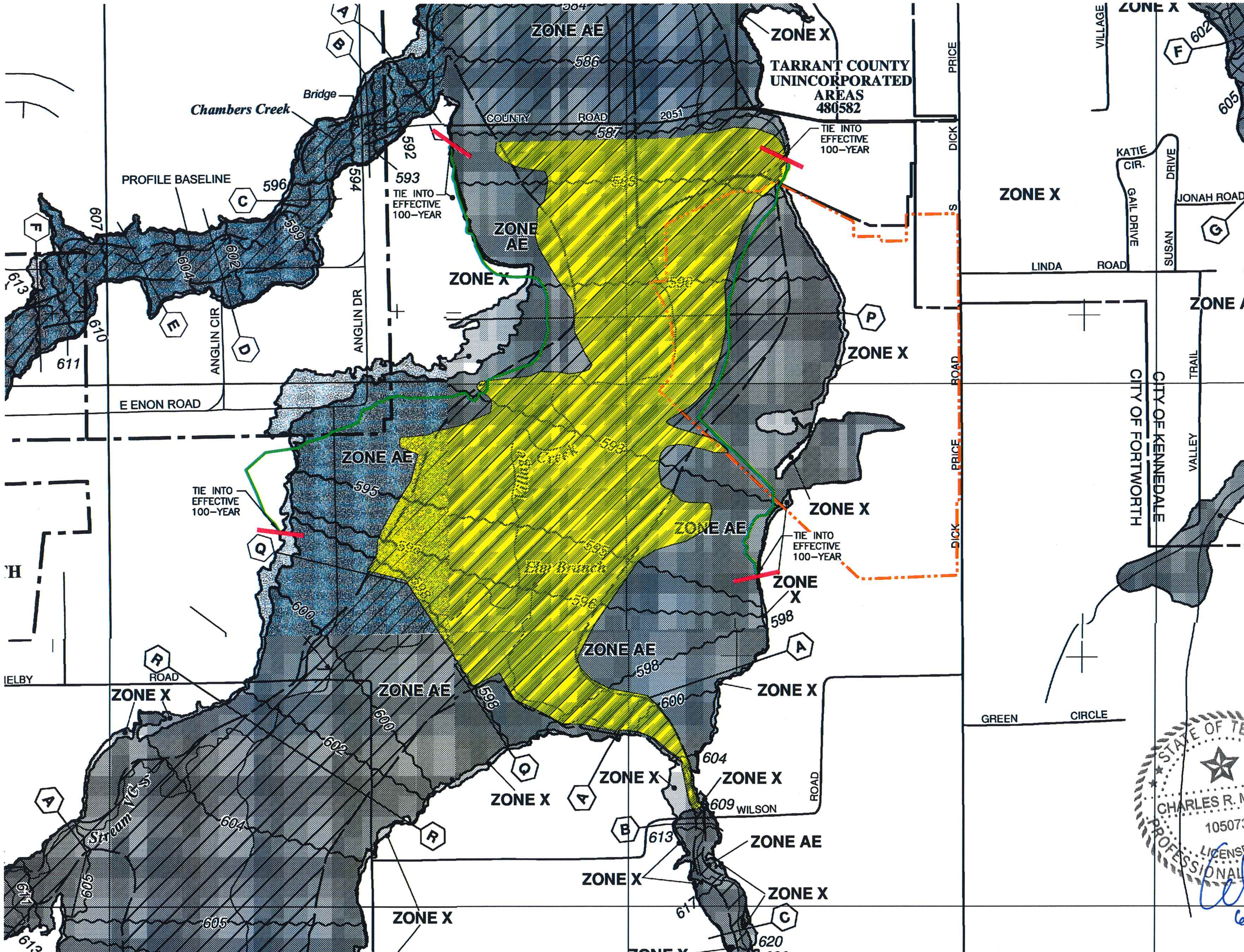

0 500 1000
SCALE IN FEET

LEGEND

PROPERTY BOUNDARY
POST–PROJECT 100–YEAR FLOODPLAIN
POST–PROJECT FLOODWAY

ZONE A	No Base Flood Elevations determined.
ZONE AE	Base Flood Elevations determined.
ZONE AH	Flood depths of 1 to 3 feet (usually areas of ponding); Base Flood Elevations determined.
ZONE AO	Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain); average depths determined. For areas of alluvial fan flooding, velocities also determined.
ZONE AR	Special Flood Hazard Area formerly protected from the 1% annual chance flood by a flood control system that was subsequently decertified. Zone AR indicates that the former flood control system is being restored to provide protection from the 1% annual chance or greater flood.
ZONE A99	Area to be protected from 1% annual chance flood by a Federal flood protection system under construction; no Base Flood Elevations determined.
ZONE V	Coastal flood zone with velocity hazard (wave action); no Base Flood Elevations determined.
ZONE VE	Coastal flood zone with velocity hazard (wave action); Base Flood Elevations determined.

FLOODWAY AREAS IN ZONE AE

The floodway is the channel of a stream plus any adjacent floodplain areas that must be kept free of encroachment so that the 1% annual chance flood can be carried without substantial increases in flood heights.

OTHER FLOOD AREAS

ZONE X	Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood.

OTHER AREAS

ZONE X	Areas determined to be outside the 0.2% annual chance floodplain.
ZONE D	Areas in which flood hazards are undetermined, but possible.

COASTAL BARRIER RESOURCES SYSTEM (CBRS) AREAS

OTHERWISE PROTECTED AREAS (OPAs)

CBRS areas and OPAs are normally located within or adjacent to Special Flood Hazard Areas.

	Floodplain boundary
	Floodway boundary
	Zone D boundary
	CBRS and OPA boundary
	Boundary dividing Special Flood Hazard Areas of different Base Flood Elevations, flood depths or flood velocities.
~~ 513 ~~~~	Base Flood Elevation line and value; elevation in feet*
(EL 987)	Base Flood Elevation value where uniform within zone; elevation in feet*

* Referenced to the North American Vertical Datum of 1988 (NAVD 88)

(A)——(A)	Cross section line
(23)- - - -(23)	Transect line
97°07'30", 32°22'30"	Geographic coordinates referenced to the North American Datum of 1983 (NAD 83)
$^{42}75^{000m}$N	1000-meter Universal Transverse Mercator grid ticks, zone 14
6000000 FT	5000-foot grid values: Texas State Plane coordinate system, north central zone (FIPSZONE 4202), Lambert Conformal Conic
DX5510 ×	Bench mark (see explanation in Notes to Users section of this FIRM panel)
• M1.5	River Mile

STATE OF TEXAS
CHARLES R. MARSH
105073
LICENSED
PROFESSIONAL ENGINEER
6-24-22

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REVISIONS

NO.	DATE	DESCRIPTION
1	06/2022	REVISED FLOODWAY AND FLOODPLAINS

Weaver Consultants Group
TBPE REGISTRATION NO. F–3727

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DRAWING A.10

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