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Part 36

MAJOR MODIFICATION Application

3A of 5

July 30, 2019

Permit Application

FOR MODIFICATION

Lea Land Landfill
OCD Facility Permit No.: NM-1-0035
Lea County, New Mexico

VOLUME III:
ENGINEERING DESIGN AND CALCULATIONS

Submitted To:

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GORDON ENVIRONMENTAL PSC

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1.0 INTRODUCTION

Lea Land LLC (the Facility) is an existing Surface Waste Management Facility (SWMF) providing oil field waste solids (OFWS) disposal services. The existing Lea Land SWMF is subject to regulation under the New Mexico Oil and Gas Rules, specifically 19.15.9.711 and 19.15.36 NMAC, administered by the Oil Conservation Division (OCD) of the NM Energy, Minerals, and Natural Resources Department (NMEMNRD). This document is a component of the "Application for Permit Modification" that proposes continued operations of the existing approved waste disposal unit; lateral and vertical expansion of the landfill via the construction of new double-lined cells; and the addition of waste processing capabilities. The proposed Facility is designed in compliance with 19.15.36 NMAC, and will be constructed and operated in compliance with a Surface Waste Management Facility Permit issued by the OCD. The Facility is owned by, and will be constructed and operated by, Lea Land LLC.

The Lea Land SWMF is one of the most recently designed facilities to meet the new more stringent standards that, for instance, mandate double liners and leak detection for land disposal. The new services that Lea Land will provide needed resources to fill an existing void in the market for technologies that exceed current OCD requirements.

The existing Lea Land Landfill is equipped with a composite liner design with an inclined leachate collection geopipe system and extraction point in the northeast corner. Liner Installation Records and Engineering Certification/CQA Reports document that the liner segments were constructed in compliance with current industry and engineering standards. Routine attempts to monitor and collect leachate flow from "Unit I" have demonstrated that oil field waste solids do not generate fluids, as no free liquids are allowed, and does not produce water.

1.1 Site Location

The Lea Land site is located approximately 27 miles northeast of Carlsbad, straddling US Highway 62-180 (Highway 62) in Lea County, NM. The Lea Land site is comprised of a 642-acre ± tract of land encompassing Section 32, Township 20 South, Range 32 East, Lea County, NM. Site access is currently provided on the south side of US Highway 62. The coordinates for the approximate center of the Lea Land site are Latitude 32°31'46.77" and Longitude -103°47'18.25".

1.2 Facility Description

The Lea Land SWMF comprises approximately 463 acres ± of the 642-acre ± site, and will include two main components: an oil field waste Processing Area and an oil field waste solids Landfill, as well as related infrastructure (i.e., access, waste receiving, stormwater management, etc.). Oil field wastes are delivered to the Lea Land SWMF from oil and gas exploration and production operations in southeastern NM and west Texas. The Permit Plans (**Attachment III.1.A**) identify the locations of the Processing Area and Landfill Disposal facilities. The proposed facilities are detailed in **Table II.1.2** (**Volume II.1**), and are anticipated to be developed in four primary phases as described in **Table II.1.3** (**Volume II.1**).

2.0 DESIGN CRITERIA

This Section, "Engineering Design" is provided as a summary of the engineering design elements for the Lea Land SWMF. The Engineering Design has been developed in accordance with the Oil and Gas Rules. More specifically, 19.15.36.17.A NMAC requires an "Engineering Design Plan" for evaporation, storage, treatment and skimmer ponds. In addition, the construction standards for these facilities are also addressed in compliance with 19.15.36.17.B NMAC.

Engineering requirements specific to landfills as referenced in 19.15.36.14.C-F NMAC, including landfill design standards, liner specifications, requirements for the soil component of composite liners, and the leachate collection and removal system are addressed herein. The Engineering Design also addresses the requirements of 19.15.36.13.M NMAC pertaining to the control of run-on and runoff from the 25-year, 24-hour design storm (**Volume III.4** and **Permit Plans, Attachment III.1.A**).

Compliance with the design standards is demonstrated on the **Permit Plans** listed in **Table III.1.1**, which are sealed by Mr. Charles W. Fiedler, P.E., of Gordon Environmental/PSC., a New Mexico Professional Engineer with extensive experience in environmental engineering and waste containment design employing geosynthetics. The **Permit Plans** are provided for reference in **Attachment III.1.A** as 11 x 17 inch (in.) plots and are also submitted as "D" size sealed plots (i.e., 24 x 36 in.) as part of this Application for Permit. The design of the Lea Land SWMF is preliminary. Construction Plans and specifications for each major element will be submitted to OCD in advance of construction.

TABLE III.1.1 - List of Permit Plans

Sheet No.	Title (ordered completely numerically)
G-001	Cover Sheet and Index
C-101	Site Plan - Existing Conditions
C-102	Site Development Plan
C-103	Existing Permit - Completion Grading Plan
C-104	Landfill Base Grading Plan
C-105	Landfill Final Grading Plan
C-106	Landfill Completion Drainage Plan
C-107	Process Area Layout
C-108	Evaporation Pond Layout
C-109	Liquid Process Area Equipment Layout
C-301	Landfill Cross-Sections
C-501	Landfill Liner System and Final Cover Details
C-502	Leachate Collection System Details
C-503	Evaporation Ponds Details
C-504	Tank Management Area Cross-Sections and Drying Pad Leak Detection Details

3.0 LANDFILL DESIGN STANDARDS

The Lea Land SWMF footprint will comprise approximately 463 acres ± of the 642-acre ± site as shown on the **Permit Plans** (**Attachment III.1.A**). The Lea Land SWMF Landfill Disposal Area footprint will be approximately 100 acres ± in size with a depth from the top of the approximately 15-foot (ft) perimeter screening berm to the base grades of approximately 35 ft below grade on the west end and from approximately natural grade; to the base grades of approximately 50 ft below grade on the east end. The base grades of the Landfill are in excess of 100 ft from groundwater. The Landfill consists of five independent units (Units I through V), with Unit I consisting of 30-acres permitted and operated in accordance with 19.15.9.711 NMAC. Units II through V will each have an independent double liner leachate collection system, cleanout risers (upgradient and downgradient), and collection sump/extraction riser located at the east end (**Permit Plans**). The Lea Land SWMF Processing Facility Area footprint will be approximately 82 acres ± in size.

3.1 Liner System

A double liner and leak detection system design is proposed for the Lea Land Surface Waste Management Facility Landfill. An alternate liner system is being proposed that meets the requirements of 19.15.36.14.C NMAC demonstrated as equivalent in the United States Environmental Protection Agency (USEPA) Hydrologic Evaluation of Landfill Performance (HELP) Model (**Volume III.4**) and has a demonstrated track record for long-term waste containment performance. The floor liner system consists of, from top to bottom:

- 24-in. protective soil/leachate drainage layer (on-site soils with permeability ≥ 2×10^{-4} cm/sec)
- 10 oz/yd^2 – 200 mil geocomposite protection/drainage liner
- 60-mil smooth HDPE primary liner
- 200-mil HDPE geonet leak detection layer
- 60-mil smooth HDPE secondary liner
- Geosynthetic Clay Liner (GCL)
- 6-in. soil compacted subgrade

The sidewall liner system consists of, from top to bottom:

- 24-in. protective soil/leachate drainage layer (on-site soils with permeability ≥ 2×10^{-4} cm/sec)
- 10 oz/yd^2 – 225 mil – 10 oz/yd^2 geocomposite protection/drainage liner
- 60-mil double-sided texture HDPE primary liner
- 200-mil HDPE geonet leak detection layer
- 60-mil double-sided texture HDPE secondary liner
- Geosynthetic Clay Liner (GCL)
- 6-in. soil compacted subgrade

The liner system is designed to meet the performance requirement of no more than one foot of leachate on the primary liner as required in 19.15.36.14.F NMAC and demonstrated in the HELP Model (**Volume III.4**).

HDPE material is proposed for the leachate collection layer, leak detection layer and liners as HDPE has proven to be the preferred material for waste containment facilities due to its durability and resistance to degradation by waste constituents. **Volume III.6** provides documentation regarding HDPE material compatibility in compliance with 19.15.36.14.D.(2)(a) NMAC.

3.2 Leachate Collection and Leak Detection System

The leachate collection system designed for the Lea Land SWMF Landfill consists of an alternate 2-ft protective soil/leachate collection layer consisting of "SC" soil material with a permeability of ≥ 2 x 10^{-4} centimeters per second (cm/sec). The leak detection system layer will incorporate a 200-mil geonet specifically prescribed for this application (**Permit Plans**). With a design transmissivity of 10 x 10^{1} square meters per second (m^2/sec), the geonet will provide fluid flow potential superior to the prescriptive soil leak detection layer of 2 ft of pervious soils (leak detection system - hydraulic conductivity of 2 x 10^{-4} cm/sec or greater) (19.15.36.14.C.(3) NMAC and (leachate collection and removal system – hydraulic conductivity of 1 x 10^{-3} cm/sec or greater) (19.15.36.14.C.(5) NMAC.

The leachate collection layer slopes at 3.78% to a 6-in. diameter standard dimension ratio (SDR) 13.5 high density polyethylene (HDPE) perforated leachate collection pipe to the center of the units and is directed at a +2.5% slope to the leachate collection sumps on the east end of the Landfill Disposal Area (**Permit Plans**). The leak detection geonet slopes at ±3.78% to the center of the units and is directed at a +2.5% slope to each of the four leak detection sumps located on the east end of the Landfill Disposal Area (**Permit Plans**). Each of the sumps is approximately 2 ft deep and contains ¾-in. to 2.0-in. diameter pre-qualified select aggregate installed on and wrapped in a geotextile cushion placed over the HDPE liners. Classification criteria for the aggregate are specified in the Liner Construction Quality Assurance (CQA) Plan (**Volume II.7**), which state that it not be angular (i.e., sharp edges which could damage the liners) or calcareous (which could degrade over time).

The fluids collected in the leachate collection and leak detection sumps will be monitored and collected by separate 12-in. diameter sidewall riser pipes, that do not penetrate the liners, in compliance with 19.15.36.14.C.(10) NMAC. The piping is demonstrated to resist degradation by the waste constituents as documented in the Geosynthetic Application and Compatibility Documentation (**Volume III.6**).

The leachate collection system pipe will consist of a minimum 6-in. diameter perforated SDR 13.5 HDPE. The leachate collection and leak detection sump riser pipes will consist of a 12-in. diameter, SDR 17 HDPE; and will be perforated or slotted for the bottom 2 ft depth within the sump (i.e., 8 ft length at 4:1 slope). HDPE piping has shown superior characteristics for waste containment applications vs. the Schedule (SCH) 80 polyvinylchloride (PVC) specified in the OCD Rule (**Tables III.1.2**). The piping is demonstrated to resist degradation by the waste constituents as documented in the Geosynthetic Application and Compatibility Documentation (**Volume III.6**).

The details in the **Permit Plans**, reflect the deployment of SDR 13.5 HDPE piping for the leachate collection pipe and leak detection sump riser pipes. Four layers of 200-mil geonet will be placed beneath the perforated pipe section in the sumps to prevent potential liner damage (**Permit Plans**). Solid-wall HDPE piping will extend from above the sumps to the permanent riser terminus shown on the **Permit Plans**.

The entire leachate collection system will be covered by 2 ft of protective soil with a hydraulic conductivity greater than or equal to 2 x 10^{-4} cm/sec. This material is available on-site, allowing for sustainable beneficial use of local resources. The HELP Model, provided in **Volume III.4**, confirms that the design meets the requirements of 19.15.36.14.F NMAC.

The leachate collection system and protective soil cover on the top of the liner system in the Landfill Disposal Area will protect the floor and sidewall liner by providing ballast and blocking sunlight (i.e., UV rays), with the upper sections of sidewall liner secured by the anchor trench as depicted on the **Permit Plans**.

TABLE III.1.2 - HDPE Leachate Collection Pipe

Characteristic	6-in. Diameter Leachate Collection Pipe	12-in. Diameter Leachate and Leak Detection Riser Pipes
	SDR 13.5 HDPE	**SDR 17 HDPE**
Dimension Ratio	13.5	17
Method of Joining	Welded	Welded
Manning's Number (n)	0.010	0.010
Outside Diameter (in.)	6.625[1]	12.75[1]
Min. Wall Thickness (in.)	0.491[1]	0.944[1]
Tensile Strength (psi)	5,000	5,000
Modulus of Elasticity (psi)	135,000	135,000
Flexural Strength (psi)	135,000	135,000

Notes:
[1]*PolyPipe, A-4 (**Attachment III.1.G**)*

3.3 Landfill Final Cover System

The final cover for the top of the Landfill Disposal Area will utilize an alternative cover system consisting of the following layers listed from top down:

- 24 in. soil vegetative (erosion) layer
- 6-in. barrier (infiltration) layer
- 12-in. intermediate cover

On-site soils will be used to construct the final cover, and the cap will be placed as the Landfill Disposal Area reaches final grades. The Landfill will have +4H:1V design sideslopes and a top slope of +5%. The final cover was modeled using the HELP Model (**Volume III.4**).

4.0 LANDFILL CONSTRUCTION

Development of the Landfill will be accomplished by constructing individual cells within the units. Detailed Construction Plans and Technical Specifications will be prepared for the Lea Land SWMF Landfill cells and submitted to OCD. Pre-qualified Liner Installation Contractors will provide and install geosynthetic liner components. The cell excavation, construction, floor grading/compaction, and geosynthetics installation will be subject to the rigorous CQA standards specified in the Liner CQA Plan (**Volume II.7**).

OCD will be provided a major milestone schedule in advance of major construction at Lea Land SWMF Landfill; and will be notified via e-mail or phone at least 3 working days prior to the installation of the primary liner. An Engineering Certification Report, sealed by a Professional Engineer with expertise in civil (geotechnical/environmental) engineering, landfill construction, and geosynthetics application will be submitted to OCD documenting compliance of completed construction with the Permit, regulatory requirements, industry standards, and the plans and specification.

The Engineering Design, as demonstrated by the Volumetric Calculations (**Volume III.2**) deliberately provides a “sustainable” configuration that does not require the import of off-site soils. The materials equation provides an excess of soils excavated (i.e., cut) and fill for the cover and perimeter berms. The in-situ and on-site fill soil will be further pre-qualified in accordance with the CQA Plan (**Volume II.7**). At least one Standard Proctor Density test will be conducted in the laboratory for each 5,000 cubic yards of subgrade soils, fill material or a change in subgrade material. These tests will be the basis for field density measurements during construction (i.e., 90% standard Proctor dry density) conducted at a minimum frequency of 4 tests/acre/lift.

The initial sequence of development is planned to involve the excavation of a unit that will likely include the development of one or more "cells"; typically, at the downgradient (i.e., east) end. The Permit Plans show a proposed "Unit II, Cell A" configuration that includes the deepest excavation for a functional initial installation. The design of Unit II provides significant capacity; with sufficient excavated soil volume to construct significant portions of the east and west perimeter berms and to provide final cover for the existing Unit I, which will include a GCL barrier in addition to the one foot of intermediate cover (**Permit Plans**).

The purpose of the west berm is two-fold: to manage stormwater run-on by directing it away from and around the landfill; and to provide visual and environmental screening from adjacent areas. The east berm also assists with stormwater control; and the lower elevations of the east face are also lined as-part of the run-off evaporation basin configuration. The berms will be constructed using pre-qualified soils and compacted to 90% Standard Proctor in maximum 9-inch thick horizontal lifts. Construction of these and future berms will be in accordance with the CQA Plan (**Volume II.7**); and they will serve as the constructed platform for landfill anchor trench installation as unit construction progress north.

The subgrade surface for the liner will be inspected to confirm the absence of any deleterious materials, abrupt changes in slope, evidence of erosion, etc. The compliance of the completed subgrade construction will be confirmed prior to secondary liner installation and documented in the Engineering Certification Report and in accordance with the CQA Plan (**Volume II.7**).

A reinforced Geosynthetic Clay Liner (GCL) will be placed on the prepared subgrade. Above the GCL, a 60-mil HDPE secondary liner will be installed for the proposed cells and in direct contact with the GCL. Installation of the leak detection system (geonet; geotextile; or combined geocomposite); sump aggregate and leak detection riser pipes in the sumps will follow. The 60-mil HDPE primary liner, above the leak detection system is overlain by a geocomposite liner protection and drainage layer which is overlain by 2-feet of on-site soils that contains the leachate collection system and serves to protect the double liner system. The installation of all soil and geosynthetic components will meet or exceed the requirements of 19.15.36.14.C NMAC, as detailed in the CQA Plan. Finally, the GCL, secondary HDPE liner, leak detection system components, primary HDPE liner and geocomposite will be secured in the common anchor trench at the crest of the Landfill sideslope**.** The anchor trench will be carefully backfilled with select on-site soils compacted to 90% of standard Proctor dry density by mechanical and/or hand-tamping devices as required by the CQA Plan. Documentation will be provided in the Engineering Certification Report submitted to OCD upon completion of construction.

5.0 EVAPORATION POND DESIGN STANDARDS

The designs for the evaporation ponds are identical, except that Pond elevations are staged depending on their site location (**Permit Plans; Attachment III.1.A**). Each pond is approximately 420 ft north-south by 200 ft east-west as measured at the top of the surrounding berms, for a footprint of 2.0 ± acres each. The floor of the ponds is designed with a 2.8% slope to facilitate drainage in the leak detection system to the two sumps in each pond situated on the interior sidewall.

Because each pond berm has a generally uniform top elevation, the 2.8% floor slope creates a pond depth that ranges from a maximum of 10 ft to a minimum of just less than 8 ft. The maximum water depth is designed at the sump locations and does not exceed 8.5 ft. Maintaining a high-water elevation in the southern Ponds and dropping the water surface 0.5 ft per pond as each Pond discharges north; will provide a freeboard in excess of 3.5 ft for each pond. This is more than adequate to meet the 3 ft minimum freeboard standard; while also accommodating the minimal impact potential of rainfall or wave action (**Volume III.9**). The resultant capacity of each pond is approximately 9.5 acre-ft, not including freeboard, below the maximum 10 acre-ft volume prescribed by 19.15.36.17.B(12) NMAC. The normal water surface is marked in each pond to define the available freeboard. **Attachment III.1.F** provides pond capacity calculations.

Section 6.0 (Pond Construction) below and the CQA Plan (**Volume II.7**) provide documentation on the installation of berms, soil subgrade, and geosynthetics. Exceeding the standards specified in 19.15.36.17.B(4) NMAC, both the exterior and interior sidewalls of all of the Ponds have design slopes of 3:1. The top platform of the berms surrounding the Ponds has a minimum design width of 20 ft, which is more than adequate for the 2 ft anchor trench shown on the **Permit Plans**; and to accommodate pipe risers.

5.1 Liner System

A double liner and leak detection system design is proposed for each pond. An alternate liner system is being proposed that meets the requirements of 19.15.36.17.B(9) NMAC and has a demonstrated track record for long-term waste containment performance. The pond liner system consists of, from top to bottom:

- 60-mil HDPE primary liner
- 200-mil HDPE geonet leak detection layer
- 60-mil HDPE secondary liner
- GCL under the leak detection sumps
- 6-in. compacted soil subgrade

HDPE material is proposed for the liners and leak detection layer as HDPE has proven to be the preferred material for waste containment facilities due to its durability and resistance to degradation by waste constituents. **Volume III.6** provides documentation regarding HDPE material compatibility in compliance with 19.15.36.17.B(3) NMAC

5.2 Leak Detection System

The leak detection system layer designed for the ponds consists of a 200-mil geonet specifically prescribed for these applications (**Permit Plans**). With a design transmissivity of 1×10^1 m^2/sec, the geonet will provide fluid flow potential superior to the prescriptive leak detection layer of 2 ft of pervious soils (19.15.36.17.B(9) NMAC).

The underlying 60-mil HDPE secondary liner, the 200-mil geonet leak detection layer, and the overlaying 60-mil HDPE primary liner, has a design slope at 2% to the 2 leak detection sumps located in each pond (**Permit Plans**). Fluids potentially collected in the leak detection layer, which encompasses the entire footprint for each pond, are directed with the 2% slope to the leak detection sumps. Each of the sumps will be approximately 2 ft deep, as measured from the secondary liner to the primary liner. The sumps will be filled with nominal ¾-in. to 2.0-in. diameter pre-qualified select aggregate installed on a geotextile cushion placed over the secondary liner. Classification criteria for the aggregate are specified in the CQA Plan (**Volume II.7**), which state that it not be angular (i.e., sharp edges which could damage the liners) or calcareous (which could degrade over time).

The fluids potentially collected in the leak detection sumps will be monitored and removed through a 6-in. diameter, SDR 17 HDPE sidewall riser pipes that do not penetrate the liners. The leak detection sump riser pipes will be perforated or slotted for the bottom 2 ft depth within the sump (i.e., 6 ft length at 3:1 slope). HDPE piping has shown superior characteristics for waste containment applications (**Table III.1.3**). The piping is demonstrated to resist degradation by the waste constituents as documented in **Volume III.6**.

TABLE III.1.3 - HDPE Sump Riser Pipe

Characteristic	6-in. Diameter Leak Detection Riser Pipes SDR 17 HDPE
Dimension Ratio	17
Method of Joining	Welded
Manning's Number (n)	0.010
Outside Diameter (in.)	6.625[1]
Min. Wall Thickness (in.)	0.491[1]
Tensile Strength (psi)	5,000
Modulus of Elasticity (psi)	135,000
Flexural Strength (psi)	135,000

Notes:
*[1]PolyPipe, A-4 (**Attachment III.1.G**)*

The details in the **Permit Plans** reflect the deployment of SDR 17 HDPE piping for the leak detection sump riser pipes. Four layers of 200-mil geonet will be placed beneath the beveled edge of the perforated risers in the sumps to prevent potential liner damage (**Permit Plans**). Solid-wall HDPE piping will extend from above the sumps to the permanent risers shown on **Permit Plans**. The sidewall liners and leak detection geonet will be secured by the anchor trench as depicted on the **Permit Plans.**

6.0 POND CONSTRUCTION

Detailed Construction Plans and Technical Specifications will be prepared for the proposed ponds and submitted to OCD. Pre-qualified Liner Installation Contractors will provide and install geosynthetic components. The berm construction, floor grading/compaction, and geosynthetics installation will be subject to the rigorous CQA standards specified in **Volume II.7**.

OCD will be provided a major milestone schedule in advance of construction; and notified via e-mail or phone at least 3 working days prior to the installation of the primary liner in compliance with 19.15.36.17.B(10) NMAC. An Engineering Certification Report, sealed by a Professional Engineer with expertise in civil (geotechnical/environmental) engineering, will be submitted to OCD documenting compliance of completed construction with the Permit, regulatory requirements, industry standards, and the plans and specification.

The Engineering Design presented on the **Permit Plans** (**Attachment III.1.A**) deliberately provides a "sustainable" and geotechnically suitable configuration that does not require import of off-site soils. The materials equation provides a balance between soils excavation (i.e., pond) and fill for the sidewalls. The in-situ and on-site fill soil will be pre-qualified in accordance with the CQA Plan (**Volume II.7**). At least one standard Proctor dry density test will be conducted in the laboratory for each pond footprint, 5,000 cubic yards (cy) of fill material for berms or change in subgrade material. These tests will be the basis for field density measurements during construction (i.e., 90% standard Proctor dry density) conducted at a minimum frequency of 4 tests/acre/lift.

Fill for the berms will be placed in horizontal compacted lifts that do not exceed 9 in. in thickness. The subgrade surface will be inspected to confirm the absence of any deleterious materials that may impact the secondary liner system, abrupt changes in slope, evidence of erosion, etc. The compliance of the completed subgrade construction shall be confirmed prior to secondary liner installation and documented in the Engineering Certification Report.

The double liner and leak detection system design, planned for the ponds, consists of proven technology with a demonstrated track record of long-term waste containment performance. The secondary liner proposed for the ponds, consists of a smooth 60-mil HDPE geomembrane placed in direct contact with a prepared and compacted soil subgrade, certified in accordance with the CQA Plan (**Volume II.7**). The same HDPE material will be used for the primary liner and the geonet for the leak detection layer. HDPE has proven to be the preferred material for waste containment facilities due to its durability and resistance to attack by waste constituents.

Volume III.6 provides documentation regarding liner and leak detection material compatibility in compliance with 19.15.36.17.B(3) NMAC. An additional protective layer of 60-mil HDPE (22.5 ft x 40 ft ±) will be welded above the primary Pond liner where active wastewater discharge will occur (**Permit Plans**). This will protect the Pond liner from hydrostatic force, mechanical damage, etc. External discharge lines and leak detection system discharge lines will not penetrate the liner. The CQA Plan (**Volume II.7**) provides the most current technical specifications for the geosynthetics.

Fluid in the Ponds will protect the floor and lower sidewall liner by providing ballast and deflecting sunlight (i.e., UV rays). The upper sections of pond sidewall liner will be secured by the anchor trench. The anchor trench will be carefully backfilled with select on-site soils compacted to 90% of standard Proctor dry density by mechanical and/or hand-tamping devices (per the CQA Plan). Documentation will be provided in the Engineering Certification Report submitted to OCD upon completion of construction.

Although the freeboard zone of the pond sidewall liner will be exposed to the elements, recent research indicates that exposed HDPE in similar environments has a functional longevity in excess of 25 years (**Attachment III.1.B**). Gordon Environmental/PSC has inspected several similar water storage ponds in New Mexico and has found exposed geomembrane liners to be functionally intact well after 25 years of exposure to the elements.

7.0 POND OPERATION

Detailed plans for the operation of the Ponds are prescribed in the Operations, Maintenance, and Inspection Plan (**Volume II.1**). Essentially, it is anticipated that some fluids may accumulate in the leak detection sumps as a result of condensation, construction water, etc. As described in **Volume II.1**, the leak detection sumps will be monitored at least monthly for the presence of fluids, which may be extracted and tested when the level in the sump(s) exceeds 24 in. A reduced monitoring frequency may be proposed to OCD dependent upon historical results. The design of the Ponds allows for isolation of potential leaks into isolated drainage basins, facilitating necessary evaluation or repair by allowing each pond to be emptied.

8.0 PROCESS AREA TANK CONTAINMENT

As proposed in this Application, produced water receiving tanks, produced water settling tanks, and the crude oil receiving tanks depicted in **Attachment III.1.C** and oil sales tanks as depicted in **Attachment III.1.D** will be installed in the excavated tank farm as shown on the **Permit Plans**. Detailed operations of the tanks are described in the Operations, Maintenance, and Inspection Plan (**Volume II.1**), and a schematic of the process area is provided in **Attachment III.1.E**. The tanks will be constructed with an underlying, continuous, system which is designed to capture any fluids within the watershed of the tank farm. The design of the processing facilities are preliminary. Construction plans and specifications for each major element will be submitted to OCD in advance of installation.

The secondary containment liner in the tank area is a 30-mil polyester liner (XR-5 8130 Reinforced Geomembrane). The use of the XR-5 8130 Reinforced Geomembrane in the tank area is primarily based on the chemical compatibility and puncture resistance of the material compared to either PVC or HDPE material. The chemical resistance of the XR-5 material exceeds the chemical compatibility of either PVC or HDPE to hydrocarbon products (see Chemical Resistance Chart, Page 13, "Technical Data and Specifications for XR-5", **Attachment III.1.H**). Since PVC material has marginal chemical resistance in a hydrocarbon environment, physical properties of the XR-5 geomembrane (**Attachment III.1.H**) are compared to 60-mil HDPE geomembrane (**Attachment III.1.I**) as shown in **Table III.1.4.**

The necessary storage capacity for the interconnected tank/containment system will be sufficiently managed by the proposed lined volume of the Ponds constructed in sequence corresponding to market conditions. In the unlikely event of a total failure of all affected storage units, the contents of the tanks will flow into the ponds, which have a lined storage capacity of 884,400 barrels (bbl) ± (excluding freeboard). When the freeboard is included, the storage capacity of the ponds is over 1,714,600 bbl, which results in a net surplus of over 830,200 bbl (i.e., 1.94%). The entire volume of the proposed storage tanks will be 70,000 bbl, providing a net excess capacity of over 760,200 bbl. Thus, the Ponds will hold the entire volume of the receiving/settling tanks within the required permanent freeboard of 3 ft.

TABLE III.1.4 - Physical Properties: XR-5 8130 Reinforced Geomembrane and 60-mil HDPE Geomembrane

Property	XR-5 8130	60-mil HDPE
Thickness	30-mil	60-mil
Tear Strength	40 lbs	42 lbs
Puncture Resistance	275 lbs	108 lbs
Break Strength	400 lbs/in.	228 lbs/in.
Break Elongation	25%	700%
Hydrostatic Resistance	800 psi	> 450 psi
Hydraulic Conductivity	1 x 10^{-12} cm/sec	2 x 10^{-13} cm/sec
Seam Properties		
Shear Strength	500 lbs	120 lbs/in.
Peel Strength	40 lbs/2 in.	91 lbs/in.

The maximum proposed number of interconnected tanks is five 1,000 bbl tanks for a total of 5,000 bbl. Allowing for an additional 30% capacity will require a minimum of 6,500 bbl of bermed capacity in the tank farm. The containment area is conservatively sized to surround the entire tank farm, which results in a holding capacity of 13,100 bbl, and is 12,100 bbl greater than the capacity of the largest tank (1,000 bbl) and 6,600 bbl greater than the combined connected tank volume, including a 30% factor of safety within the containment area. Therefore, the containment area surrounding the receiving/settling tanks is more than sufficient. Included in this Section is a spreadsheet (**Attachment III.1.F**), which identifies each of the proposed tanks and Evaporation Ponds in this Application. The design of the processing facilities are preliminary. Construction plans and specifications for each major element will be submitted to OCD in advance of installation.

9.0 STABILIZATION AND SOLIDIFICATION AREA

The design for the stabilization and solidification (S&S) area relies on many of the Pond design characteristics, except that the S&S area is designed to allow dump trucks and tanker trucks delivering materials that require stabilization and/or solidification to discharge directly onto the S&S area concrete unloading pad. (**Permit Plans, Attachment III.1.A**). The initial S&S design area covers approximately 5-acres and measures 660 ft north-south by 330 ft east-west at the surrounding walls. The floor of this area is designed with a 2% slope to facilitate drainage on the concrete liner and in the leak detection system to collect in a sump situated at the downgradient end of the S&S area.

Because the three walls have a uniform top elevation, the 2% floor slope creates a containment depth that ranges from a minimum of 5 ft at the unloading pad to a maximum of 20 ft in the sump at the downgradient end. The concrete floor slope allows for up to a 5-ft-thick protective and operational cover on the floor. This slope also provides operation capacity for the S&S function proposed for this area while providing the capacity to meet the 3 ft minimum freeboard standard and accommodating the minimal impact potential of rainfall. The resultant capacity of the S&S area is approximately 5.6 acre-ft, not including freeboard, well below the maximum 10 acre-ft volume prescribed by 19.15.36.17.B(12) NMAC. The design of the processing facilities are preliminary. Construction plans and specifications for each major element will be submitted to OCD in advance of installation.

9.1 Liner System

The S&S area is designed with a leak detection system that meets the requirements of 19.15.36.17.B(9) NMAC utilizing concrete and a geomembrane to provide secondary containment. Section 6.0 (Pond Construction) and the CQA Plan (**Volume II.7**) will provide documentation on the installation of walls, soil subgrade, and geosynthetics. The construction standards specified are as conservative as the standards of 19.15.36.17.B(4) NMAC, vertical concrete containment walls and the concrete floor provide the primary containment. A 60-mil HDPE geomembrane provides the secondary containment and the opportunity for leak detection at the sump. The S&S Area liner system consists of, from top to bottom:

- 5 ft protective soil and operational layer
- 1.5 thick structural concrete primary liner
- 60-mil HDPE secondary liner
- GCL under the leak detection sumps
- 6-in. compacted soil subgrade

HDPE material is proposed for the leak detection layer as HDPE has proven to be the preferred material (compared to PVC) for waste containment facilities due to its durability and resistance to attack by waste constituents. **Volume III.6** provides documentation regarding HDPE material compatibility in compliance with 19.15.36.17.B(3) NMAC

9.2 Leak Detection System

The leak detection system layer designed for the S&S area is designed to meet the requirements of 19.15.36.17.B(9) NMAC and consists of the underlying 60-mil HDPE secondary liner beneath the 1.5-inch structural concrete process surface. There is are 2-ft of cushion soil to allow for the placement of the structural concrete. All structural concrete will be installed in accordance with American Concrete Institute (ACI) standards (i.e., both Design Codes and Construction Specifications) Detail for these standards will be included in the Project manual that accompanies the Construction Plans that will be developed and provided to the OCD prior to construction. Both liners have a design slope at 2% to the liquid collection and leak detection sumps located on the downgradient end of the S&S area. Fluids collected in the leak detection layer, which encompasses the entire footprint of the S&S area, are directed with the 2% slope to the leak detection sump for monitoring and removal. This sump will be approximately 2 ft deep, as measured from the secondary liner to the primary liner. The sump will contain ¾-in. to 2.0-in. diameter pre-qualified select aggregate installed on a geotextile cushion placed over the secondary liner. Classification criteria for the aggregate are specified in the CQA Plan (**Volume II.7**), which state that it not be angular (i.e., sharp edges which could damage the liners) or calcareous (which could degrade over time).

The fluids collected in the leak detection sump will be monitored and removed through a 12-in. diameter, SDR 13.5 HDPE vertical riser pipe that does not penetrate the liners. The leak detection sump riser pipe will be perforated or slotted for the bottom 2 ft depth within the sump. HDPE piping has shown superior characteristics for waste containment applications (**Table III.1.3**). The piping is demonstrated to resist degradation by the waste constituents as documented in **Volume III.6**. The details in the **Permit Plans** reflect the deployment of SDR 13.5 HDPE piping for the leak detection sump riser pipe.

Four layers of geonet will be placed beneath the perforated pipe section of the riser in the sump to prevent potential liner damage. Solid-wall HDPE piping will extend from above the sump to the permanent riser terminus shown on the **Permit Plans**. The geomembrane liners will be secured by the anchor trench as depicted on the **Permit Plans**.

9.3 Stabilization & Solidification Area Construction

Detailed Construction Plans and Technical Specifications will be prepared for the proposed S&S area and submitted to OCD. Pre-qualified Liner Installation Contractors will provide and install geosynthetic liner components. The design of this processing facility is preliminary. Construction plans and specifications for each major element will be submitted to OCD in advance of installation. The concrete construction, floor grading/compaction, and geosynthetics installation will be subject to the rigorous CQA standards specified in **Volume II.7**.

OCD will be provided a major milestone schedule in advance of construction; and notified via e-mail or phone at least 3 working days prior to the installation of the primary liner in compliance with 19.15.36.17.B(10) NMAC. An Engineering Certification Report, sealed by a Professional Engineer with expertise in civil (geotechnical/environmental) engineering and geosynthetics design, will be submitted to OCD documenting compliance of completed construction with the Permit, regulatory requirements, industry standards, and the plans and specification.

The Engineering Design presented on the **Permit Plans** (**Attachment III.1.A**) deliberately provides a "sustainable" configuration that does not require import of off-site soils. The materials equation provides a balance between soils excavation (i.e., S&S area) and backfill. The in-situ and on-site fill soil will be pre-qualified in accordance with the CQA Plan (**Volume II.7**). At least one standard Proctor dry density test will be conducted in the laboratory for every 5,000 cubic yards (cy) of fill material for backfill, or for every change in subgrade material. These tests will be the basis for field density measurements during construction (i.e., 90% standard Proctor dry density) conducted at a minimum frequency of 4 tests/acre/lift.

Fill for the backfill will be placed in horizontal compacted lifts that do not exceed 9 in. in thickness. The subgrade surface will be inspected to confirm the absence of any deleterious materials, abrupt changes in slope, evidence of erosion, etc. The compliance of the completed subgrade construction shall be confirmed prior to secondary liner installation and documented in the Engineering Certification Report.

The liner/leak detection system design planned for the S&S area consists of proven technology with a demonstrated track record of long-term waste containment performance. The primary liner proposed for the area, consists of a smooth 60-mil HDPE geomembrane placed in direct contact with a prepared and compacted soil subgrade, certified in accordance with the CQA Plan (**Volume II.7**). HDPE has proven to be the preferred material (compared to PVC) for waste containment facilities due to its durability and resistance to attack by waste constituents. **Volume III.6** provides documentation regarding liner and leak detection material compatibility in compliance with 19.15.36.17.B(3) NMAC. Leak detection system discharge lines will not penetrate the liner. The CQA Plan (**Volume II.7**) provides the most current technical specifications for the geosynthetics.

Containment for the drying pad will consist of the concrete walls and floor to control the materials deposited for processing. A protective soil cover layer in the S&S area will protect the concrete floor and wall by providing an operational cushion. The geomembrane primary liner will be secured by the anchor trench (**Permit Plans**). The anchor trench will be carefully backfilled with select on-site soils compacted to 90% of standard Proctor dry density by mechanical and/or hand-tamping devices (per the CQA Plan). Documentation will be provided in the Engineering Certification Report submitted to OCD upon completion of construction.

9.4 Stabilization and Solidification Area Operation

Detailed plans for the operation of the S&S area are prescribed in the Operations, Maintenance, and Inspection Plan (**Volume II.1**). To ensure compliance with the capacity limits imposed on the operation of this area, volumes in and out of this area will be tracked to document the volume in processing at any time. Equipment operating within the S&S area may be equipped with Global Positioning System (GPS) equipment (see **Attachment III.1.J** for information on the Computer Aided Earthmoving System provided by Caterpillar) to monitor the location of the equipment relative to the concrete floor and sidewall system. This system may be implemented to maintain adequate separation of equipment and the concrete working surface during the stabilization and solidification operation. Material that has completed the S&S operation will be relocated to the Landfill for disposal. Solidification material will be excavated from borrow sources within the solid waste management facility.

10.0 FACILITY DRAINAGE DESIGN

The **Permit Plans**, **Attachment III.1.A,** show the stormwater management systems that will be employed to manage both run-on and run-off for the Lea Land Landfill and Processing Facilities. The design event, pursuant to 19.15.36.13.M NMAC (i.e., 25-year, 24-hour storm) will be managed by a series of drainageways that surround the proposed Ponds, Processes, and Landfill and capture stormwater from other on-site areas.

Stormwater retention and detention basins are planned for installation as shown on the **Permit Plans**; and the Stormwater Management Plan is included in **Volume III.3** that demonstrates the efficacy of the proposed system.

The berms surrounding the Landfill and processing area have a maximum exterior slope of 4:1, and an average height of less than 20 ft, minimizing the potential for soil erosion. The drainageways, retention and detention basins will be regularly inspected and cleaned, as necessary. Stormwater retention basins (contact water basins) are lined with a 40-mil HDPE material to minimize infiltration and enhance evaporation.

ATTACHMENT III.1.A

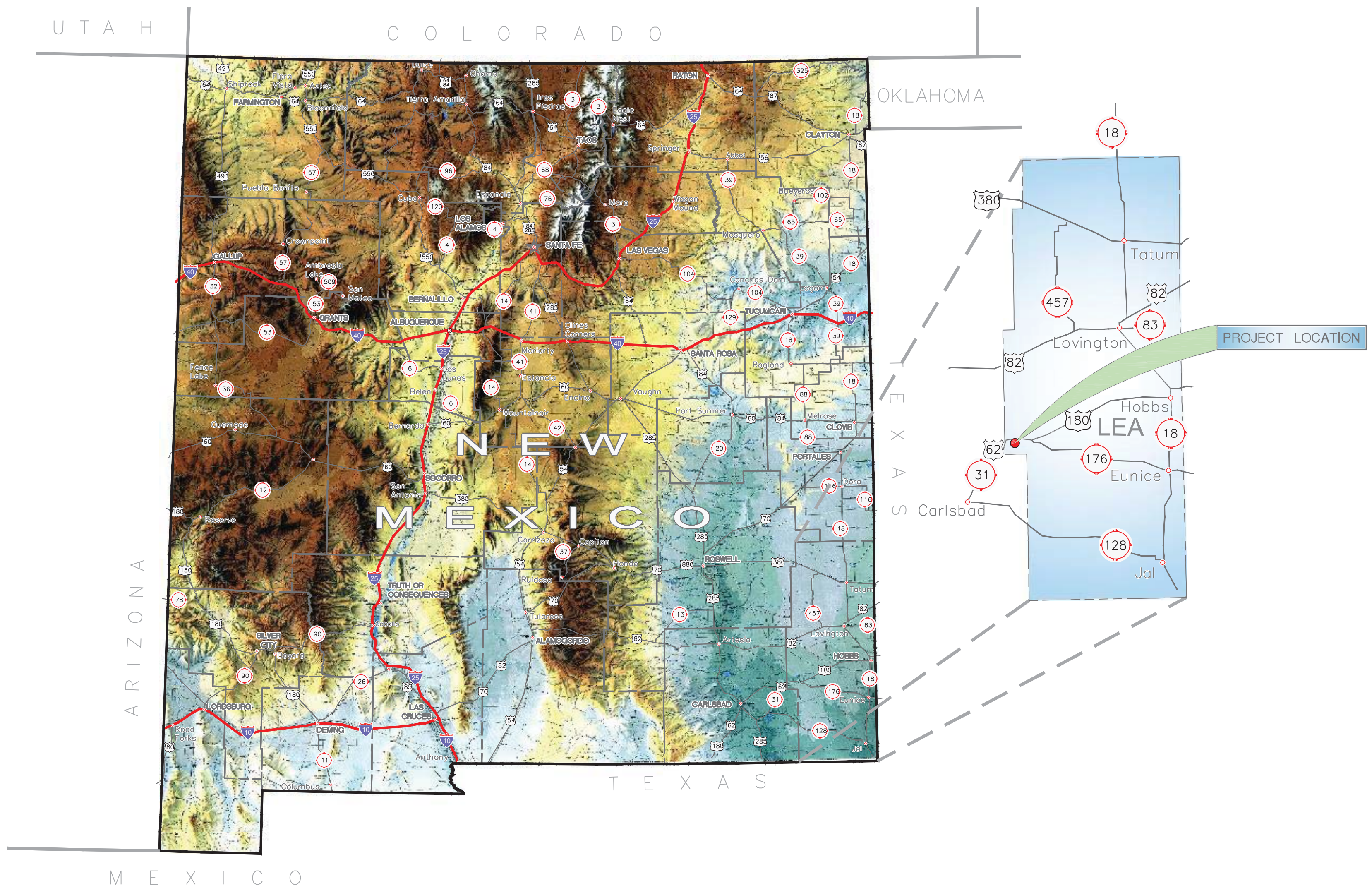
PERMIT PLANS

Sheet No.	Title (ordered completely numerically)
G-001	Cover Sheet and Index
C-101	Site Plan - Existing Conditions
C-102	Site Development Plan
C-103	Existing Permit - Completion Grading Plan
C-104	Landfill Base Grading Plan
C-105	Landfill Final Grading Plan
C-106	Landfill Completion Drainage Plan
C-107	Process Area Layout
C-108	Evaporation Pond Layout
C-109	Liquid Process Area Equipment Layout
C-301	Landfill Cross-Sections
C-501	Landfill Liner System and Final Cover Details
C-502	Leachate Collection System Details
C-503	Evaporation Ponds Details
C-504	Tank Management Area Cross-Sections and Drying Pad Leak Detection Details

LEA LAND, LLC. SURFACE WASTE MANAGEMENT FACILITY

PERMIT APPLICATION

LEA COUNTY, NEW MEXICO



GORDON ENVIRONMENTAL PSC
333 Rio Rancho Blvd. NE
Suite 400
Rio Rancho, NM 87124
505.867.6990

SHEET INDEX

GORDON ENVIRONMENTAL PSC

CHARLES WALTER FIEDLER
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19731
06/26/2019
PROFESSIONAL ENGINEER

Charles W. Fiedler, P.E.

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LEA LAND LLC SURFACE WASTE MANAGEMENT FACILITY

PERMIT APPLICATION

LEA COUNTY, NEW MEXICO

KEY PLAN

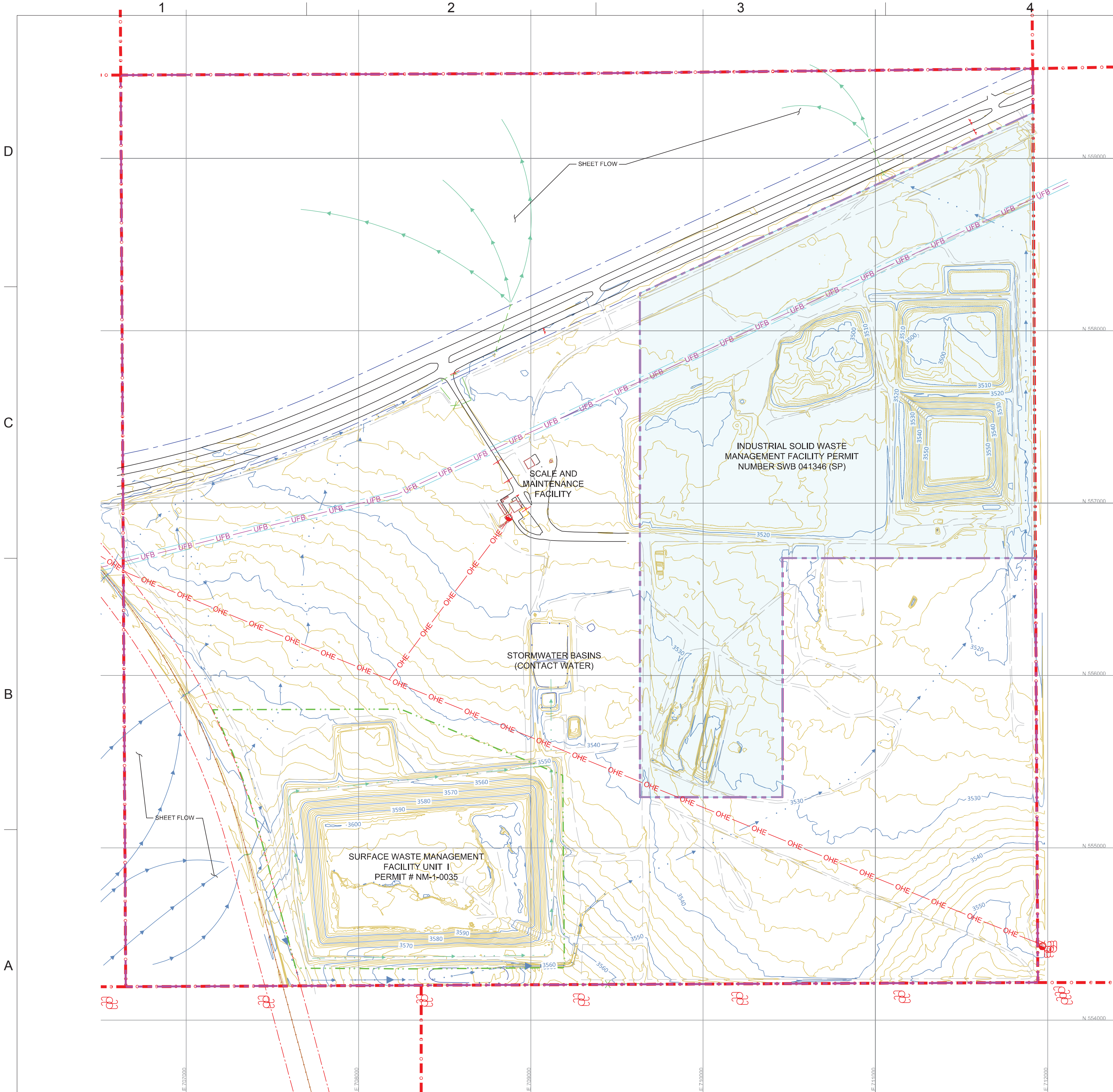
NO	DATE	DESCRIPTION
1	06/03/2019	REVISIONS PER OCD REVIEW
-	04/10/19	OCD PERMIT APPLICATION REVIEW

ISSUING OFFICE: RIO RANCHO PROJECT NO: 0416.18

COVER SHEET AND INDEX

G-001

FILE NAME: X:\2018\0416.18\02_DSGN\02_DWG\010_GEN\G-001-0416-18.dwg LAYOUT NAME: G-001 PRINTED: Wednesday, June 26, 2019 - 12:27pm USER: dmill



LEGEND

- SECTION BOUNDARY
- PROPERTY BOUNDARY
- EXISTING OCD PERMIT BOUNDARY
- EXISTING HIGHWAY RIGHT-OF-WAY
- EXISTING RAILWAY RIGHT-OF-WAY
- EXISTING TELECOM EASEMENT
- OHE — OHE: EXISTING OVERHEAD ELECTRICAL LINE
- UFB — UFB: EXISTING UNDERGROUND FIBER-OPTIC LINE
- EXISTING EDGE OF PAVED ROADWAY
- EXISTING EDGE OF UNPAVED ROADWAY
- EXISTING CENTERLINE OF RAILWAY
- EXISTING DRAINAGE RUN-ON FLOW PATH
- EXISTING DRAINAGE RUNOFF FLOW PATH
- 3600: EXISTING GRADE ELEVATION CONTOUR - INDEX (10')
- EXISTING GRADE ELEVATION CONTOUR - INTERMEDIATE (2')
- EXISTING CULVERT
- TRANSMISSION LINE
- NOT IN OCD PERMIT
- N 5580000 / E 7080000: SITE GRID

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CHARLES WALTER FIEDLER NEW MEXICO 19731 06/26/2019 PROFESSIONAL ENGINEER

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LEA LAND LLC SURFACE WASTE MANAGEMENT FACILITY

PERMIT APPLICATION

LEA COUNTY, NEW MEXICO

KEY PLAN

N

0 300' 600'

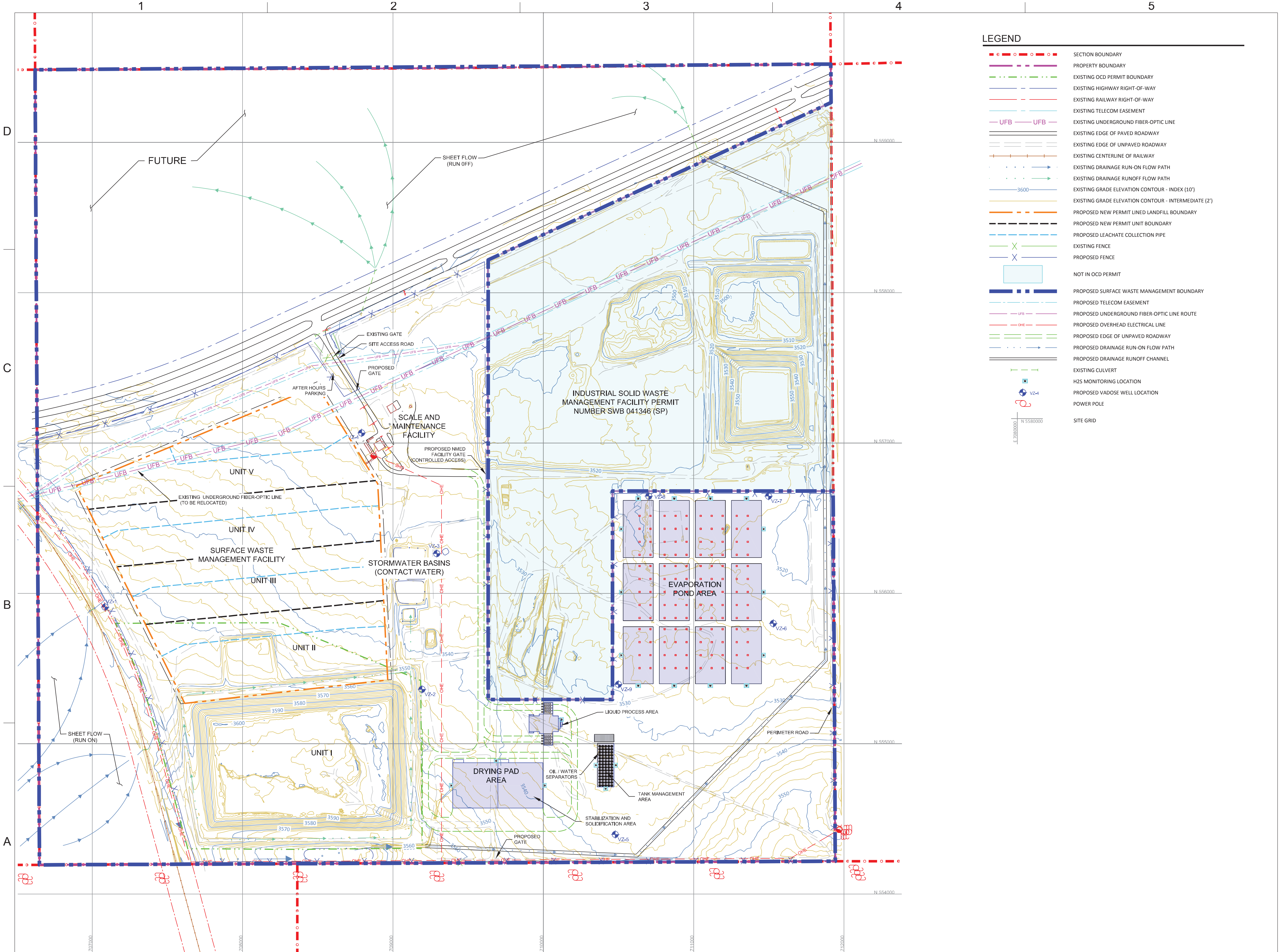
NO	DATE	DESCRIPTION
1	06/03/2019	REVISIONS PER OCD REVIEW
-	04/10/19	OCD PERMIT APPLICATION REVIEW

ISSUING OFFICE: RIO RANCHO PROJECT NO: 0416.18

SITE PLAN EXISTING CONDITIONS

C-101

FILE NAME: X:\2018\0416.18\02_DSGN\02_DWG\050_CIVIL\03_SHEETS\C-101-0416-18 REVISED.dwg LAYOUT NAME: C-101 PRINTED: Wednesday, June 26, 2019 - 10:25am USER: dmill



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- PROPOSED NEW PERMIT UNIT BOUNDARY
- PROPOSED LEACHATE COLLECTION PIPE
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- PROPOSED FENCE
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- PROPOSED TELECOM EASEMENT
- PROPOSED UNDERGROUND FIBER-OPTIC LINE ROUTE
- PROPOSED OVERHEAD ELECTRICAL LINE
- PROPOSED EDGE OF UNPAVED ROADWAY
- PROPOSED DRAINAGE RUN-ON FLOW PATH
- PROPOSED DRAINAGE RUNOFF CHANNEL
- EXISTING CULVERT
- H2S MONITORING LOCATION
- PROPOSED VADOSE WELL LOCATION
- POWER POLE
- SITE GRID

GORDON ENVIRONMENTAL PSC

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LEA LAND LLC SURFACE WASTE MANAGEMENT FACILITY

PERMIT APPLICATION

LEA COUNTY, NEW MEXICO

KEY PLAN

N

0 300' 600'

NO	DATE	DESCRIPTION
1	06/03/2019	REVISIONS PER OCD REVIEW
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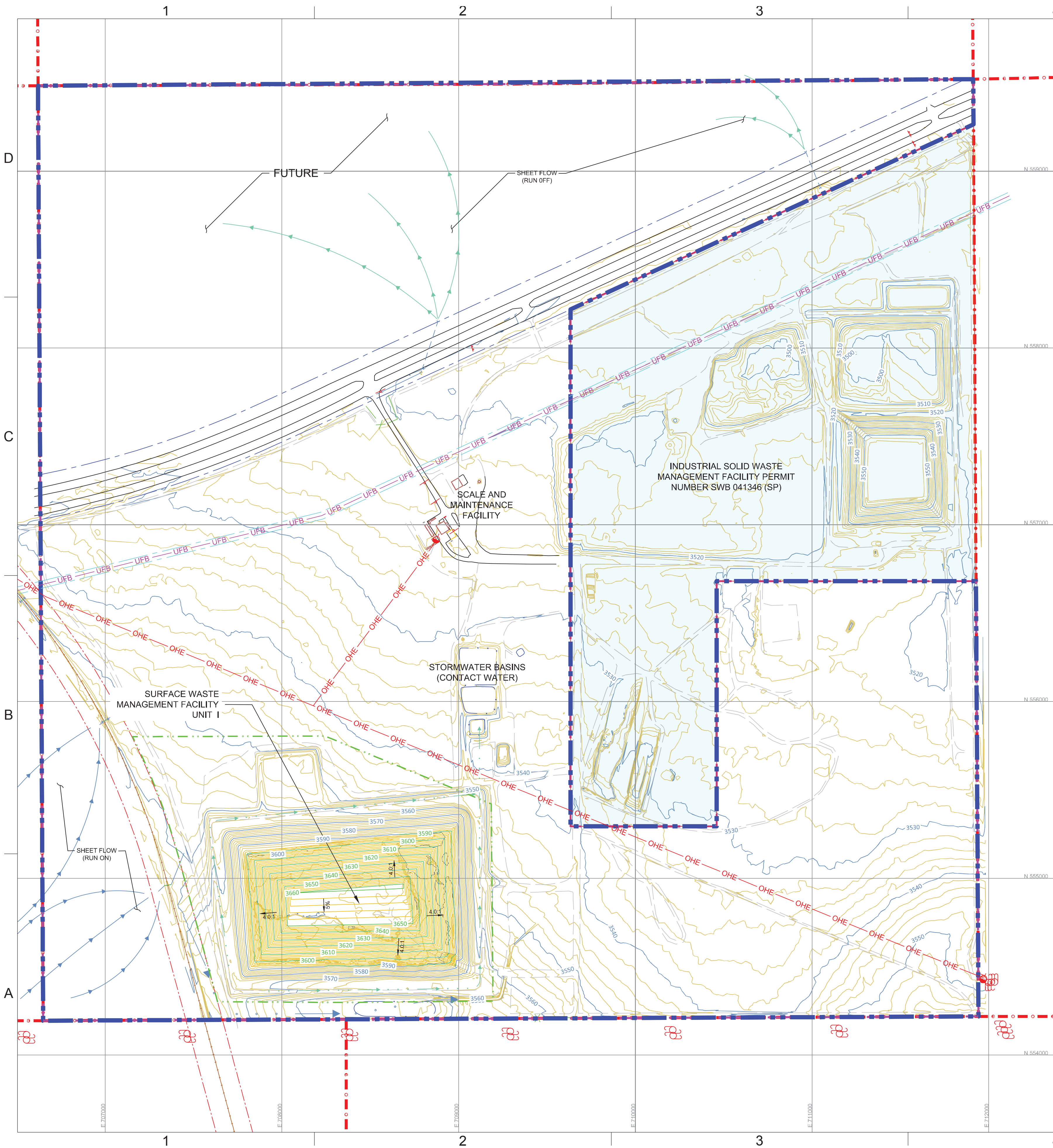
SITE DEVELOPMENT PLAN

C-102



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- POWER POLE
- N 5580000 / E 7080000: SITE GRID

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LEA LAND LLC SURFACE WASTE MANAGEMENT FACILITY

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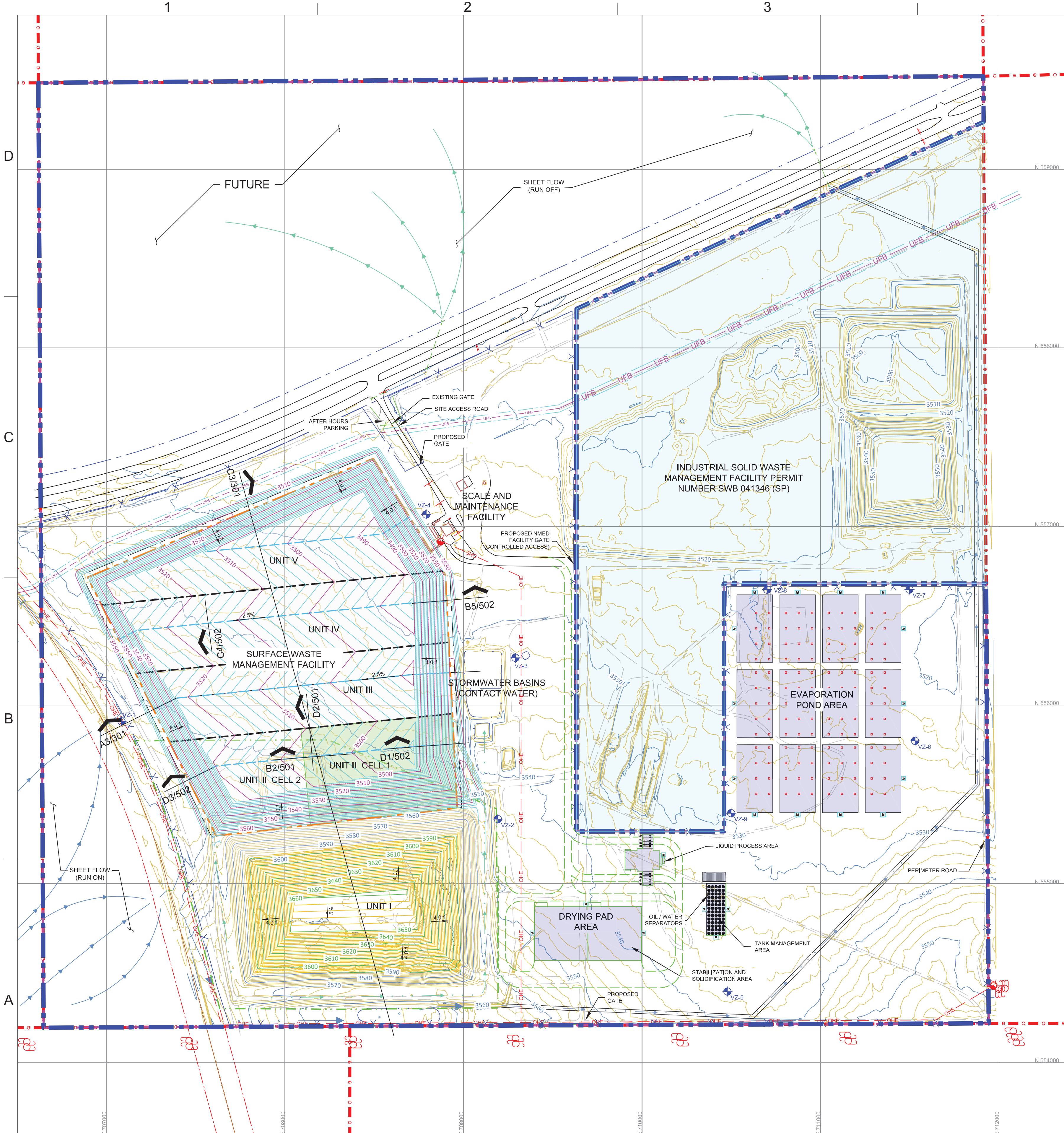
EXISTING PERMIT - COMPLETION GRADING PLAN

C-103



LEGEND

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- EXISTING CULVERT
- H2S MONITORING LOCATION
- VZ-4 PROPOSED VADOSE WELL LOCATION
- POWER POLE
- D3/502 CROSS-SECTION LOCATION
- N 5580000 E 708000 SITE GRID

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Charles W. Fiedler, P.E.

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LEA LAND LLC SURFACE WASTE MANAGEMENT FACILITY

PERMIT APPLICATION

LEA COUNTY, NEW MEXICO

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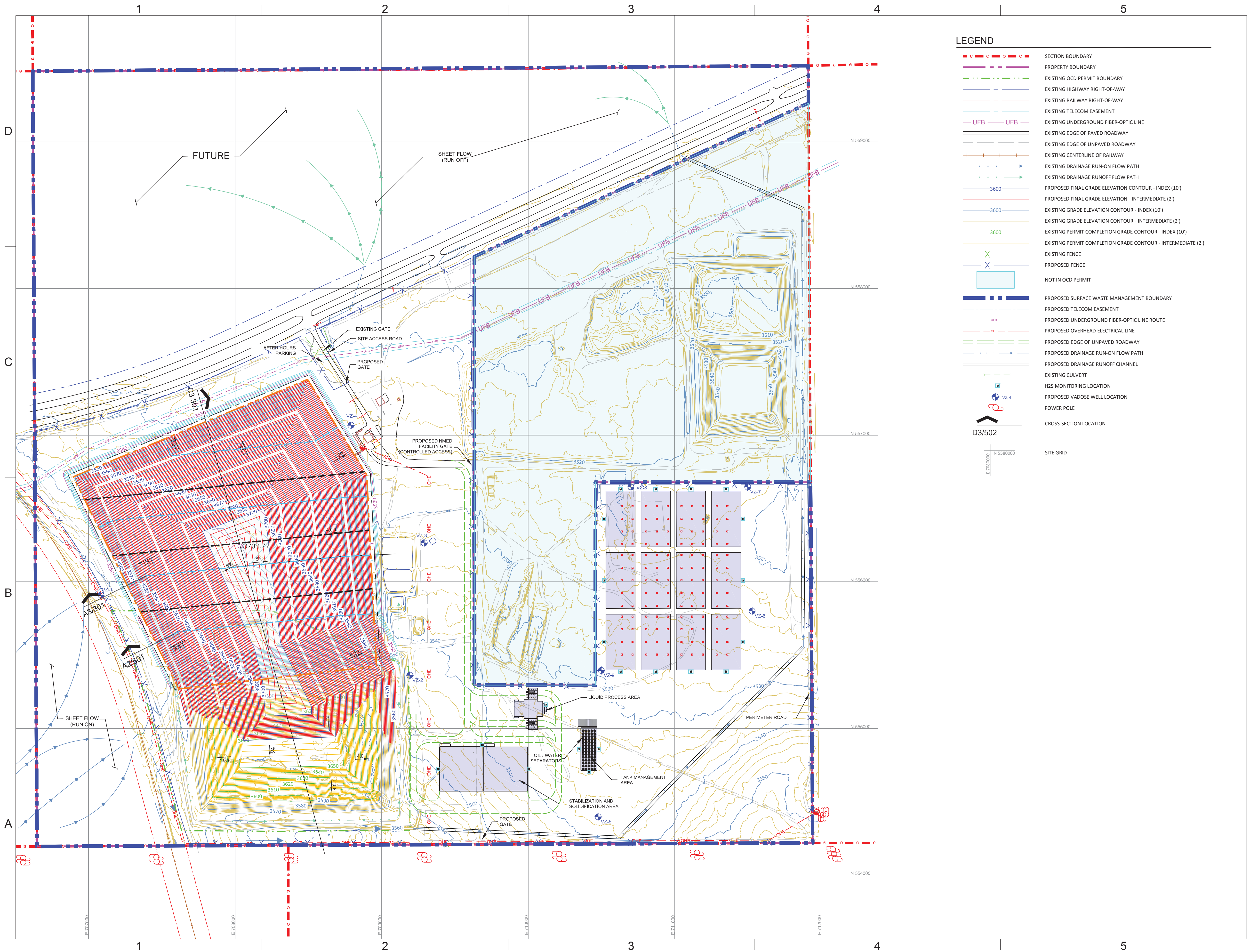
LANDFILL BASE GRADING PLAN

C-104

FILE NAME: X:\2018\0416.18\02_DSGN\02_DWG\050_CIVIL\03_SHEETS\C-104-0416-18 REVISED.dwg LAYOUT NAME: C-104 PRINTED: Wednesday, June 26, 2019 - 10:48am USER: dmill



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LEA LAND LLC SURFACE WASTE MANAGEMENT FACILITY

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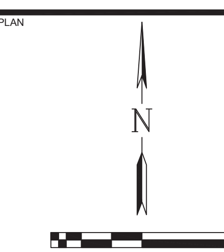
LEA COUNTY, NEW MEXICO

KEY PLAN



NO	DATE	DESCRIPTION
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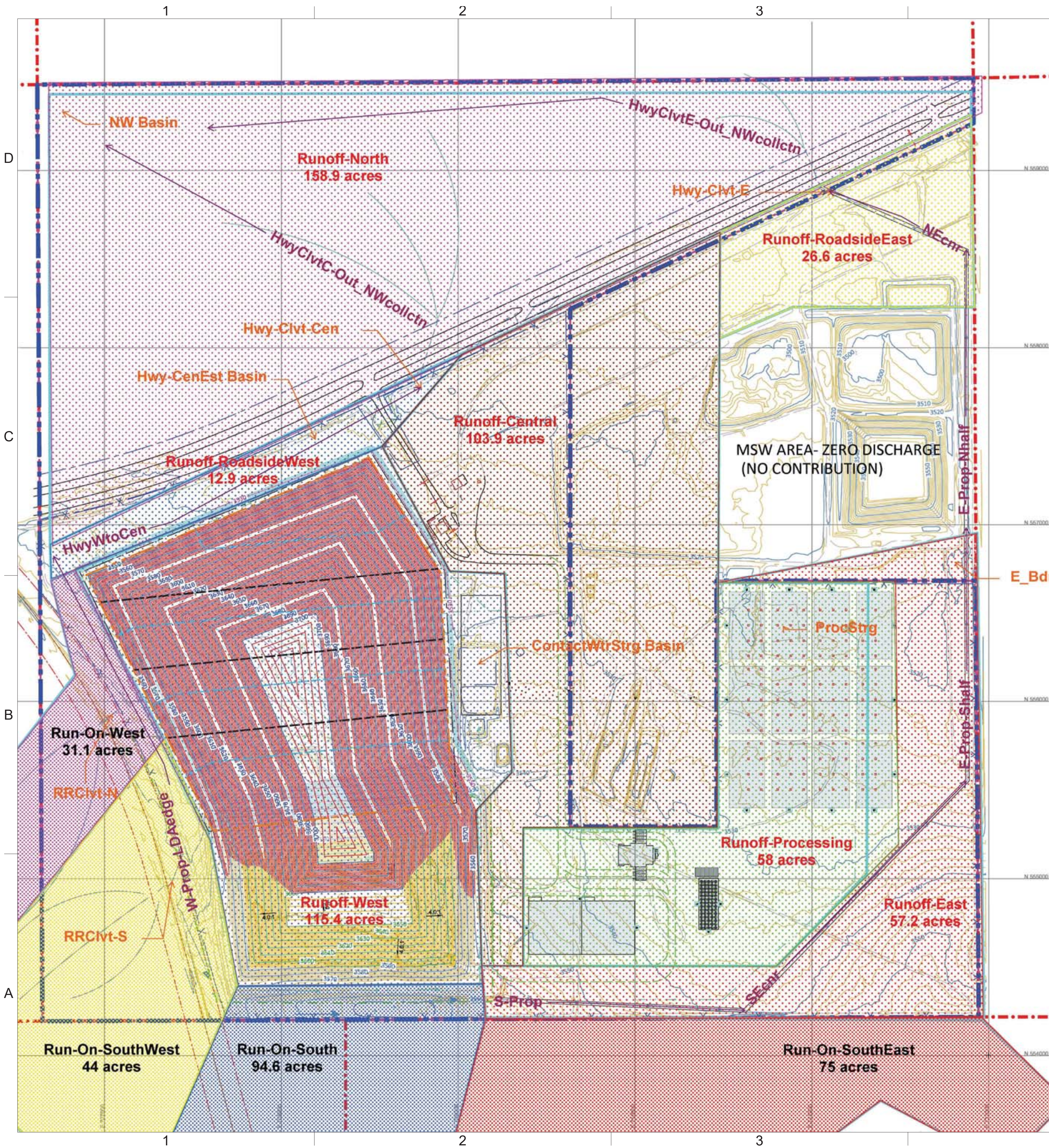
ISSUING OFFICE: RIO RANCHO PROJECT NO: 0416.18

LANDFILL FINAL GRADING PLAN

C-105



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- PROPERTY BOUNDARY
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- PROPOSED FINAL GRADE ELEVATION CONTOUR -INTERMEDIATE (2')
- PROPOSED SURFACE WASTE MANAGEMENT BOUNDARY
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- 3600 EXISTING PERMIT COMPLETION GRADE CONTOUR - INDEX (10')
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- PROPOSED NEW PERMIT UNIT BOUNDARY
- PROPOSED LEACHATE COLLECTION PIPE
- PROPOSED DRAINAGE CATCHMENT BOUNDARY
- N 5580000 / E 7080000 PROPOSED DRAINAGE CATCHMENT BOUNDARY

RUN-ON DRAINAGE

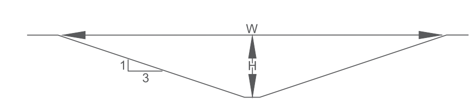
RUN-OFF DRAINAGE

Channel Flow Results and Sizing

Element ID	Length (ft)	(H) Height (ft)	(W) Width (ft)	Peak Flow (cfs)	Peak Flow Depth (ft)	Design Flow Capacity (cfs)
E-Prop-Nhalf	1,499.64	3.00	20.00	107.63	1.53	384.13
E-Prop-Shalf	1,284.15	3.00	20.00	86.72	1.36	384.13
HwyClvtC-Out_NWcollctn	2,194.11	2.00	32.00	33.26	0.43	465.74
HwyClvtE-Out_NWcollctn	3,882.48	2.00	32.00	164.32	1.11	465.74
HwyWtoCen	2,375.24	1.00	14.00	15.13	0.41	69.56
NEcnr	905.56	3.00	20.00	107.61	1.43	432.15
NWcollctn_NWbsn	5,342.82	2.00	34.00	336.84	1.38	620.76
SEcnr	1,816.82	3.00	20.00	86.73	1.36	384.13
S-Prop	1,421.18	3.00	20.00	65.42	1.16	384.13
W-Prop-LDAedge	863.84	1.00	10.00	7.54	0.36	43.90

Subbasin Summary

Subbasin ID	Area (ac)	Peak Runoff (cfs)	Total Runoff Volume (ac-in)
Processing	57.98	54.06	127.21
RoadsideEast	26.63	90.89	58.42
RoadsideW	12.87	21.99	28.23
Runoff-Central	103.88	183.00	227.91
Runoff-East	57.16	82.47	125.42
Runoff-North	158.86	200.79	348.55
Runoff-West	115.39	159.59	253.16
RunOn-South	94.60	65.68	207.56
RunOn-SouthEast	75.01	30.52	164.57
RunOn-SouthWest	43.95	69.02	96.43
RunOn-West	31.10	78.93	68.22



TYPICAL TRAPEZOIDAL CHANNEL CROSS-SECTION

GORDON ENVIRONMENTAL PSC

CHARLES WALTER FIEDLER NEW MEXICO 19731 06/26/2019 PROFESSIONAL ENGINEER

Charles W. Fiedler P.E.

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GORDON ENVIRONMENTAL | PSC

LEA LAND LLC SURFACE WASTE MANAGEMENT FACILITY

PERMIT APPLICATION

LEA COUNTY, NEW MEXICO

KEY PLAN

N

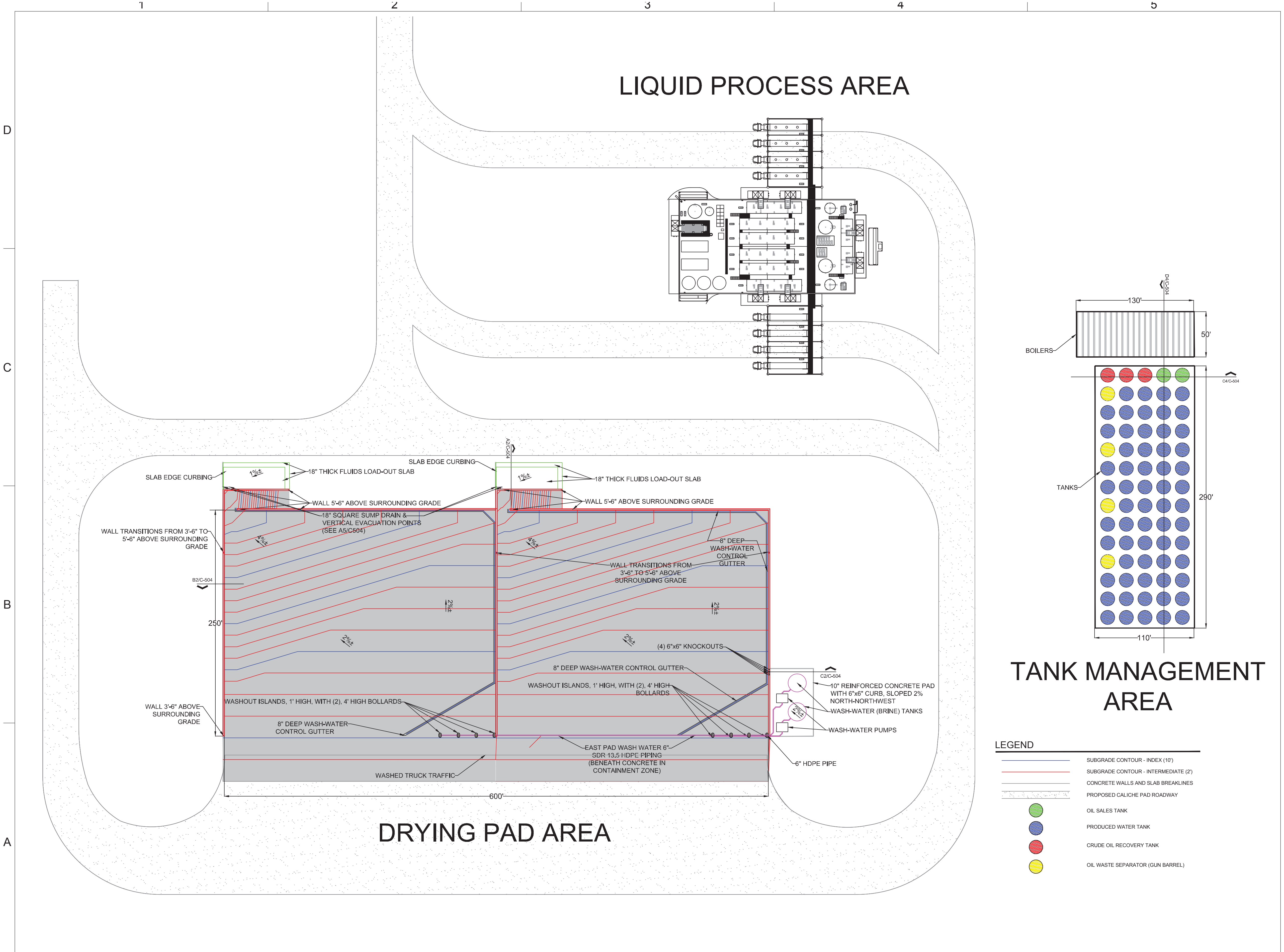
0 300' 600'

NO	DATE	DESCRIPTION
1	06/03/2019	REVISIONS PER OCD REVIEW
-	04/10/19	OCD PERMIT APPLICATION REVIEW

ISSUING OFFICE: RIO RANCHO PROJECT NO: 0416.18

LANDFILL COMPLETION DRAINAGE PLAN

C-106



FILE NAME: X:\2018\0416.18\02_DSGN\02_DWG\050_CIVIL\03_SHEETS\C-107-0416-18 REVISED.dwg LAYOUT NAME: C-107 PRINTED: Wednesday, June 26, 2019 - 11:24am USER: dmill

GORDON ENVIRONMENTAL PSC

CHARLES WALTER FIEDLER NEW MEXICO 19731 06/26/2019 PROFESSIONAL ENGINEER

P.E.

LEA LAND LLC SURFACE WASTE MANAGEMENT FACILITY

PERMIT APPLICATION

LEA COUNTY, NEW MEXICO

KEY PLAN

N

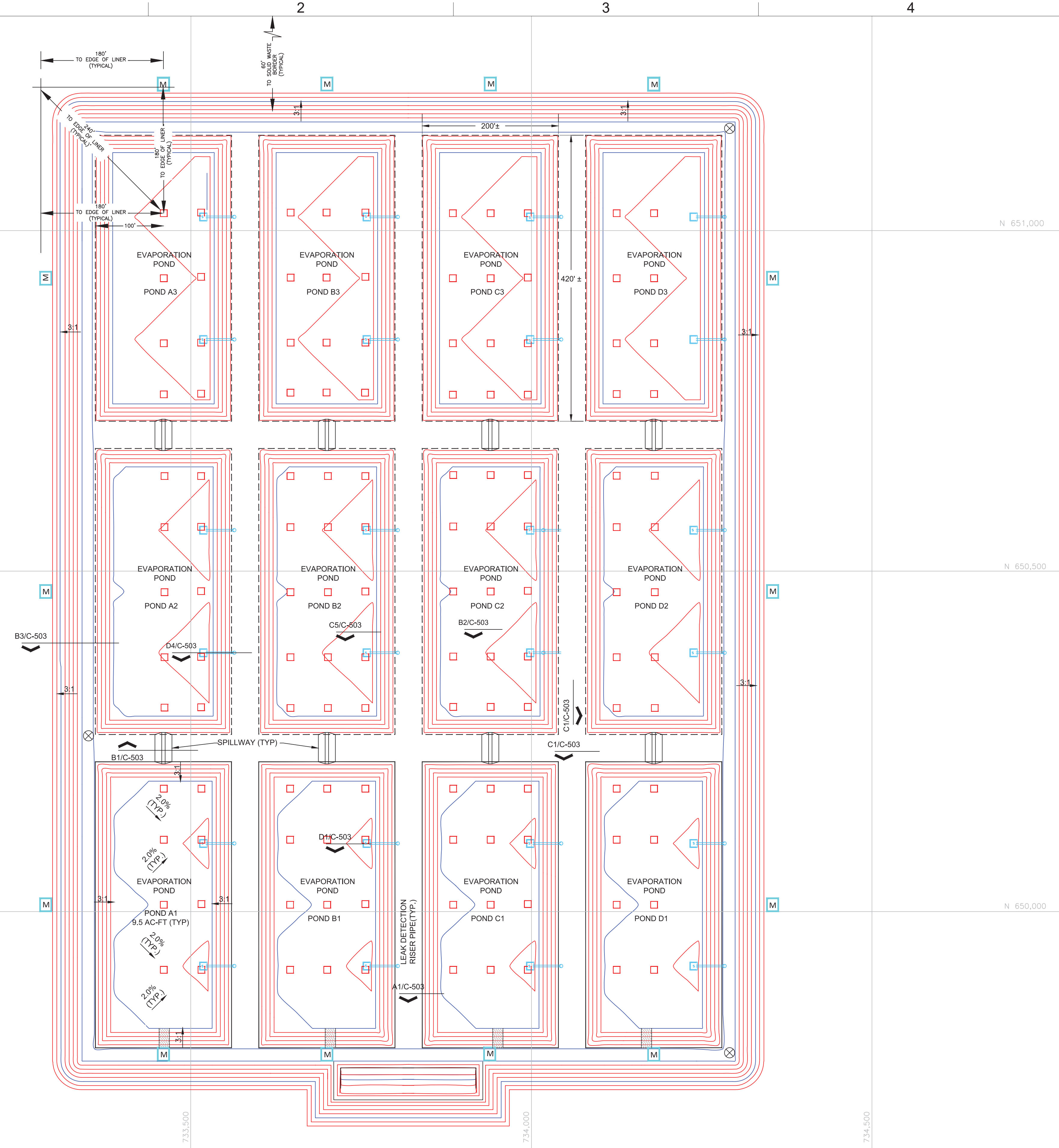
0 50' 100'

NO	DATE	DESCRIPTION
1	06/03/2019	REVISIONS PER OCD REVIEW
-	04/10/19	OCD PERMIT APPLICATION REVIEW

ISSUING OFFICE: RIO RANCHO PROJECT NO: 0416.18

PROCESS AREA LAYOUT

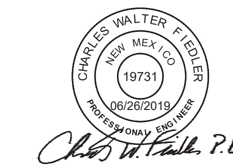
C-107



LEGEND

	10' DESIGN CONTOUR
	2' DESIGN CONTOUR
	TOP/TOE OF SLOPE
	PROPOSED FACILITY ACCESS ROAD
M	HYDROGEN SULFIDE MONITORING STATION
	EVAPORATORS
	PPE AND EMERGENCY EQUIPEMENT
	PUMP DISCHARGE PROTECTION (60 mil TEXTURED HDPE TO WATERLINE ELEVATION)
	LEAK DETECTION SUMP & RISER PIPE
B3/C-503	DETAIL NUMBER SHEET NUMBER
N 650,500 / E 738,000	SITE GRID

GORDON ENVIRONMENTAL PSC



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LEA LAND LLC SURFACE WASTE MANAGEMENT FACILITY

PERMIT APPLICATION

LEA COUNTY, NEW MEXICO

KEY PLAN

N

0 80' 160'

NO	DATE	DESCRIPTION
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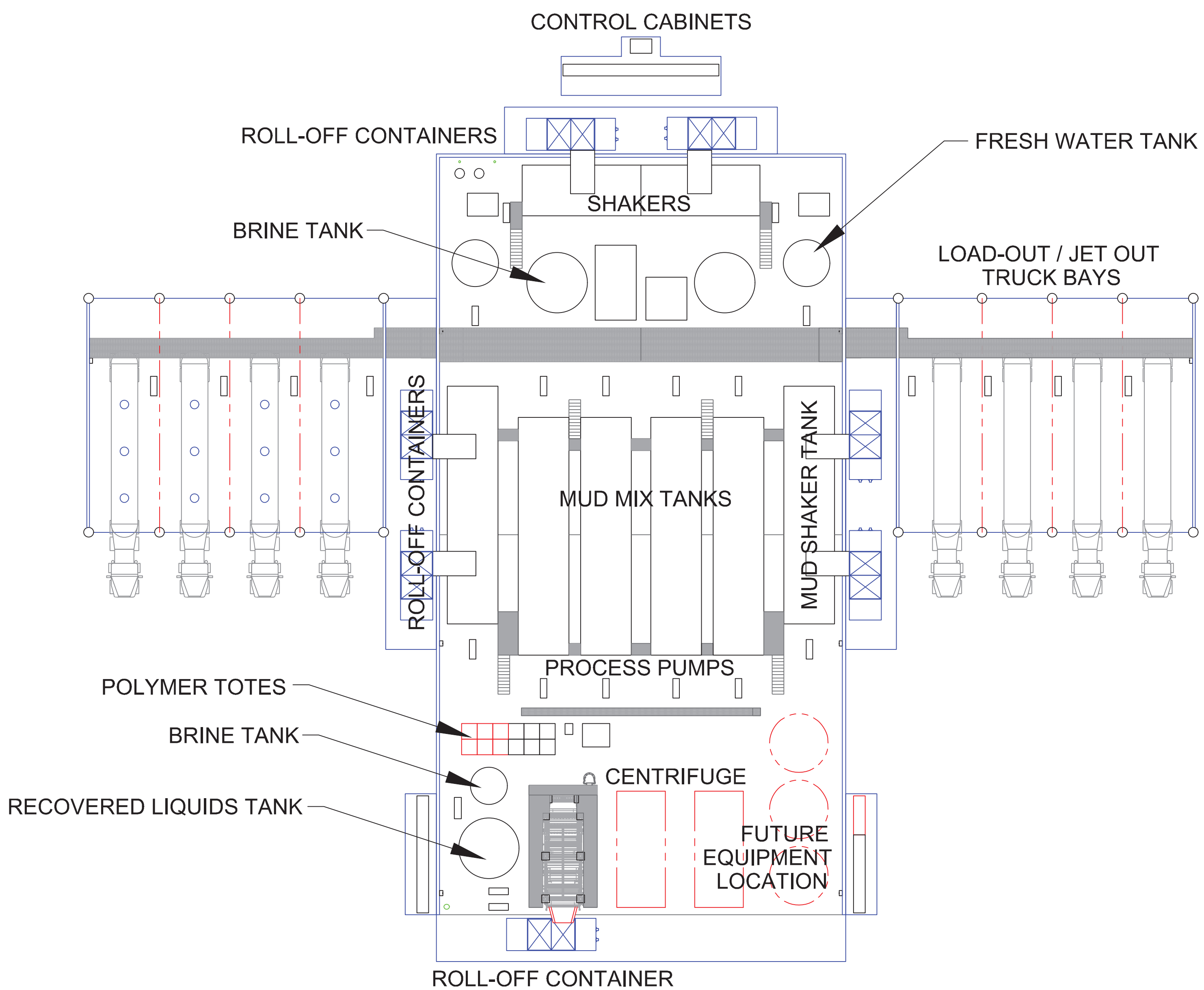
-	04/10/19	OCD PERMIT APPLICATION REVIEW

ISSUING OFFICE: RIO RANCHO PROJECT NO: 0416.18

EVAPORATION POND LAYOUT

C-108

FILE NAME: X:\2018\0416.18\02_DSGN\02_DWGS\050_CIVIL\03_SHEETS\C-108-0416-18-REVISED 7-23-2019.dwg LAYOUT NAME: C-108 PRINTED: Tuesday, July 23, 2019 - 8:48am USER: dmill



GORDON ENVIRONMENTAL PSC

CHARLES WALTER FIEDLER
NEW MEXICO
19731
06/26/2019
PROFESSIONAL ENGINEER

P.E.

LEA LAND LLC SURFACE WASTE MANAGEMENT FACILITY

PERMIT APPLICATION

LEA COUNTY, NEW MEXICO

KEY PLAN

N

0 20' 40'

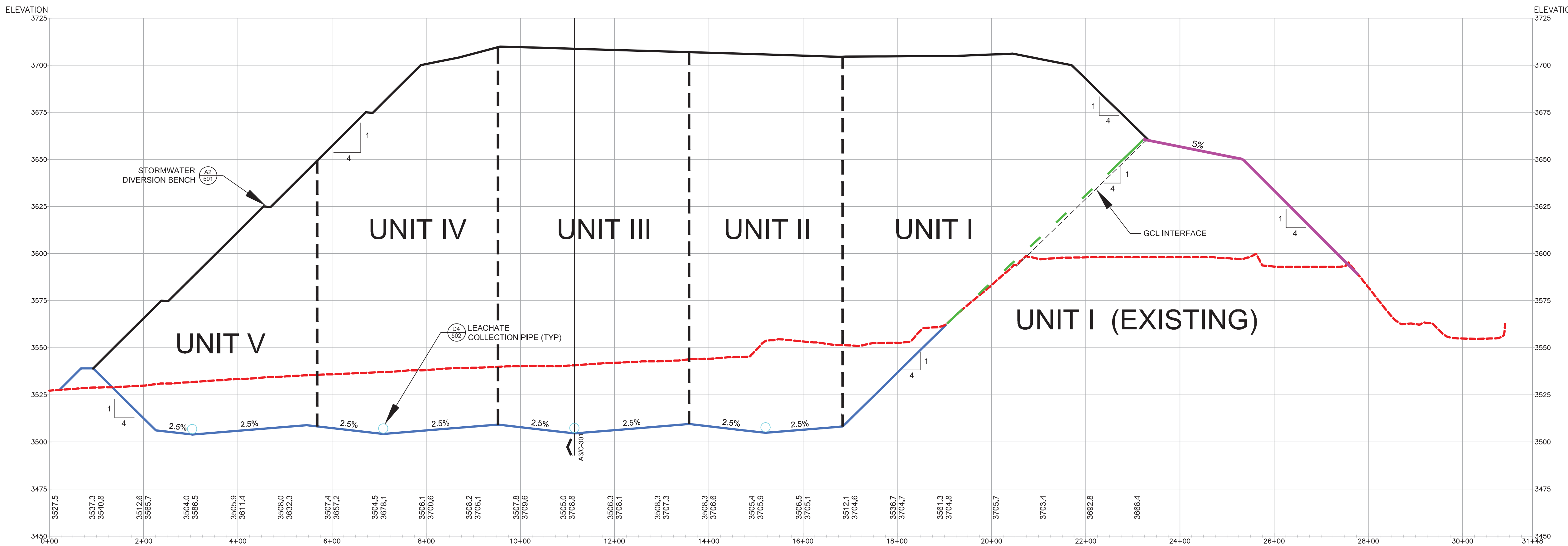
NO	DATE	DESCRIPTION
1	06/03/2019	REVISIONS PER OCD REVIEW
-	04/10/19	OCD PERMIT APPLICATION REVIEW

ISSUING OFFICE: RIO RANCHO PROJECT NO: 0416.18

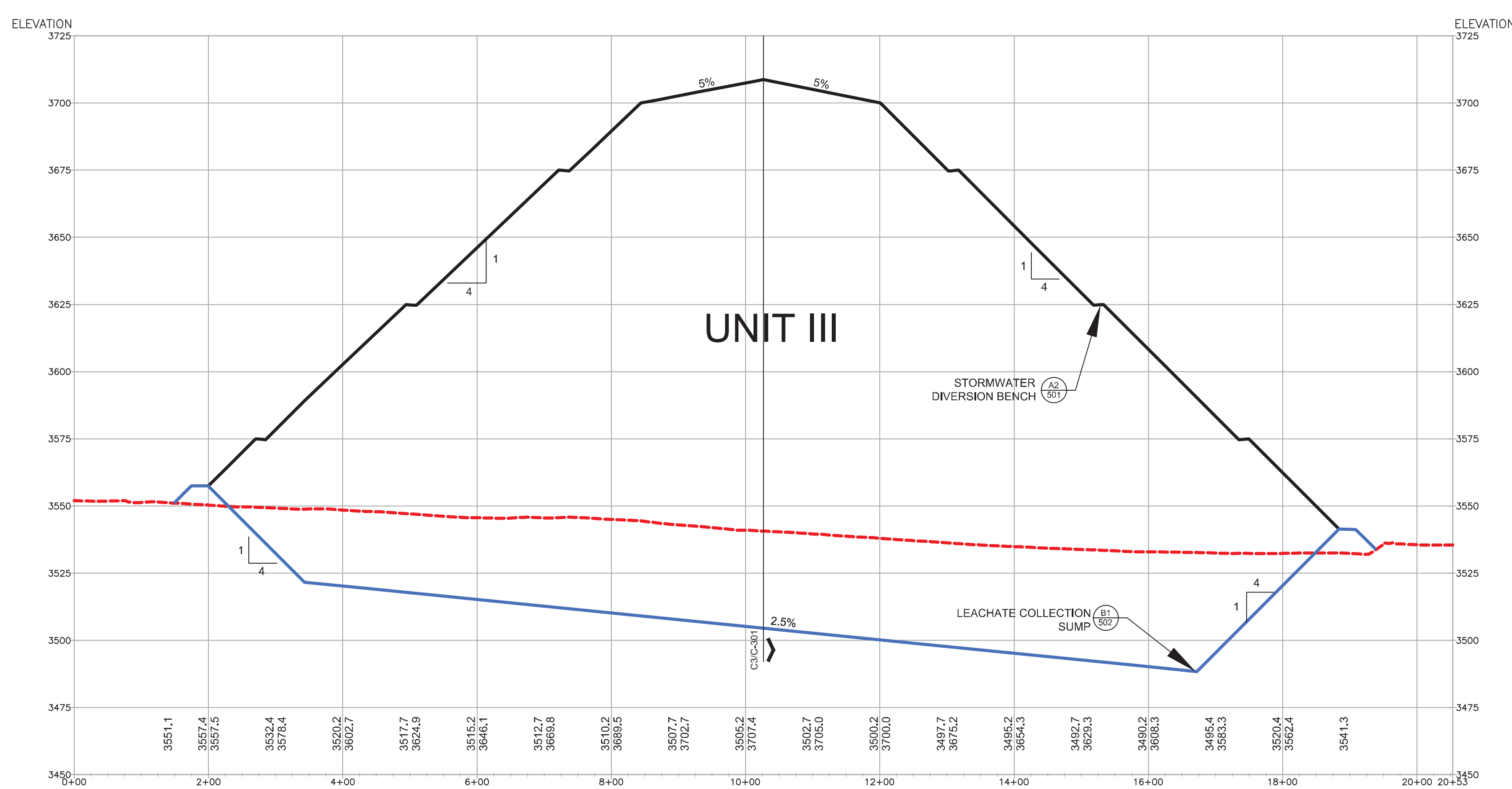
LIQUID PROCESS AREA EQUIPMENT LAYOUT

C-109

FILE NAME: X:\2018\0416.18\02_DSGN\02_DWG\050_CIVIL\03_SHEETS\C-109-0416-18 REVISED.dwg LAYOUT NAME: C-109 PRINTED: Thursday, July 18, 2019 - 1:30pm USER: dmill



C3 NORTH-SOUTH SECTION ALONG CREST
1"=120' (Horiz.) 4:1 VERTICAL EXAGGERATION



A3 WEST-EAST SECTION ALONG DEEPEST LEACHATE LINE
1"=120' (Horiz.) 4:1 VERTICAL EXAGGERATION

LEGENDTITLE

- SECTION VIEW – EXISTING GRADES
- SECTION VIEW – EXISTING PERMIT FINAL GRADES
- SECTION VIEW – PROPOSED PERMIT BASE GRADES
- SECTION VIEW – PROPOSED PERMIT FINAL GRADES
- SECTION VIEW – UNIT I CLOSURE GRADES

VERTICAL SCALE 1"=30'

VERTICAL EXAGGERATION 4:1

HORIZONTAL SCALE 1"=120'

GORDON ENVIRONMENTAL PSC

CHARLES WALTER FIEDLER, NEW MEXICO, 19731, PROFESSIONAL ENGINEER, 06/26/2019

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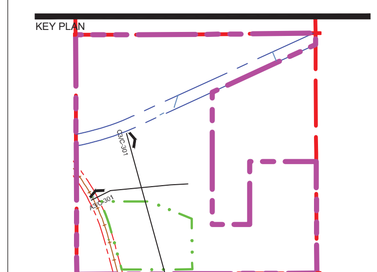
LEA LAND LLC SURFACE WASTE MANAGEMENT FACILITY

PERMIT APPLICATION

LEA COUNTY, NEW MEXICO

KEY PLAN



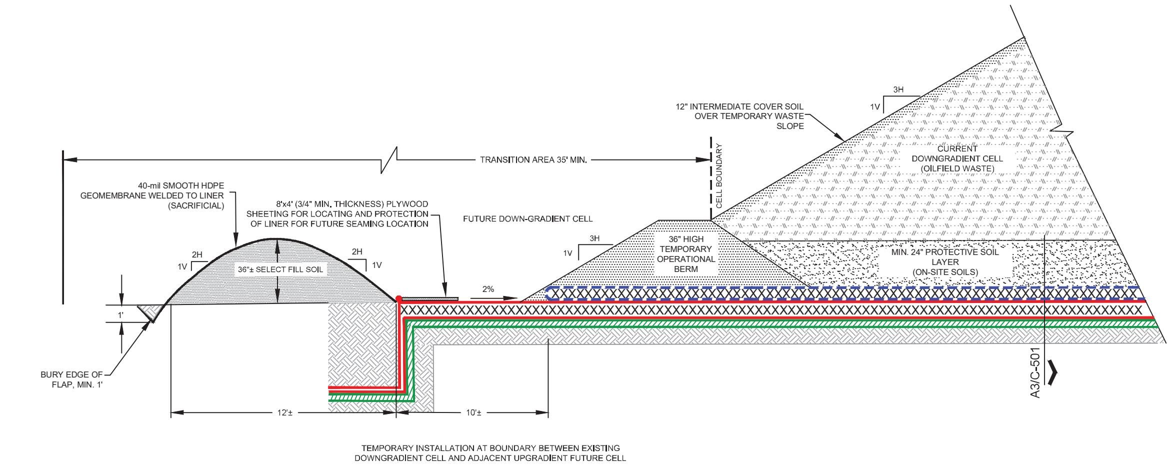
NO	DATE	DESCRIPTION
1	06/03/2019	REVISIONS PER OCD REVIEW
-	04/10/19	OCD PERMIT APPLICATION REVIEW

ISSUING OFFICE: RIO RANCHO PROJECT NO: 0416.18

LANDFILL CROSS-SECTIONS

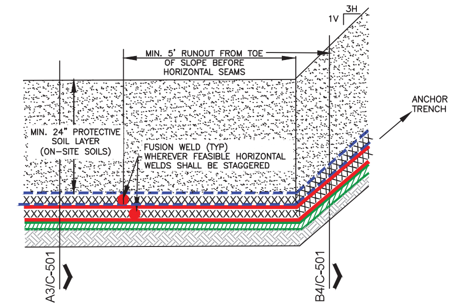
C-301

FILE NAME: X:\2018\0416.18\02_DSGN\02_DWG\050_CIVIL\03_SHEETS\C-301-0416-18 REVISED 7-18-2019.dwg LAYOUT NAME: C-301 PRINTED: Friday, July 19, 2019 - 7:26am USER: dmill

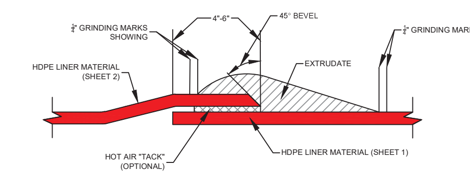


TEMPORARY INSTALLATION AT BOUNDARY BETWEEN EXISTING DOWNGRADIENT CELL AND ADJACENT UPGRADIENT FUTURE CELL

D2 TYPICAL CELL TRANSITION AREA
NOT TO SCALE — SECTION VIEW

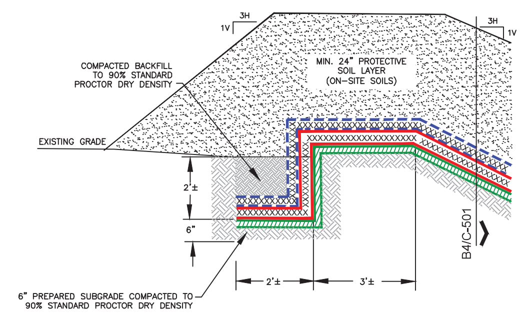


D3 LINER RUNOUT ON LANDFILL FLOOR
NOT TO SCALE — SECTION VIEW

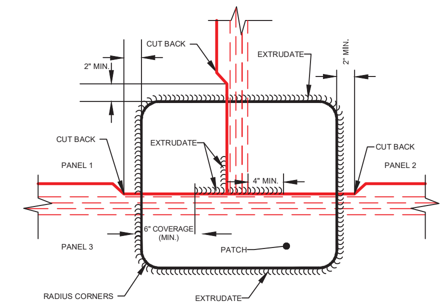


NOTES:
1. ALL EXTRUSION WELDS SHALL BE VACUUM TESTED

D5 TYPICAL EXTRUSION WELD
NOT TO SCALE — SECTION VIEW

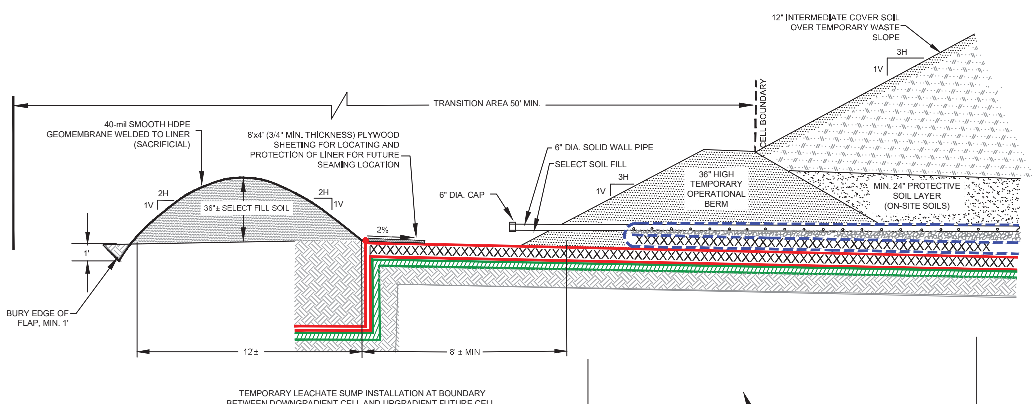


C3 PERMANENT ANCHOR TRENCH
NOT TO SCALE — SECTION VIEW



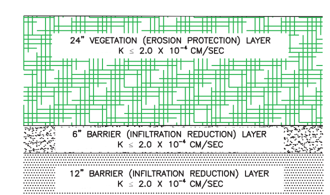
NOTES:
1. ALL EXTRUSION WELDS SHALL BE VACUUM TESTED
2. ALL DOUBLE TRACK FUSION WELDS SHALL BE AIR PRESSURE TESTED
3. PANELS SHALL BE OVERLAPPED IN NUMERICAL ORDER - HIGHEST ON TOP

C5 TYPICAL "T" AT BUTT SEAM
NOT TO SCALE — SECTION VIEW

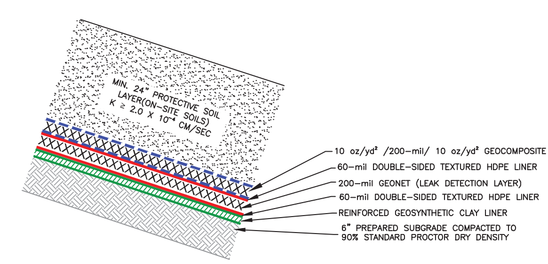


TEMPORARY LEACHATE SUMP INSTALLATION AT BOUNDARY BETWEEN DOWNGRADIENT CELL AND UPGRADIENT FUTURE CELL.

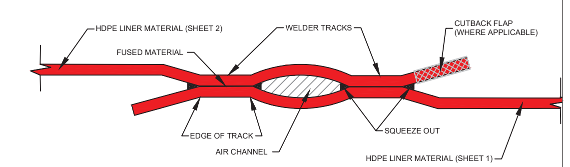
B2 TEMPORARY LEACHATE COLLECTION DETAIL
NOT TO SCALE — SECTION VIEW



B3 FINAL CROWN & SIDESLOPE FINAL COVER DETAIL
SECTION VIEW

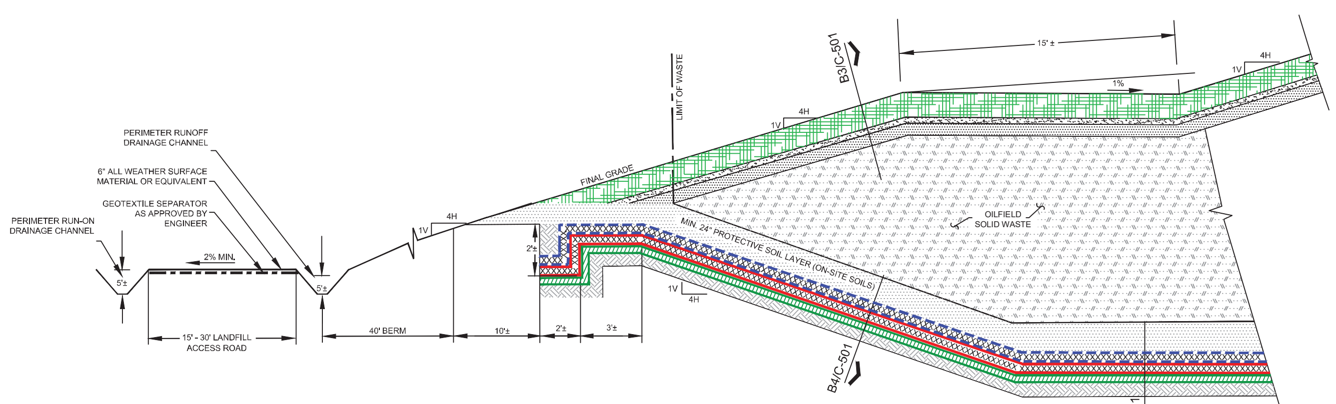


B4 SIDEWALL LINER SYSTEM
NOT TO SCALE — SECTION VIEW

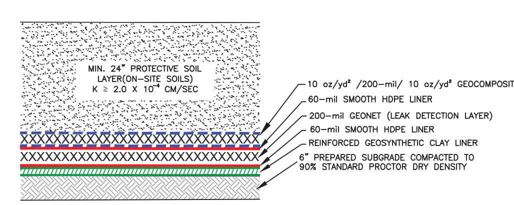


NOTES:
1. BOTH AREAS OF FUSED MATERIAL SHOULD BE VOID OF ANY SEAM LINES
2. EDGE OF TRACKS NOT TO CUT THE LINER
3. AIR CHANNEL SHOULD BE CLEAR
4. BOTH WELDER TRACKS SHALL BE EQUAL WIDTHS
5. SQUEEZE OUT SHOULD BE JUST BARELY VISIBLE IN ALL FOUR (4) LOCATIONS WHEN VIEWED IN THE TEST CROSS-SECTION
6. ALL DOUBLE TRACK FUSION WELDS SHALL BE AIR PRESSURE TESTED

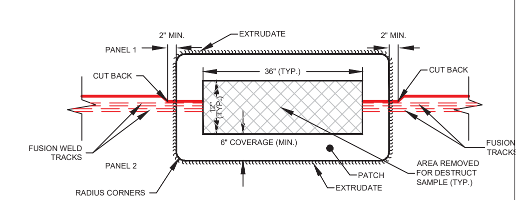
B5 TYPICAL FUSION WELD
NOT TO SCALE — SECTION VIEW



A2 FINAL COVER TIE-IN
NOT TO SCALE — SECTION VIEW



A3 FLOOR LINER SYSTEM
NOT TO SCALE — SECTION VIEW



NOTES:
1. ALL EXTRUSION WELDS SHALL BE VACUUM TESTED
2. ALL DOUBLE TRACK FUSION WELDS SHALL BE AIR PRESSURE TESTED
3. PANELS SHALL BE OVERLAPPED IN NUMERICAL ORDER - HIGHEST ON TOP

A5 TYPICAL HDPE LINER SEAM PATCH
NOT TO SCALE — SECTION VIEW

GORDON ENVIRONMENTAL | PSC

CHARLES WALTER FIEDLER — NEW MEXICO — 19731 — 06/26/2019 — PROFESSIONAL ENGINEER

P.E.

LEA LAND LLC SURFACE WASTE MANAGEMENT FACILITY

PERMIT APPLICATION

LEA COUNTY, NEW MEXICO

KEY PLAN

NO	DATE	DESCRIPTION
1	06/03/2019	REVISIONS PER OCD REVIEW
-	04/10/19	OCD PERMIT APPLICATION REVIEW

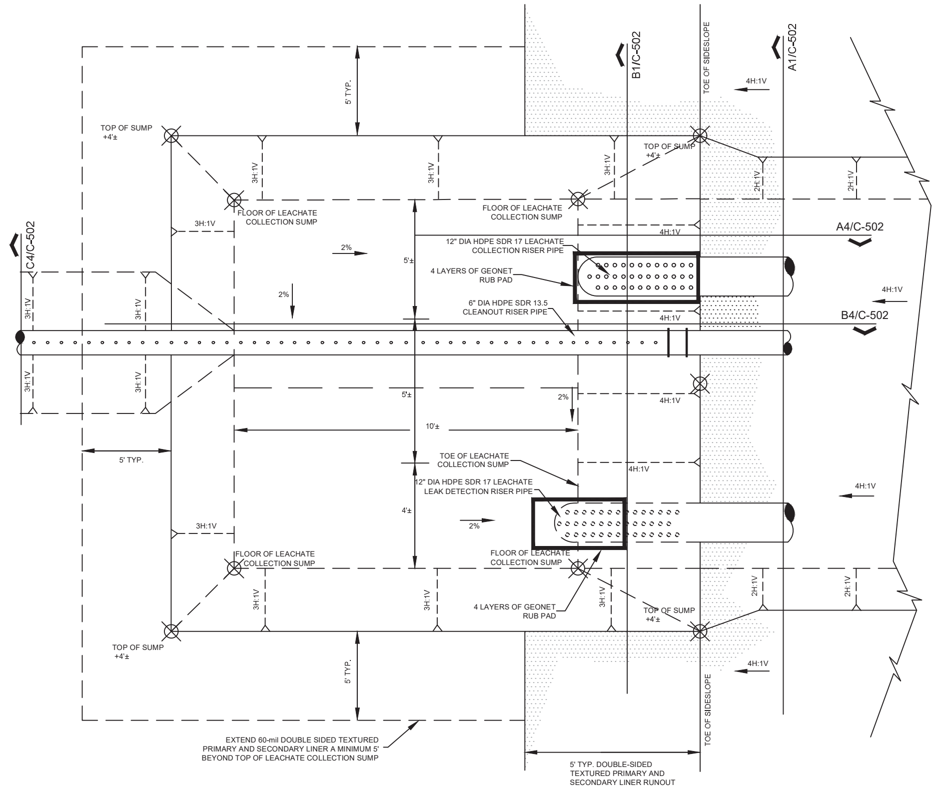
ISSUING OFFICE: RIO RANCHO   PROJECT NO: 0416.18

LANDFILL LINER SYSTEM AND FINAL COVER DETAILS

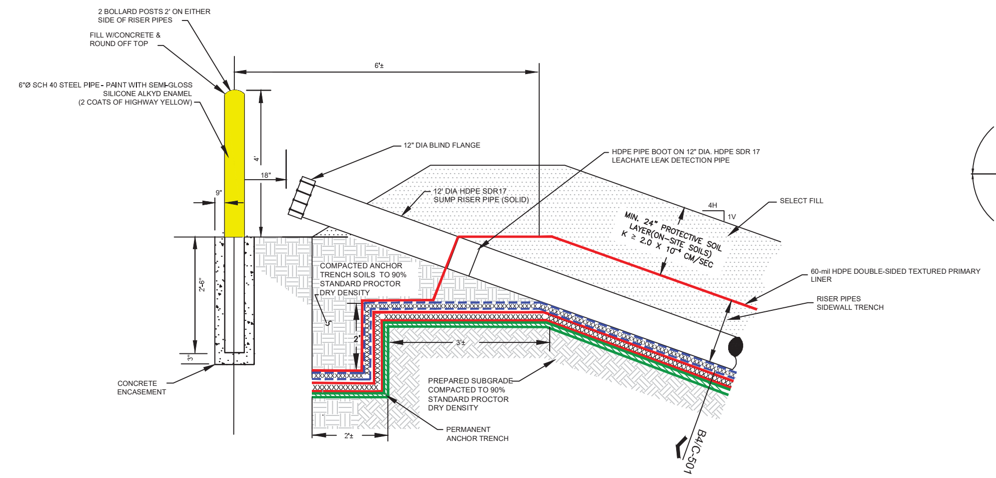
C-501

FILE NAME: X:\2018\0416.18\02_DSGN\02_DWG\050_CIVIL\03_SHEETS\C-501-0416-18 REVISED 7-18-2019.dwg LAYOUT NAME: C-501 PRINTED: Friday, July 19, 2019 - 9:13am USER: dmill

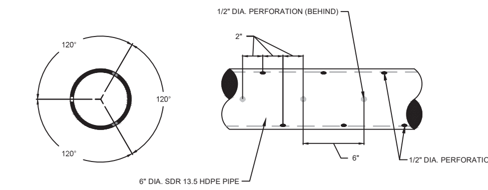
FILE NAME: X:\2018\0416.18\02_DSGN\02_DWG\050_CIVIL\03_SHEETS\C-502 -0416-18 REVISED.dwg LAYOUT NAME: C-502 PRINTED: Thursday, July 18, 2019 - 1:03pm USER: dmill



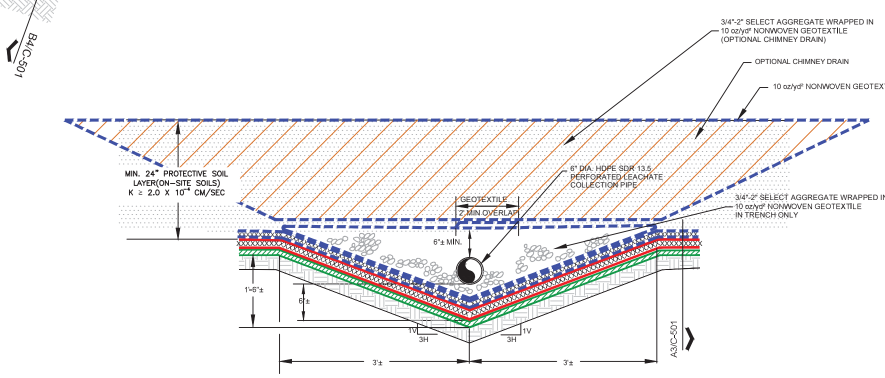
D1 LEACHATE COLLECTION SUMP
NOT TO SCALE — PLAN VIEW



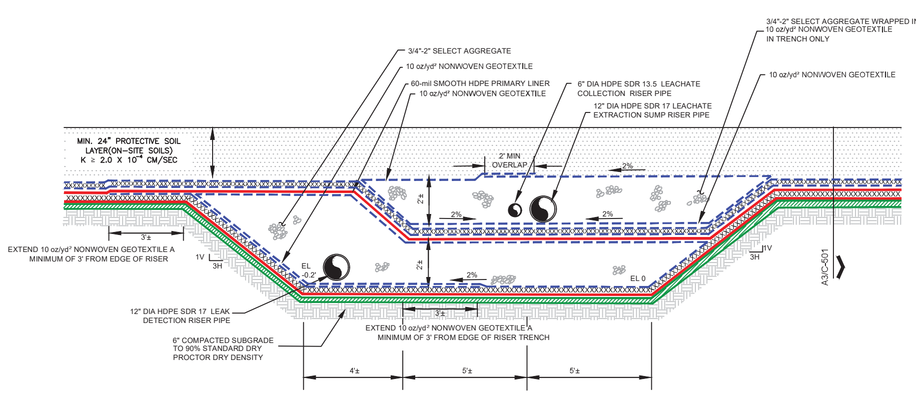
D3 RISER PIPE TERMINATION
NOT TO SCALE — SECTION VIEW



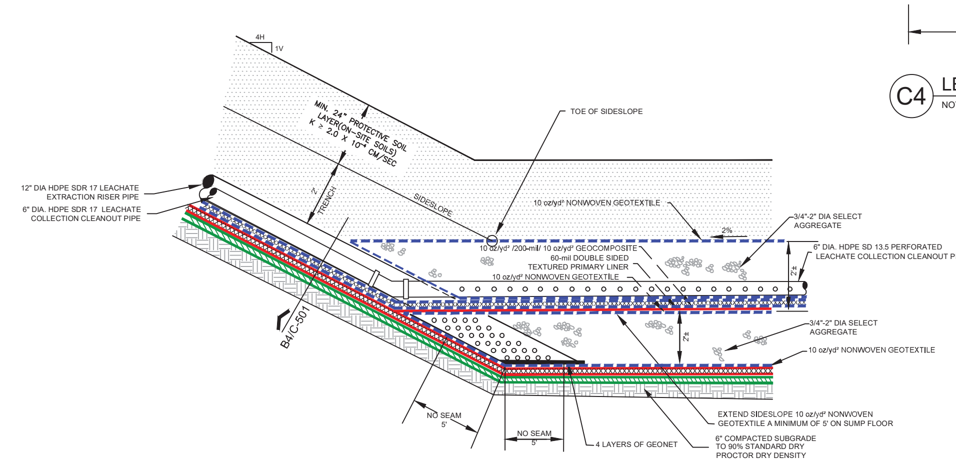
D4 PERFORATED LEACHATE COLLECTION PIPE
NOT TO SCALE — SECTION VIEW



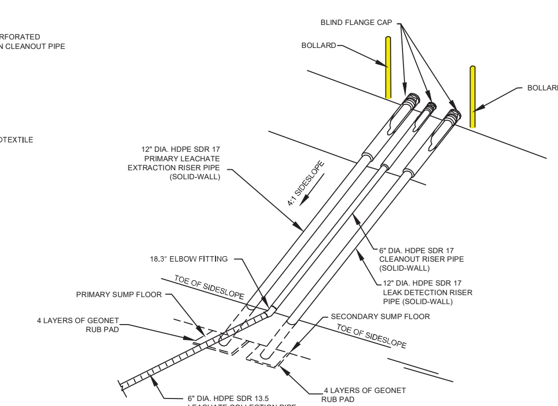
C4 LEACHATE COLLECTION TRENCH AND PIPE
NOT TO SCALE — SECTION VIEW



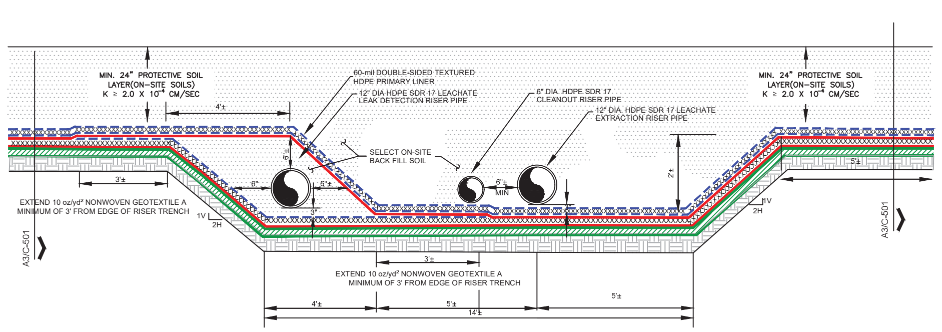
B1 LEACHATE COLLECTION SUMP
NOT TO SCALE — SECTION VIEW



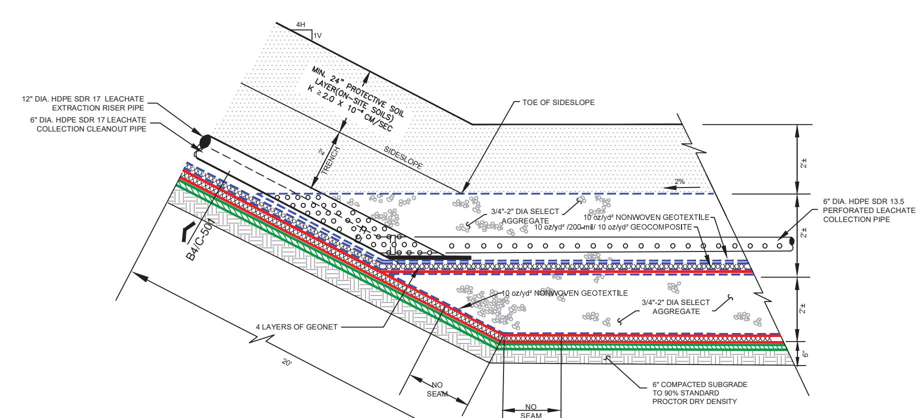
B4 LEAK DETECTION RISER PIPE
NOT TO SCALE — SECTION VIEW



B5 SUMP CLEANOUT/RISER CONFIGURATION SCHEMATIC
NOT TO SCALE — SECTION VIEW



A1 LEACHATE COLLECTION SUMP RISER TRENCH
NOT TO SCALE — SECTION VIEW



A4 LEACHATE EXTRACTION RISER PIPE
NOT TO SCALE — SECTION VIEW

GORDON ENVIRONMENTAL PSC

CHARLES WALTER FIEDLER
NEW MEXICO
19731
06/26/2019
PROFESSIONAL ENGINEER
P.E.

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LEA LAND LLC SURFACE WASTE MANAGEMENT FACILITY

PERMIT APPLICATION

LEA COUNTY, NEW MEXICO

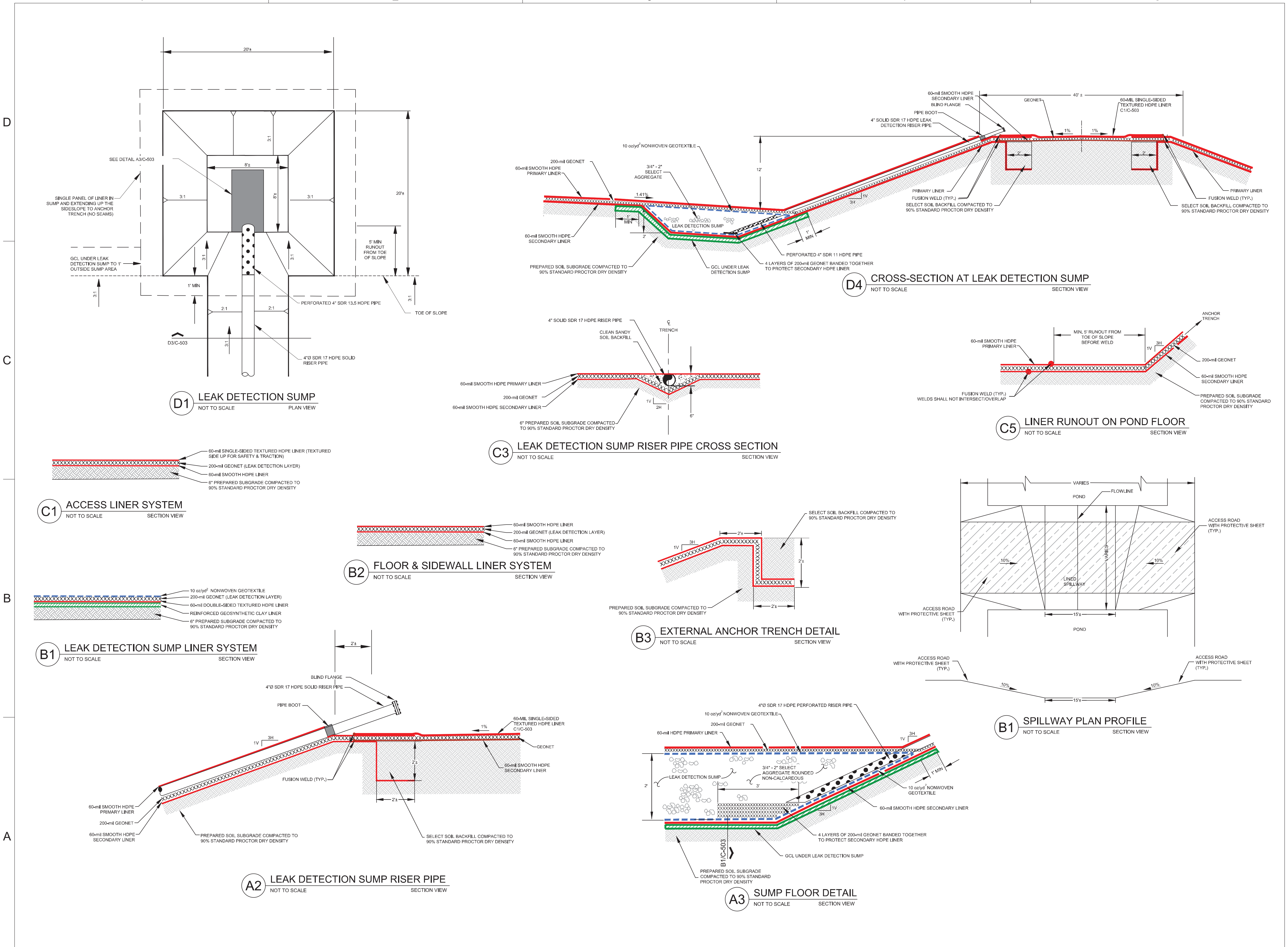
KEY PLAN

NO	DATE	DESCRIPTION
1	06/03/2019	REVISIONS PER OCD REVIEW
-	04/10/19	OCD PERMIT APPLICATION REVIEW

ISSUING OFFICE: RIO RANCHO — PROJECT NO: 0416.18

LEACHATE COLLECTION SYSTEM DETAILS

C-502



FILE NAME: X:\2018\0416.18\02_DSGN\02_DWG\050_CIVIL\03_SHEETS\C-503-0416-18 REVISED.dwg LAYOUT NAME: C-503 PRINTED: Thursday, July 18, 2019 - 1:41pm USER: dmill

GORDON ENVIRONMENTAL PSC

CHARLES WALTER FIEDLER NEW MEXICO 19731 06/26/2019 PROFESSIONAL ENGINEER

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LEA LAND LLC SURFACE WASTE MANAGEMENT FACILITY

PERMIT APPLICATION

LEA COUNTY, NEW MEXICO

KEY PLAN

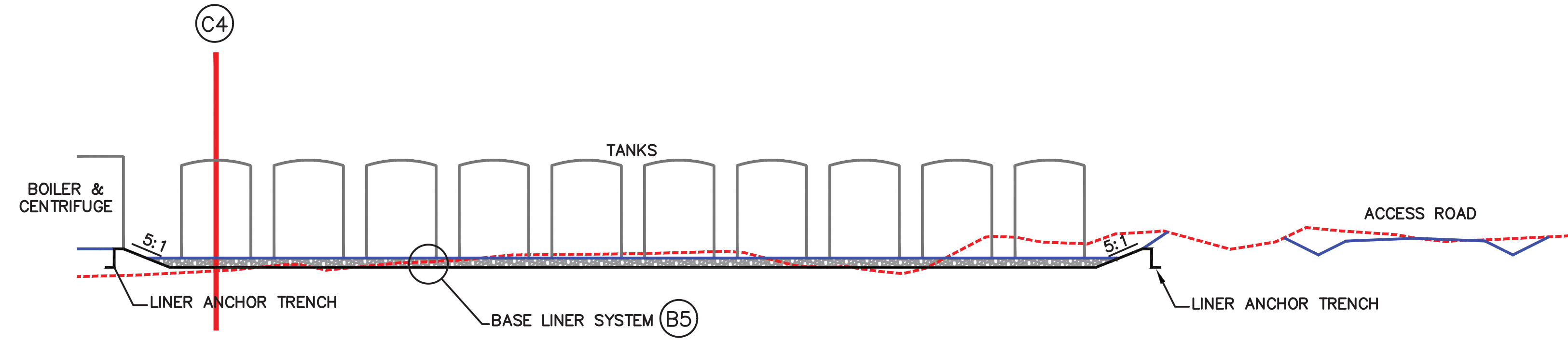
NO	DATE	DESCRIPTION
1	06/03/2019	REVISIONS PER OCD REVIEW
-	04/10/19	OCD PERMIT APPLICATION REVIEW

ISSUING OFFICE: RIO RANCHO PROJECT NO: 0416.18

EVAPORATION PONDS DETAILS

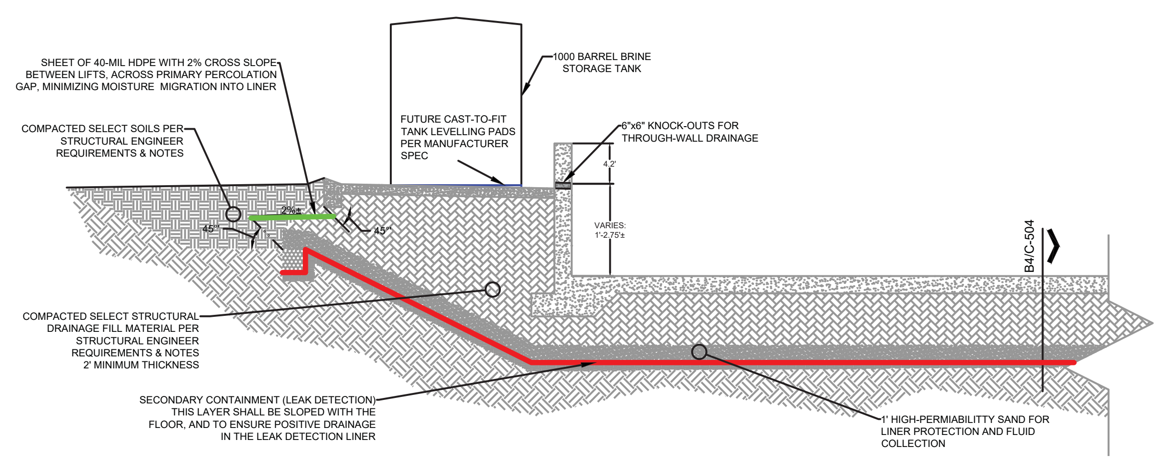
C-503

D

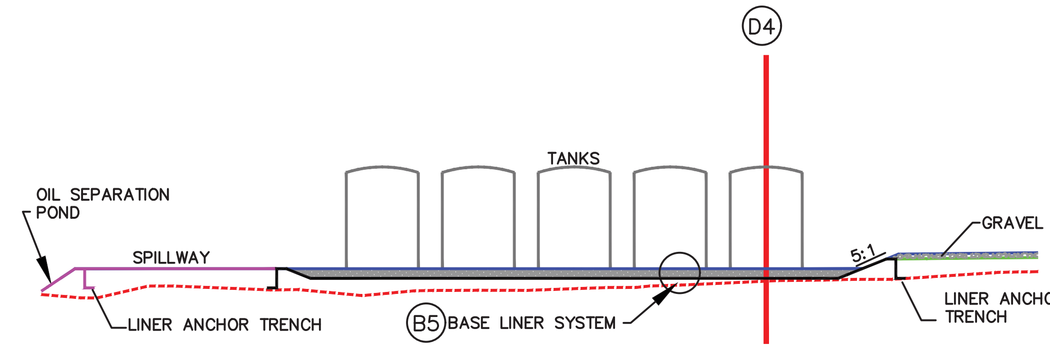


D4 CROSS SECTION J-J'

C

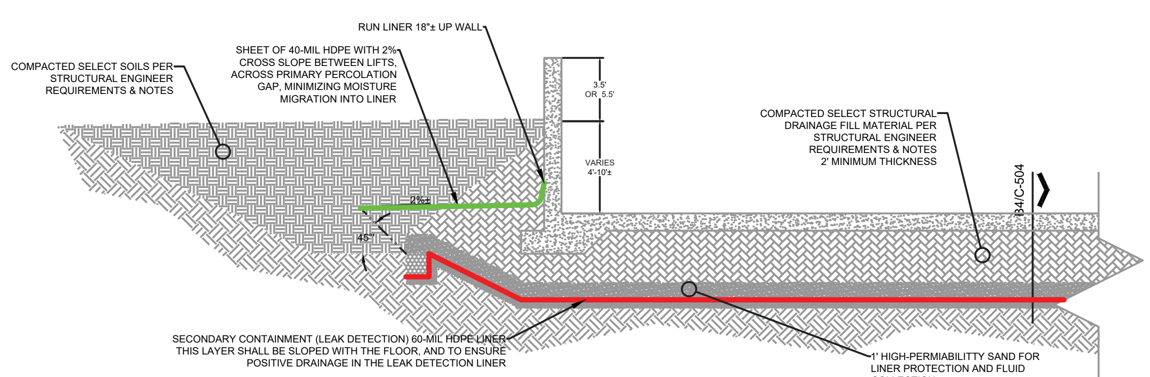


C2 TYPICAL EQUIPMENT PAD, LOW WALL LINER SURROUND & SUBGRADE
NOT TO SCALE SECTION VIEW

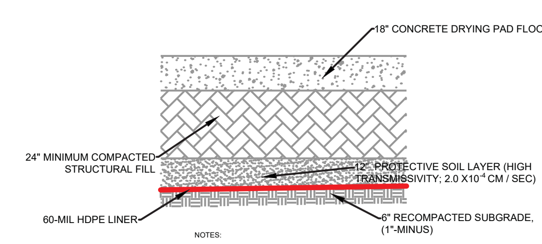


C4 CROSS SECTION L-L'

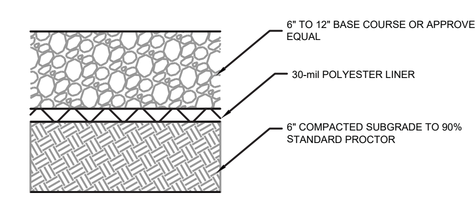
B



B2 TYPICAL WEST PUSH WALL LINER SURROUND & SUBGRADE
NOT TO SCALE SECTION VIEW

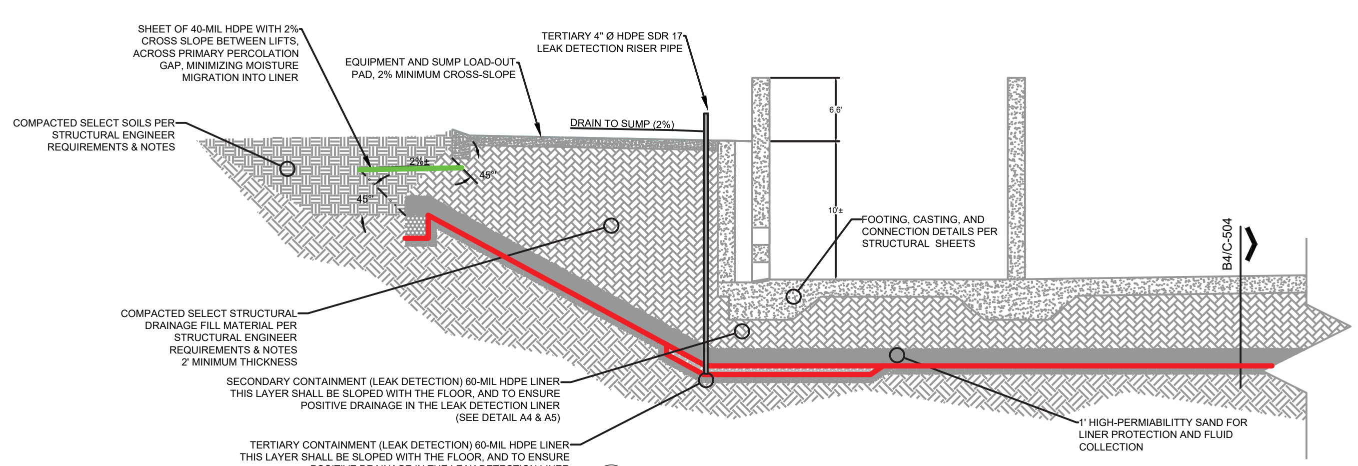


B4 CONCRETE SLAB SECTION
NOT TO SCALE SECTION VIEW

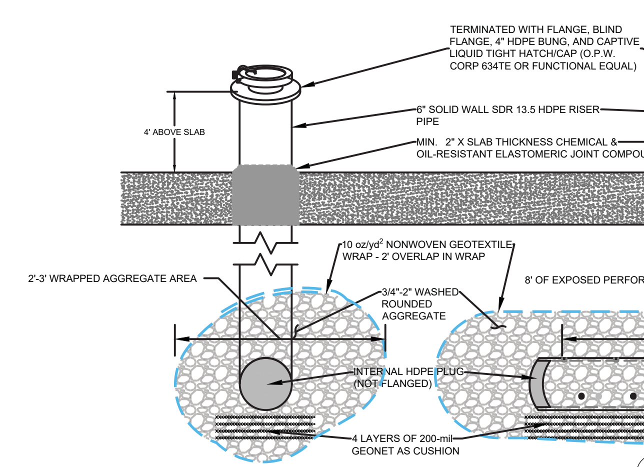


B5 BASE LINER SYSTEM
NOT TO SCALE SECTION VIEW

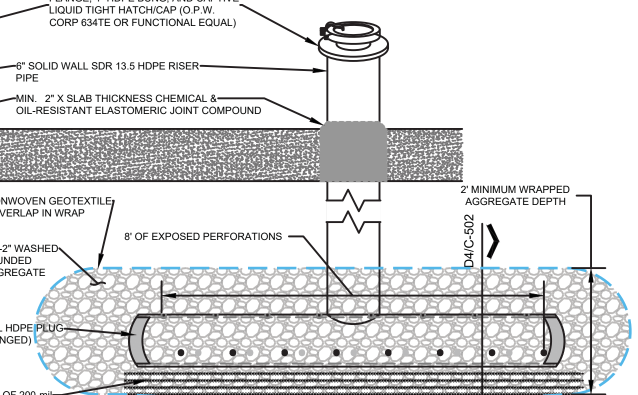
A



A2 TYPICAL EQUIPMENT PAD, SUMP WALL LINER SURROUND & SUBGRADE
NOT TO SCALE SECTION VIEW



A4 LEAK DETECTION RISER PIPE
NOT TO SCALE SECTION VIEW



A5 LEAK DETECTION RISER PIPE
NOT TO SCALE ELEVATION VIEW

FILE NAME: X:\2018\0416.18\02_DSGN\02_DWG\050_CIVIL\03_SHEETS\C-504-0416-18 REVISED 7-22-2019.dwg LAYOUT NAME: C-504 PRINTED: Friday, July 26, 2019 - 9:04am USER: dmill

GORDON ENVIRONMENTAL PSC

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06/26/2019
PROFESSIONAL ENGINEER
P.E.

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LEA LAND LLC SURFACE WASTE MANAGEMENT FACILITY

PERMIT APPLICATION

LEA COUNTY, NEW MEXICO

KEY PLAN

NO	DATE	DESCRIPTION
1	06/03/2019	REVISIONS PER OCD REVIEW
-	04/10/19	OCD PERMIT APPLICATION REVIEW

ISSUING OFFICE: RIO RANCHO PROJECT NO: 0416.18

TANK MANAGEMENT AREA CROSS-SECTIONS & DRYING PAD LEAK DETECTION DETAILS

C-504

ATTACHMENT III.1.B

LINER LONGEVITY ARTICLE:

GEOSYNTHETICS MAGAZINE, OCT/NOV 2008

How long will my liner last?

| What is the remaining service life of my HDPE geomembrane?

By Ian D. Peggs, P.E., P.Eng., Ph.D.

Introduction

In his keynote lecture at the GeoAmericas-2008 conference last March, Dr. Robert Koerner (et al., 2008) of the Geosynthetic Institute (GSI) reported the ongoing Geosynthetic Research Institute (GRI) work to make the first real stab at assessing the service lives of high-density polyethylene (HDPE), linear low-density polyethylene (LLDPE), reinforced PE, ethylene propylene diene terpolymer (EPDM), and flexible polypropylene (fPP) exposed geomembranes.

The selected environment simulated that of Texas, USA, in sunny ambient temperatures between ~7°C (45°F) and 35°C (95°F). Of course, an exposed black HDPE geomembrane in the sun will achieve much higher temperatures, probably in excess of 80°C (176°F).

I do not know what the temperature would be at 150-300mm above the liner (for those still specifying this parameter), but it is quite immaterial. The only temperature of concern is the actual geomembrane temperature.

The lifetimes are shown in **Table 1**, but it must be recognized that these data are for specific manufactured products with specific formulations. The "greater than" notation indicates that laboratory exposures (incubations) are still on-going, not that some samples have failed after the indicated time period. The PE-R-1 material is a thin LLDPE, so it might be expected to be the first to reach the defined end of life; the half-life—the time to loss of 50% of uniaxial tensile properties.

It is interesting to note that HDPE-1 and LLDPE-1 are proceeding apace, but it would be expected that the LLDPE-1 would reach its half-life earlier than HDPE-1. However, this does not automatically follow. With adequate additive formulations, perhaps LLDPE could be left exposed and demonstrate more weathering resistance than some HDPEs. This demonstrates the fact that all PEs, whether HD or LLD, are not identical—they can have different long-term performances dependent on the PE resin used and the formulation of the stabilizer package. However, such differences are not evident in the conventional mechanical properties such as tensile strength/elongation, puncture and tear resistances, and so on.

The two fPPs are performing well. However, there had also been an fPP-1, one of the first PP geomembranes that did not perform well. This was due to a totally inappropriate stabilizer formulation. That particular product lasted 1.5 years in service. In

Final Inspection continued on page 44

Type	Specification	Predicted Lifetime in Texas, USA
HDPE-1	GRI-GM13	>28 years (Incubation ongoing)
LLDPEE-1	GRI-GM17	>28 years (Incubation ongoing)
EPDM-1	GRI-GM21	>20 years (Incubation ongoing)
PE-R-1	GRI-GM22	≈17 years (reached halflife)
fPP-2	GRI-GM18 (temp. susp.)	>27 years (Incubation ongoing)
fPP-3	GRI-GM18 (temp. susp.)	>17 years (Incubation ongoing)

Table 1 | Estimated exposed geomembrane lifetimes

| Ian Peggs is president of I-CORP International Inc. and is a member of *Geosynthetics* magazine's Editorial Advisory Committee.

Final Inspection continued from page 56

the QUV weatherometer, it lasted 1,800 light hours at 70°C (158°F). Therefore, the lab/field correlation is that 1,000 QUV light hours is equivalent to a 0.83yr service life under those specific environmental conditions.

At another location in Texas, Koerner/GRI found 1,000hr of QUV exposure was equivalent to 1.1 year actual field exposure. Consequently, for Texas exposures GRI is using a correlation of 1000hr QUV exposure as equivalent to lyr of in-service exposure. Clearly, the correlation would be different in less sunny and colder environments.

The failed fPP-1 liner was replaced with a correctly stabilized fPP that, subsequently, performed well.

So how can we evaluate the condition of our exposed liners in a simple and practical manner to ensure they will continue to provide adequate service lifetimes and to get sufficient warning of impending expiration?

For each installation, a baseline needs to be established, and changes from that baseline need to be monitored.

A liner lifetime evaluation program

Rather than be taken by surprise when a liner fails or simply expires, it should be possible to monitor the condition of the liner to obtain a few years of notice for impending expiration. One can then plan for a timely replacement without the potential for accidental environmental damage and undesirable publicity. A program of periodic liner-condition assessment is proposed.

... it should be possible to monitor the condition of the liner to obtain a few years of notice for impending expiration.

While estimated correlations might be made for other locations using historical weather station sunshine and temperature data, there is no question that the best remaining lifetime assessments will be obtained using samples removed from the field installation of interest.

A lifetime in excess of 28yr, demonstrated for a recently-made HDPE geomembrane, is comparable to the present actual service periods of as long as 30-35yr. However, actual lifetimes of as low as ~15yr have also been experienced.

Do service lifetimes now exceeding 30yr mean that we might expect to see another round of stress cracking failures as exposed liners finally oxidize sufficiently on the surface to initiate stress cracking?

This would be frustrating after resolving the early 1980s problems with stress cracking failures at welds and stone protrusions when the liners contracted at low temperatures, but it is the way end-of-life will become apparent. And will that be soon or in another 5-20 years? It would be useful to know.

For baseline data, it would be useful to have some archive material to test, but that is not usually available. Manufacturers often discard retained samples after about 5 years. Perhaps facility owners should be encouraged to keep retained samples at room temperature and out of sunlight. The next best thing is to use material from the anchor trench or elsewhere that has not experienced extremes in temperature and that has not been exposed to UV radiation or to expansion/contraction stresses.

Less satisfactory options are to use the original NSF 54 specifications, the manufacturer's specifications, or the GRI-GM13 specifications at the appropriate time of liner manufacturing. The concern with using these specifications is that while aged material may meet them, there is no indication of whether the measured values have significantly decreased from the actual as-manufactured values that generally significantly exceed the specification.

A final option for the baseline would be to use the values at the time of the first liner assessment.

The first liner condition assessment would consist of a site visit during which a general visual examination would be done together with a mechanical probing of the edges of welds. A visual examination would include the black/gray shades of different panels that might indicate low carbon contents.

A closer examination should be done using a loupe (small magnifier) on suspect areas such as wrinkle peaks, the tops and edges of multiple extrusion weld beads, and the apex-down creases of round die-manufactured sheet.

The last detail is significant because the combination of oxidizing surface and exposed surface tension when the liner contracts at low temperatures and the crease is pulled flat can be one of the first locations to crack. The apex-up creases do not fail at the same time because the oxidized exposed surface is under compression (or less tension) when the crease is flattened out.

Appropriate samples for detailed laboratory testing will be removed.

It may be appropriate to do a water lance electrical integrity survey on the exposed sideslopes, but this would only be effective on single liners, and on double liners with a composite primary liner, a conductive geomembrane, or a geocomposite with a conductive geotextile on top.

A sampling and testing regime

A liner lifetime evaluation program should be simple, meaningful, and cost-effective.

While it will initially require expert polymer materials science/engineering input to analyze the test data and to define the critical parameters, it should ultimately be possible to use an expert system to automatically make predictions using the input test data.

Small samples will be taken from deep in the anchor trench and from appropriate

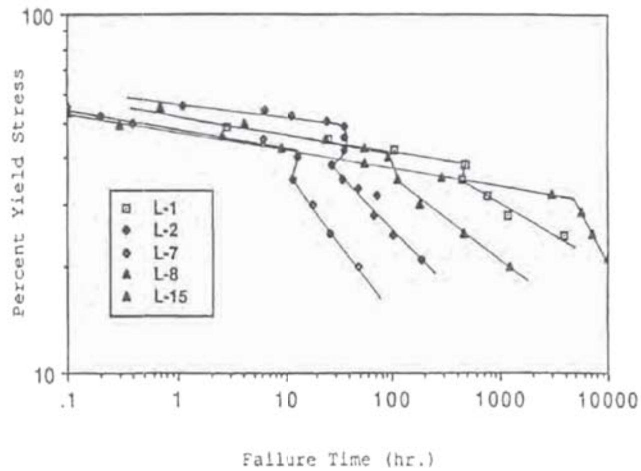

Figure 1 | Standard stress rupture curves for five HDPE geomembranes (Hsuan, et al. 1992)

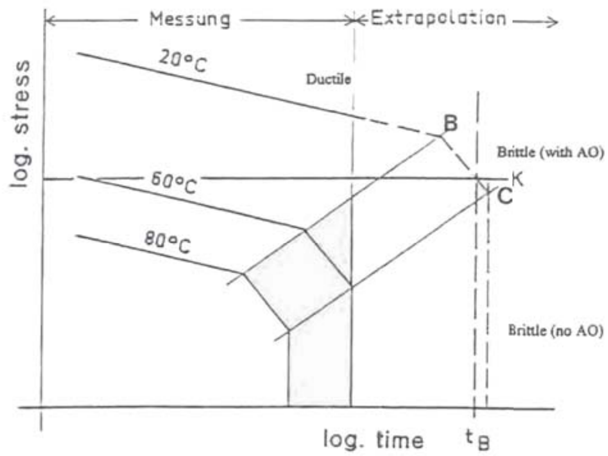

Figure 2 | Stress rupture curves showing third stage (Brittle no AO) oxidized limit. (Gaube, et al. 1985)

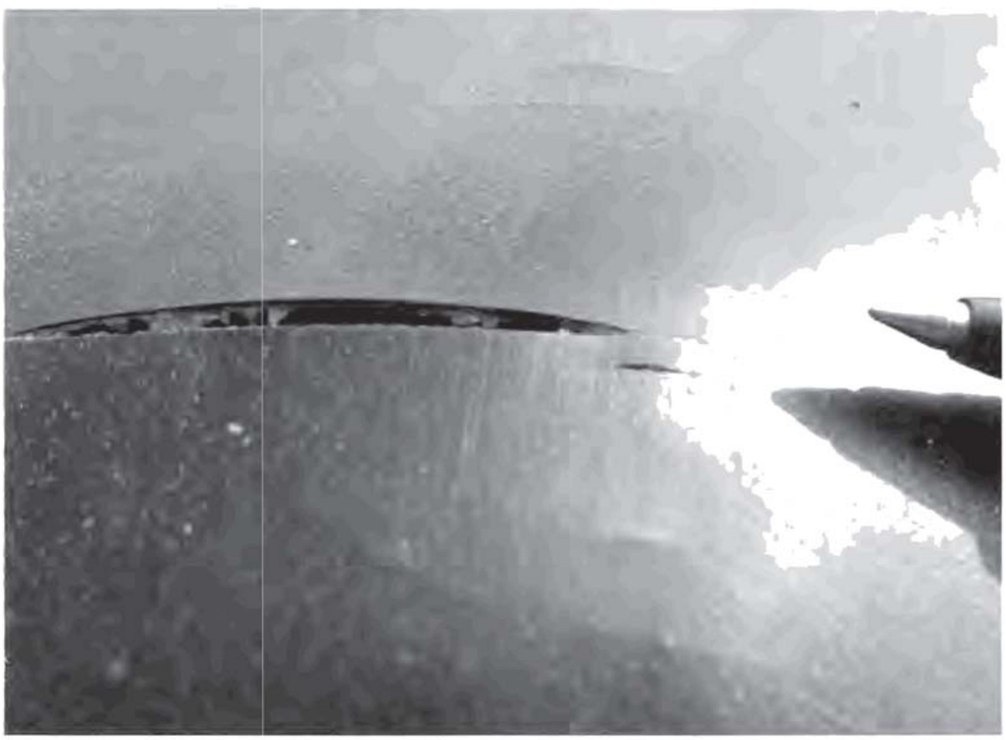

Figure 3 | Stress crack initiated by extruder die line at stone protrusion

exposed locations. Potential sites for future sample removal by the facility owner for future testing will be identified and marked by the expert during the first site visit.

The baseline sample(s) will be tested as follows:

- Single-point stress cracking resistance (SCR) on a molded plaque by ASTM D5397
- High-pressure oxidative induction time (HP-OIT) by ASTM D5885
- Fourier transform infrared spectroscopy (FTIR-ATR) on upper surface to determine carbonyl index (CI) on nonarchive samples only
- Oven aging/HP-OIT (GRI-GM13)
- UV resistance/HP-OIT (GRI-GM13)

The exposed samples will be tested as follows:

- Carbon content (ASTM D1603)
- Carbon dispersion (ASTM D5596)
- Single-point SCR on molded plaque (ASTM D5397)
- Light microscopy of exposed surface, through-thickness cross sections, and thin microsections (~15 µm thick) as necessary
- HP-OIT on 0.5-mm-thick exposed surface layers from basic sheet and from sheet at edge of extruded weld bead (ASTM D5885), preferably at a double-weld bead
- FTIR-ATR on exposed surface to determine CI
- Oven aging/HP-OIT on 0.5mm surface layer (GRI-GM13)
- UV resistance/HP-OIT on 0.5 mm surface layer (GRI-GM13)

Carbon content is done to ensure adequate basic UV protection. Carbon dispersion is done to ensure uniform surface UV protection and to evaluate agglomerates that might act as initiation sites for stress cracking.

HP-OIT is used to assess the remaining amount of stabilizer additives, both in the liner panels and in the sheet adjacent to an extrusion weld. Most stress cracking is observed at the edges of extrusion weld beads in the lower sheet, so it is important to monitor this location.

While standard OIT (ASTM D3895 at 200°C) better assesses the relevant stabilizers effective at processing (melting) and welding temperatures, the relevant changes in effective stabilizer content during continued service, including in the weld zone, will be provided by measurement of HP-OIT. There will be no future high temperature transient where knowledge of S-OIT will be useful. It is expected that the liner adjacent to the weld bead will be more deficient in stabilizer than the panel itself. Therefore, S-OIT is not considered in this program.

Note that HP-OIT is measured on a thin surface layer because the surface layer may be oxidized while the body of the geomembrane may not. If material from the full thickness of the geomembrane is used it could show a significant value of OIT, implying that there is still stabilizer present and that oxidation is far from occurring. However, the surface layer could be fully oxidized with stress cracks already initiated and propagating. A crack will then propagate more easily through unoxidized material than would initiation and propagation occur in unoxidized material.

The fact that the HP-OIT meets a certain specification value in the as-manufactured condition provides no guarantee that thermo- and photo-oxidation protection will be provided for a long time. Stabilizers might be consumed quickly or slowly while providing protection. They may also be consumed quickly to begin with, then more slowly, or vice versa.

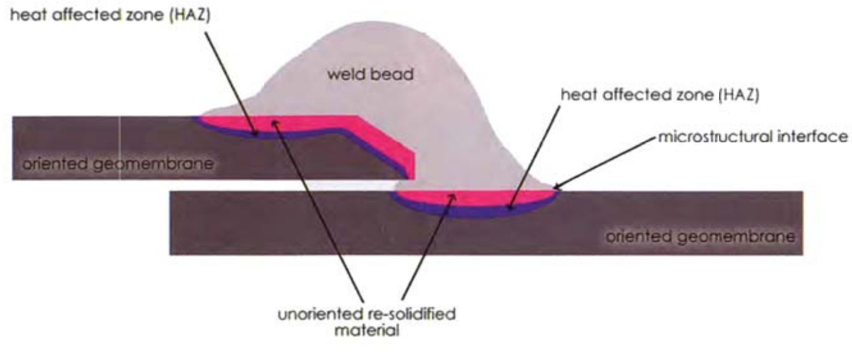

Figure 4 | Schematic of microstructure at extrusion weld

Hence, the need for continuing oven (thermal) aging and UV resistance tests. These two parameters, assessed by measuring retained HP- OIT, are critical to the assessment of remaining service life.

Oven (thermal) aging and UV resistance tests performed in this program will provide an extremely valuable data base that relates laboratory testing to in-service performance and that will further aid in more accurately projecting in-service performance from laboratory testing results.

Special considerations

Because we do not know, by OIT measurements alone, whether the surface layer is or is not oxidized (unless OIT is zero), and since we do not yet know at what level of OIT loss there might be an oxidized surface layer (the database has not yet been generated), FTIR directly on the surface of the geomembrane is performed using the attenuated total reflectance (ATR) technique to deny or confirm the presence of oxidation products (carbonyl groups).

Following the practice of Broutman, et al. (1989) and Duvall (2002) on HDPE pipes, if the ratio of the carbonyl peak at wave number 1760 cm-1 and the C-H stretching (PE) peak at wave number 1410 cm -1 is more than 0.10, there is a sufficiently oxidized surface layer that stress cracking might be initiated. For those familiar with the two slope stress rupture curve (**Figure 1**) where the brittle stress cracking region is the steeper segment below the knee, there is a third vertical part of the curve (**Figure 2**) where the material is fully oxidized and fracture occurs at the slightest stress. This is what will happen at the end of service life. But first note the times to initiation of stress cracking (the knees in the curves) in **Figure 1**—they range from ~10/hr to ~5,000/hr—clearly confirming that all HDPEs are not the same. Some are far more durable than others.

At the end of service life, at some level of OIT, there will be a critically oxidized surface layer that when stressed, such as at low temperatures by an upwards protruding stone, or by flexing due to wind uplift, will initiate a stress crack on the surface that will propagate downward through the geomembrane, as shown by the crack in **Figure 3**.

This crack, initiated at a stress concentrating surface die mark, occurred when the liner contracted at low temperatures, and tightened over an upwardly protruding stone. The straight morphology of the crack, and the ductile break at the bottom surface as the stress in the remaining ligament rose above the knee in the stress rupture curve, are typical of a stress crack. Note the shorter stress cracks initiated along other nearby die marks.

Stress cracks are preferentially initiated along the edges of welds because the adjacent geomembrane has been more depleted of stabilizers during the high temperature welding process. Thus, under further oxidizing service conditions, it will become the first location to

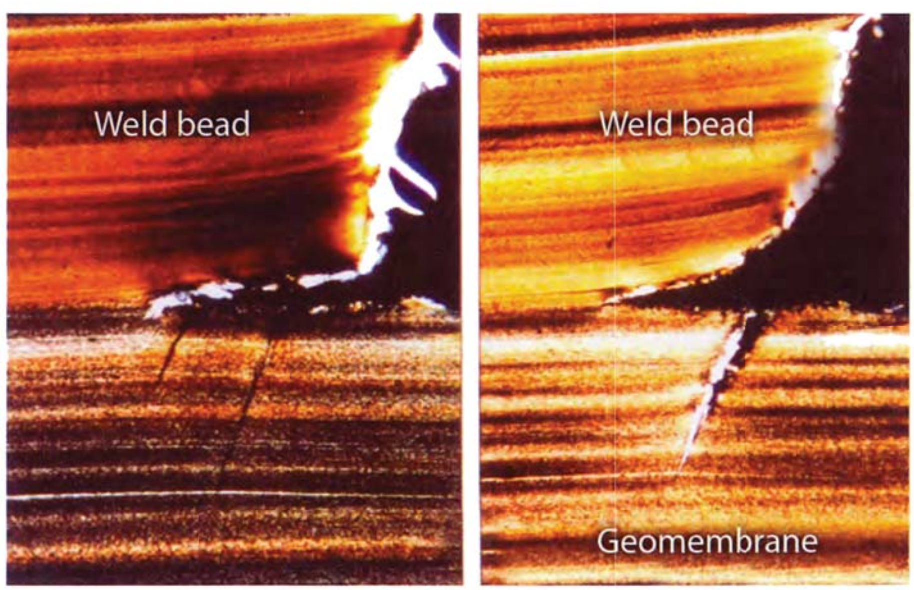

Figure 5 | Typical off-normal angle of precursor crazes (left) and stress crack (right) at edge of extrusion weld.

Type	Specification	Predicted Lifetime in Texas, USA
Side wall exposed	54	5
Side wall concrete side	81	71
Lower launder exposed	16	3
Lower launder concrete side	145	1

Table 2 | S-OIT values on solution and concrete liner surfaces (Peggs, 2008).

be oxidized to the critical level at which stress cracks will be initiated under any applied stress. In addition, the geometrical notches at grinding gouges and at the edges of the bead increase local stresses to critical levels for SC to occur.

I also believe that an internal microstructural flaw exists between the originally oriented geomembrane structure and the pool of more isotropic melted and resolidified material at the edge of the weld zone, as shown schematically in **Figure 4**. Most stress cracks occur at an off-normal angle at the edge of the weld bead that may be related to the angle of this molten-pool to oriented-structure interface (**Figure 5**). It is also known that stress increases the extraction of stabilizers from polyolefin materials.

With all of these agencies acting synergistically, it is not surprising that stress cracking often first occurs adjacent to extrusion welds.

Looking ahead

With the first field assessment test results available to us, and the extent of changes from the baseline sample known, removal of a second set of samples by the facility owner (at locations previously identified and marked by the initial surveyor), will be planned for a future time, probably in 2 or 3 years.

Why 2 or 3 years? In an extreme chemical environment, extensive reductions in S-OIT of studded HDPE concrete protection liners in mine solvent extraction facilities using kerosene/aromatic hydrocarbon/sulfuric acid process solutions at 55°C (131°F) have been observed on the solution and concrete sides of the liner (**Table 2**) within 1 year (Peggs 2008). But it is unlikely that such rapid decreases will be observed in air-exposed material.

With this second set of field samples, and with three sets of data points, practically reliable extrapolations of remaining lifetime can start to be made.

It is expected that a few years of notice for impending failures will be possible.

The key point to note in making these condition assessments is that, while all HDPE geomembranes have very similar conventional index properties, they can have widely variable photo-oxidation, thermal-oxidation, and stress-cracking resistances. Therefore, some HDPEs are more durable than others.

Thus, while one HDPE geomembrane manufactured in 1990 failed after 15 years in 2005, another HDPE geomembrane made in 1990 from a different HDPE resin (or more correctly a medium-density polyethylene [MDPE] resin), and with a better stabilizer additive package, could still have a remaining lifetime of 5, 20, or 30 years.

So, keep a close eye on those exposed liners and we'll learn a great deal more about liner performance and get notice of the end of service lifetime. And if owners can retain some archive material from new installations, so much the better.

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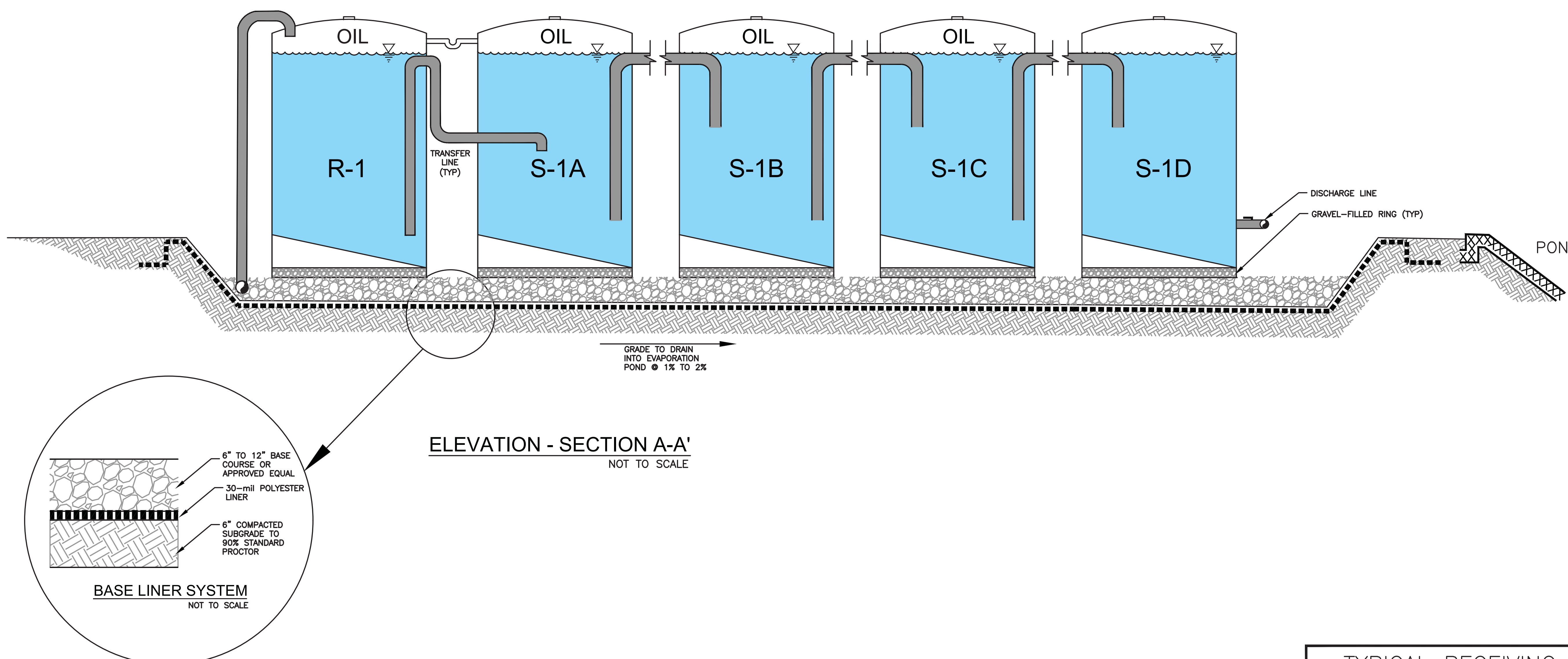
ATTACHMENT III.1.C

TYPICAL RECEIVING TANK INSTALLATION DETAILS



ELEVATION - SECTION A-A'

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Date/Time:Mar. 25, 2019-06:57:42

TYPICAL RECEIVING TANK INSTALLATION DETAILS

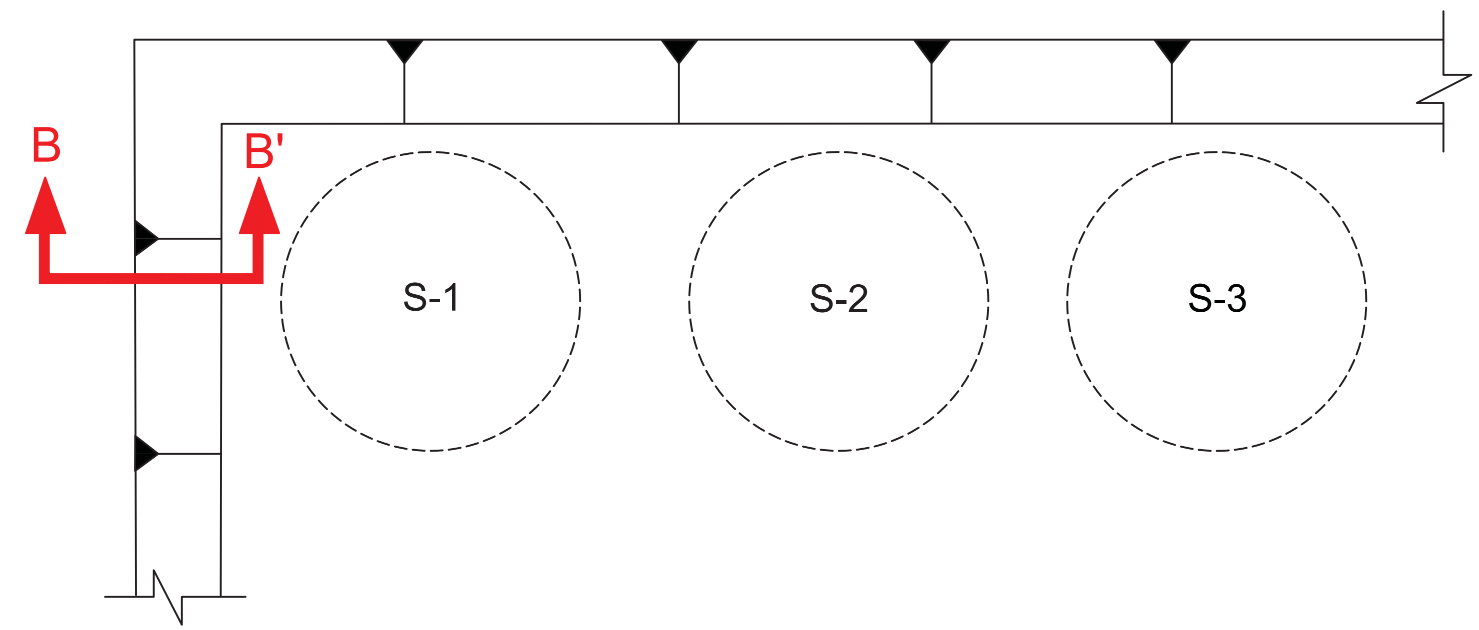
LEA LAND LLC SURFACE WASTE MANAGEMENT FACILITY
LEA COUNTY, NEW MEXICO

GORDON ENVIRONMENTAL PSC
333 Rio Rancho Blvd. NE
Rio Rancho, NM 87124
Phone: 505-867-6990
Fax: 505-867-6991

DATE: 03/19/2019	CAD: TANK ELEV.dwg	PROJECT #: 0416.18
DRAWN BY: DMI	REVIEWED BY: CRK	ATTACHMENT III.1.C
APPROVED BY: CWF	www.team-psc.com	

ATTACHMENT III.1.D

TYPICAL SALES TANK INSTALLATION DETAILS



PLAN VIEW

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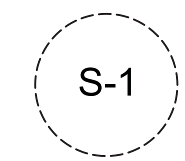
LEGEND



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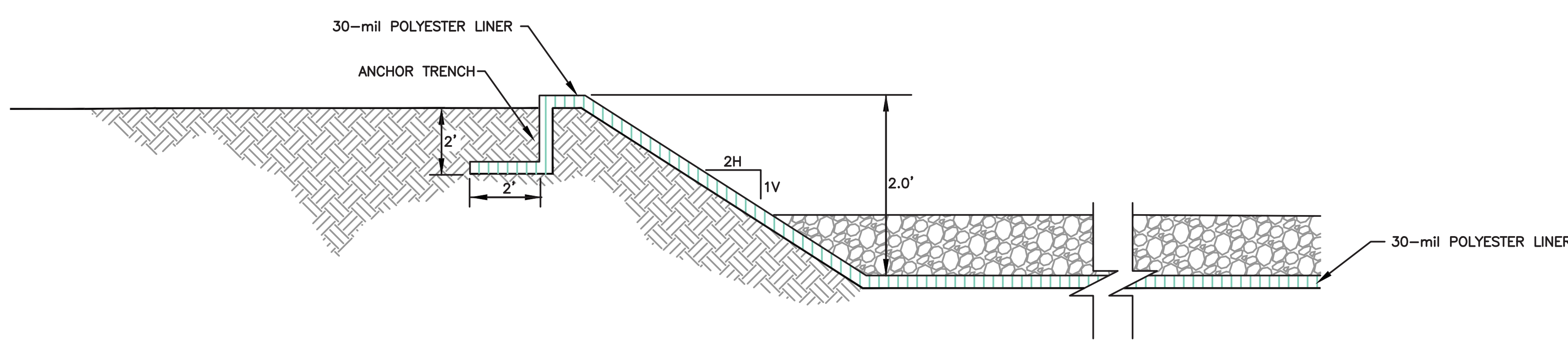

CROSS SECTION B-B'

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TYPICAL SALES TANK INSTALLATION DETAILS

LEA LAND LLC SURFACE WASTE MANAGEMENT FACILITY
LEA COUNTY, NEW MEXICO

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333 Rio Rancho Blvd. NE
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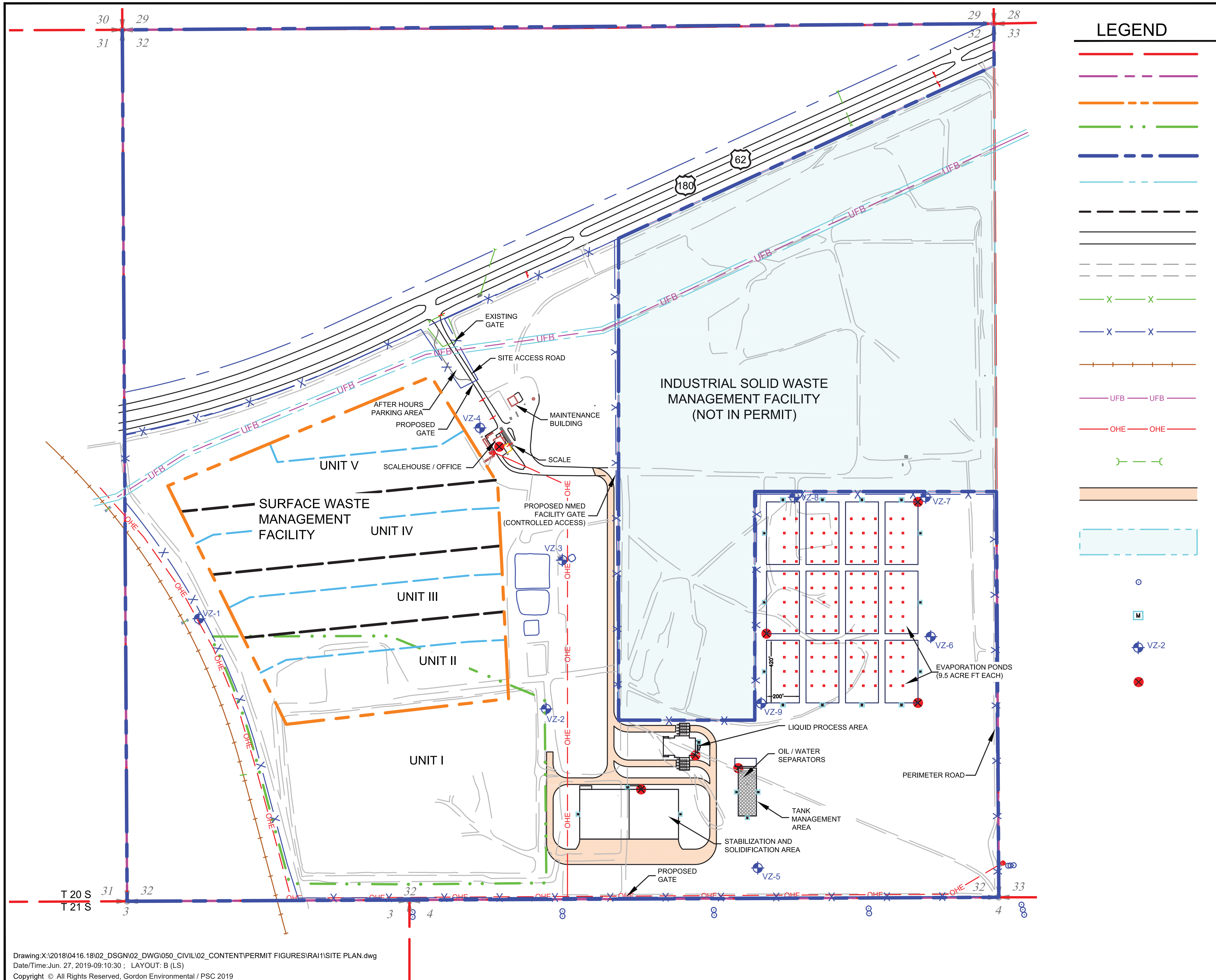
DRAWN BY: DMI	REVIEWED BY: CRK	ATTACHMENT III.1.D
APPROVED BY: CWF	www.team-psc.com	

ATTACHMENT III.1.E
SITE PLAN



SITE PLAN

LEA LAND LLC SURFACE WASTE MANAGEMENT FACILITY
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APPROVED BY: CWF	www.team-psc.com	

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ATTACHMENT III.1.F
TANK CAPACITY CALCULATIONS

ATTACHMENT III.1.F - Tank and Pond Capacity Calculations

Lea Land is a surface waste management facility.

A. Produced Water is delivered by trucking companies into one of twelve proposed heated Produced Water Receiving Tanks located within a bermed, lined containment area:

Proposed Tank No.	Volume	Permitted
R-1	1000 bbls	Permitted under this Application
R-2	1000 bbls	Permitted under this Application
R-3	1000 bbls	Permitted under this Application
R-4	1000 bbls	Permitted under this Application
R-5	1000 bbls	Permitted under this Application
R-6	1000 bbls	Permitted under this Application
R-7	1000 bbls	Permitted under this Application
R-8	1000 bbls	Permitted under this Application
R-9	1000 bbls	Permitted under this Application
R-10	1000 bbls	Permitted under this Application
R-11	1000 bbls	Permitted under this Application
R-12	1000 bbls	Permitted under this Application

i. The Receiving tanks serve to gravity separate solids and oil from the water. Solids collect in the bottoms and oil floats to the tops of the receiving tanks.
ii. The Receiving Tanks bottoms are solidified and taken to the OCD permitted Landfill.
iii. The Receiving Tanks are set on gravel or sand pads on top of a lined bermed impermeable pad.

B. Water from each Receiving Tanks flows in series through four additional Settling Tanks to remove oil prior to discharge in the mechanical oil water separator:

Proposed Tank No.	Volume	Permitted
S-1A	1000 bbls	Permitted under this Application
S-1B	1000 bbls	Permitted under this Application
S-1C	1000 bbls	Permitted under this Application
S-1D	1000 bbls	Permitted under this Application
S-2A	1000 bbls	Permitted under this Application
S-2B	1000 bbls	Permitted under this Application
S-2C	1000 bbls	Permitted under this Application
S-2D	1000 bbls	Permitted under this Application
S-3A	1000 bbls	Permitted under this Application
S-3B	1000 bbls	Permitted under this Application
S-3C	1000 bbls	Permitted under this Application
S-3D	1000 bbls	Permitted under this Application
S-4A	1000 bbls	Permitted under this Application
S-4B	1000 bbls	Permitted under this Application
S-4C	1000 bbls	Permitted under this Application
S-4D	1000 bbls	Permitted under this Application
S-5A	1000 bbls	Permitted under this Application
S-5B	1000 bbls	Permitted under this Application
S-5C	1000 bbls	Permitted under this Application
S-5D	1000 bbls	Permitted under this Application
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S-6B	1000 bbls	Permitted under this Application
S-6C	1000 bbls	Permitted under this Application
S-6D	1000 bbls	Permitted under this Application
S-7A	1000 bbls	Permitted under this Application

ATTACHMENT III.1.F - Tank and Pond Capacity Calculations

S-7B	1000 bbls	Permitted under this Application
S-7C	1000 bbls	Permitted under this Application
S-7D	1000 bbls	Permitted under this Application
S-8A	1000 bbls	Permitted under this Application
S-8B	1000 bbls	Permitted under this Application
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S-11C	1000 bbls	Permitted under this Application
S-11D	1000 bbls	Permitted under this Application
S-12A	1000 bbls	Permitted under this Application
S-12B	1000 bbls	Permitted under this Application
S-12C	1000 bbls	Permitted under this Application
S-12D	1000 bbls	Permitted under this Application

i. The Settling Tanks increase the detention time available to provide additional gravity separation of oil from the water,
ii. The Settling Tank bottoms are taken to the Stabilization/Solidification Area.
iii. The Settling Tanks are set on gravel or sand pads on top of a lined bermed impermeable pad.

C. The separated oil flows into one of five heated Crude Oil Receiving Tanks:

Proposed Tank No.	Volume	Permitted
C-1	1000 bbls	Permitted under this Application
C-2	1000 bbls	Permitted under this Application
C-3	1000 bbls	Permitted under this Application
C-4	1000 bbls	Permitted under this Application
C-5	1000 bbls	Permitted under this Application

i. The Crude Oil Receiving Tanks are set inside the proposed lined containment berm.
ii. The Crude Oil Receiving Tanks are interconnected at the top of the tanks for oil removal.
iii. Water recovered from the Crude Oil Receiving Tanks is redirected to the Produced Water Receiving Tanks.
iv. Sludges recovered from the Crude Oil Receiving Tanks are stabilized, solidified and sent for landfill disposal.

D. The water from the Settling Tanks is discharged through one of up to four Dissolved Air Floatation (DAF) Units.

Proposed Tank No.	Volume	Permitted
D-1	10 bbls	Permitted under this Application
D-2	10 bbls	Permitted under this Application
D-3	10 bbls	Permitted under this Application
D-4	10 bbls	Permitted under this Application

i. The DAF Units are situated on the lined Evaporation Pond berm in a location where any leackage would drain
ii. The DAF use air bubles to lift any remaining oil from the water prior to dischage into one of four Ponds.
iii. The oil containing foam generated by the DAF is collected and discharged into the Crude Oil Receiving Tanks for further processing.

ATTACHMENT III.1.F - Tank and Pond Capacity Calculations

E.

Proposed Pond No.	Storage Volume	Permitted
Pond A1	73,700 bbls	Permitted under this Application
Pond A2	73,700 bbls	Permitted under this Application
Pond A3	73,700 bbls	Permitted under this Application
Pond B1	73,700 bbls	Permitted under this Application
Pond B2	73,700 bbls	Permitted under this Application
Pond B3	73,700 bbls	Permitted under this Application
Pond C1	73,700 bbls	Permitted under this Application
Pond C2	73,700 bbls	Permitted under this Application
Pond C3	73,700 bbls	Permitted under this Application
Pond D1	73,700 bbls	Permitted under this Application
Pond D2	73,700 bbls	Permitted under this Application
Pond D3	73,700 bbls	Permitted under this Application

i. Surface aeration and bleach are used to maintain water chemistry parameters:
:O_2 at or above 0.5 ppm one foot off the bottom of the pond.
:pH above 8
ii. H2S monitors are placed around the pond covering the four major points on the compass.
iii. The H2S monitors continually monitor the ambient air.
iv. Two chlorine monitors are placed around the ponds covering the North and West borders.
v. Treatment capacity of each Pond is 73,994 bbls (~9.5 acre feet)
vi. 3.5 Feet of Freeboard is proposed, storage volume does include freeboard
vii. Volume including freeboard is 122,640 bbls (15.76 acre-feet)per pond
viii. Inside grade shall be no steeper than 3H:1V
ix. Levees shall have an outside grade no steeper than 3H:1V
x. Levees' tops shall be wide enough to install an anchor trench and provide adequate room for inspection/maintenance.
xi. Liner seams shall be minimized and oriented up and down, not across a slope
Each pond shall have a:
:primary liner (60-mil HDPE liner, UV resistant)
:secondary liner (60-mil HDPE liner, UV resistant)
xii. Slope shall be 2% (2 ft V for 100 ft H)
xiii. A mechanical evaporation system shall be installed in each pond to enhance evaporation.
xiv. Approximate size of each pond is 200 x 420 feet x 7.6 feet deep

F. Bleach for H2S management is stored in two proposed chemical tanks:

Proposed Tank No.	Volume	Permitted
B-1	60 bbls	Permitted under this Application
B-2	60 bbls	Permitted under this Application

i. The Chemical Tanks are set on a bermed concrete pad that drains into the pond.
ii. The Bleach is pumped through lines to discharge points in each of the ponds.

G. Water is discharged from the mechanical oil-water separators to Ponds A1, B1, and C1:
i. Floating evaporator in Ponds A1, B1 and C1 atomize water for evaporation.
ii. Six floating evaporators are situated in each Pond.
iii. Excess water from the first Ponds (A1, B1 or C1) decants through a spillway to Ponds A2, B2, and C2.
iv. Excess water from the second Ponds (A2, B2 or C2) decants through a spillway to Ponds A3, B3, and C3.
v. Excess water from the third Ponds (A3, B3 or C3) decants through a spillway to Ponds A4, B4, and C4.

ATTACHMENT III.1.F - Tank and Pond Capacity Calculations

H. The Jet-Out Pit receives discharges from tankers bringing oil contaminated drilling mud, BS&W, tank bottoms and washout from tank cleanings.

Proposed Pit No.	Volume	Permitted
J-1	1000 bbls	Permitted under this Application

Proposed Tank No.	Volume	Permitted
WW-1	1000 bbls	Permitted under this Application
FW-1	1000 bbls	Permitted under this Application

i. Wash-Water for the Jet-Out Pit is recycled through a line from Pond A4 to WW-1. A pump connected to WW-1 pumps the water through a line to one of six wash-out stations for use cleaning the tankers.
ii. Fresh-Water for the Jet-Out Pit is discharged from the water supply through an air gap into FW-1. A pump connected to FW-1 pumps the water through a line to one of six wash-out stations for use cleaning the tanks.
ii. Oil from the Jet-Out Pit is transferred through a line to the Crude Oil Receiving Tanks for further Processing..
iii. Water from the Jet-Out Pit is transferred through a line to the Produced Water Receiving Tanks for processing.
iv. Sludges and sediments from the Jet Out Pit is removed with a bucket loader and transferred to the waste stabilization area for stabilization, solidification and disposal.

I. Oil from the Crude Oil Receiving Tanks C1-C5 completed the dewatering process with the finished product transferred to the Oil Sales Tanks.

Proposed Tank No.	Volume	Permitted
S-1	1000 bbls	Permitted under this Application
S-2	1000 bbls	Permitted under this Application
S-3	1000 bbls	Permitted under this Application
S-4	1000 bbls	Permitted under this Application
S-5	1000 bbls	Permitted under this Application

i. The proposed Oil Sales Tanks are set inside the lined berm next to the Crude Oil Receiving Tanks.
ii. Oil is removed from the Oil Sales tank to a tanker at the Oil Sales Load-Out

J. Pond Capacity Calculations:

Truncated Rectangular Pyramid Volume		
Dimension	Freeboard	Pond Volume
a	420	402
b	200	182
c	402	363
d	182	143
h	3	6.5
Volume (GAL)	1,762,291	3,028,410
Acre-FT	5.41	9.29
Barrels		72,075

i. Calculated using:
http://www.aqua-calc.com/calculate/volume-truncated-pyramid
ii. Truncated pyramid or frustum of a pyramid is a pyramid whose vertex is cut away by a plane parallel to the base. The distance between the bottom and the top bases is the truncated pyramid height h.

ATTACHMENT III.1.G

PIPE WALL THICKNESS INFORMATION

Table A-2 (cont'd)
PIPE WEIGHTS AND DIMENSIONS *(IPS)*
PE3608 *(BLACK)*

OD Nominal in.	OD Actual in.	OD Actual mm.	SDR	Nominal ID in.	Nominal ID mm.	Minimum Wall in.	Minimum Wall mm.	Weight lb. per foot	Weight kg. per meter
			7	2.44	61.98	0.500	12.70	2.047	3.047
			7.3	2.48	63.08	0.479	12.18	1.978	2.943
			9	2.68	67.96	0.389	9.88	1.656	2.464
			9.3	2.70	68.63	0.376	9.56	1.609	2.395
			11	2.83	71.77	0.318	8.08	1.387	2.065
3	3.500	88.90	11.5	2.85	72.51	0.304	7.73	1.333	1.984
			13.5	2.95	74.94	0.259	6.59	1.153	1.716
			15.5	3.02	76.74	0.226	5.74	1.015	1.511
			17	3.06	77.81	0.206	5.23	0.932	1.386
			21	3.15	79.93	0.167	4.23	0.764	1.136
			26	3.21	81.65	0.135	3.42	0.623	0.927
			7	3.14	79.68	0.643	16.33	3.384	5.037
			7.3	3.19	81.11	0.616	15.66	3.269	4.865
			9	3.44	87.38	0.500	12.70	2.737	4.073
			9.3	3.47	88.24	0.484	12.29	2.660	3.958
			11	3.63	92.27	0.409	10.39	2.294	3.413
4	4.500	114.30	11.5	3.67	93.23	0.391	9.94	2.204	3.280
			13.5	3.79	96.35	0.333	8.47	1.906	2.836
			15.5	3.88	98.67	0.290	7.37	1.678	2.497
			17	3.94	100.05	0.265	6.72	1.540	2.292
			21	4.05	102.76	0.214	5.44	1.262	1.879
			26	4.13	104.98	0.173	4.40	1.030	1.533
			32.5	4.21	106.84	0.138	3.52	0.831	1.237
			7	3.88	98.51	0.795	20.19	5.172	7.697
			7.3	3.95	100.27	0.762	19.36	4.996	7.435
			9	4.25	108.02	0.618	15.70	4.182	6.224
			9.3	4.29	109.09	0.598	15.19	4.065	6.049
			11	4.49	114.07	0.506	12.85	3.505	5.216
5	5.563	141.30	11.5	4.54	115.25	0.484	12.29	3.368	5.012
			13.5	4.69	119.11	0.412	10.47	2.912	4.334
			15.5	4.80	121.97	0.359	9.12	2.564	3.816
			17	4.87	123.68	0.327	8.31	2.353	3.502
			21	5.00	127.04	0.265	6.73	1.929	2.871
			26	5.11	129.78	0.214	5.43	1.574	2.343
			32.5	5.20	132.08	0.171	4.35	1.270	1.890
			7	4.62	117.31	0.946	24.04	7.336	10.917
			7.3	4.70	119.41	0.908	23.05	7.086	10.545
			9	5.06	128.64	0.736	18.70	5.932	8.827
			9.3	5.11	129.92	0.712	18.09	5.765	8.579
			11	5.35	135.84	0.602	15.30	4.971	7.398
6	6.625	168.28	11.5	5.40	137.25	0.576	14.63	4.777	7.109
			13.5	5.58	141.85	0.491	12.46	4.130	6.147
			15.5	5.72	145.26	0.427	10.86	3.637	5.413
			17	5.80	147.29	0.390	9.90	3.338	4.967
			21	5.96	151.29	0.315	8.01	2.736	4.072
			26	6.08	154.55	0.255	6.47	2.233	3.322
			32.5	6.19	157.30	0.204	5.18	1.801	2.680

See ASTM D3035, F714 and AWWA C-901/906 for OD and wall thickness tolerances.
Weights are calculated in accordance with PPI TR-7.

Table A-2 (cont'd)
PIPE WEIGHTS AND DIMENSIONS *(IPS)*
PE3608 *(BLACK)*

OD Nominal in.	OD Actual in.	OD Actual mm.	SDR	Nominal ID in.	Nominal ID mm.	Minimum Wall in.	Minimum Wall mm.	Weight lb. per foot	Weight kg. per meter
			7	6.01	152.73	1.232	31.30	12.433	18.503
			7.3	6.12	155.45	1.182	30.01	12.010	17.872
			9	6.59	167.47	0.958	24.34	10.054	14.962
			9.3	6.66	169.14	0.927	23.56	9.771	14.541
			11	6.96	176.85	0.784	19.92	8.425	12.538
8	8.625	219.08	11.5	7.04	178.69	0.750	19.05	8.096	12.049
			13.5	7.27	184.67	0.639	16.23	7.001	10.418
			15.5	7.45	189.11	0.556	14.13	6.164	9.174
			17	7.55	191.76	0.507	12.89	5.657	8.418
			21	7.75	196.96	0.411	10.43	4.637	6.901
			26	7.92	201.21	0.332	8.43	3.784	5.631
			7	7.49	190.35	1.536	39.01	19.314	28.743
			7.3	7.63	193.75	1.473	37.40	18.656	27.764
			9	8.22	208.73	1.194	30.34	15.618	23.242
			9.3	8.30	210.81	1.156	29.36	15.179	22.589
			11	8.68	220.43	0.977	24.82	13.089	19.478
10	10.750	273.05	11.5	8.77	222.71	0.935	23.74	12.578	18.717
			13.5	9.06	230.17	0.796	20.23	10.875	16.184
			15.5	9.28	235.70	0.694	17.62	9.576	14.251
			17	9.41	239.00	0.632	16.06	8.788	13.078
			21	9.66	245.48	0.512	13.00	7.204	10.721
			26	9.87	250.79	0.413	10.50	5.878	8.748
			32.5	10.05	255.24	0.331	8.40	4.742	7.058
			7	8.89	225.77	1.821	46.26	27.170	40.433
			7.3	9.05	229.80	1.747	44.36	26.244	39.056
			9	9.75	247.57	1.417	35.98	21.970	32.695
			9.3	9.84	250.03	1.371	34.82	21.353	31.777
			11	10.29	261.44	1.159	29.44	18.412	27.400
12	12.750	323.85	11.5	10.40	264.15	1.109	28.16	17.693	26.330
			13.5	10.75	272.99	0.944	23.99	15.298	22.767
			15.5	11.01	279.56	0.823	20.89	13.471	20.047
			17	11.16	283.46	0.750	19.05	12.362	18.397
			21	11.46	291.16	0.607	15.42	10.134	15.081
			26	11.71	297.44	0.490	12.46	8.269	12.305
			32.5	11.92	302.73	0.392	9.96	6.671	9.928
			7	9.76	247.90	2.000	50.80	32.758	48.750
			7.3	9.93	252.33	1.918	48.71	31.642	47.089
			9	10.70	271.84	1.556	39.51	26.489	39.420
			9.3	10.81	274.54	1.505	38.24	25.745	38.313
			11	11.30	287.07	1.273	32.33	22.199	33.036
14	14.000	355.60	11.5	11.42	290.05	1.217	30.92	21.332	31.746
			13.5	11.80	299.76	1.037	26.34	18.445	27.449
			15.5	12.09	306.96	0.903	22.94	16.242	24.170
			17	12.25	311.25	0.824	20.92	14.905	22.181
			21	12.59	319.70	0.667	16.93	12.218	18.183
			26	12.86	326.60	0.538	13.68	9.970	14.836
			32.5	13.09	332.40	0.431	10.94	8.044	11.970

See ASTM D3035, F714 and AWWA C-901/906 for OD and wall thickness tolerances.
Weights are calculated in accordance with PPI TR-7.

ATTACHMENT III.1.H

TECHNICAL DATA AND SPECIFICATIONS FOR XR GEOMEMBRANES

Technical Data and Specifications for XR® Geomembranes

XR-3®
XR-5®
XR-3® PW

Industrial, Municipal and Potable Water Grade Geomembranes

Seaman Corporation
1000 Venture Blvd.
Wooster, Ohio 44691
(330) 262-1111
www.xr-5.com

Seaman Corp. XR Geomembranes

Section 1 - Product Overview/Applications

- All XR Geomembrane products are classified as an Ethylene Interpolymer Alloy (EIA)
- XR-5 grade is high strength and chemically resistant for maximum resistance to high temperature, and broad chemical resistance, including acids, oils and methane
- XR-3 grade for moderate chemical resistant requirement applications such as stormwater and domestic wastewater
- NSF 61 approved XR-3 PW grade for potable water contact
- Heat weldable-thermal weldable for seams as strong as the membrane. Factory panels over 15,000 square feet (1400 sq meters) for less field seaming
- Stability is excellent, with low thermal expansion-contraction properties
- 30+ year application history

Product Application Chart

	XR-5			XR-3	XR-3 PW
	8130	**8138**	**6730**	**8228**	**8130**
High Puncture Resistance	X	X	X		X
UV Resistance	X	X	X	X	X
High Strength Applications	X	X	X		X
Floating Covers (Nonpotable)	X	X	X	X	
Diesel/Jet Fuel Containment	X	X	X		
Industrial Wastewater	X	X	X		
Stormwater	X	X	X	X	
Municipal/Domestic Wastewater	X	X	X	X	
Floating Diversion Baffles/Curtains	X		X		X
Potable Water					X
<-65 Deg F Applications	Contact Seaman Corp.				
Chemically Resistant Applications	X	X	X		

Section 2 - Physical Properties

Part 1- Material Specifications

Property	Test Method	8130 XR-5	8138 XR-5	6730 XR-5
Base Fabric Type Base Fabric Weight	ASTM D 751	Polyester 6.5 oz/yd^2 nominal (220 g/m^2 nominal)	Polyester 6.5 oz/yd^2 nominal (220 g/m^2 nominal)	Polyester 7 oz/yd^2 nominal (235 g/m^2 nominal)
Thickness	ASTM D 751	30 mils min. (0.76 mm min.)	40 mils nom. (1.0 mm nom.)	30 mils min. (0.76 mm min.)
Weight	ASTM D 751	30.0 +- 2 oz/sq yd (1017 +- 2 g/m^2)	38.0 +- 2 oz/sq yd (1288 +- 70 g/m^2)	30.0 +- 2 oz/sq yd (1017 +- 70 g/m^2)
Tear Strength	ASTM D 751 Trap Tear	40/55 lbs. min. (175/245 N min.)	40/55 lbs. min. (175/245 N min.)	
Breaking Yield Strength	ASTM D 751 Grab Tensile	550/550 lbs. min. (2,447/2,447 N min.)	550/550 lbs. min. (2,447/2,447 N min.)	600/550 lbs. min. (2,670/2,447 N min.)
Low Temperature Resistance	ASTM D 2136 4 hrs-1/8" Mandrel	Pass @ -30° F Pass @ -35° C	Pass @ -30° F Pass @ -35° C	Pass @ -30° F Pass @ -35° C
Dimensional Stability	ASTM D 1204 100° C-1 Hr.	0.5% max. each direction	0.5% max. each direction	0.5% max. each direction
Hydrostatic Resistance	ASTM D 751 Procedure A	800 psi min. (5.51 MPa min.)	800 psi min. (5.51 MPa min.)	800 psi min. (5.51 MPa min.)
Blocking Resistance	ASTM D 751 180° F	#2 Rating max.	#2 Rating max.	#2 Rating max.
Adhesion-Ply	ASTM D 413 Type A	15 lbs./in. min. or film tearing bond (13 daN/5 cm min. or FTB)	15 lbs./in. min. or film tearing bond (13 daN/5 cm min. or FTB)	15 lbs./in. min. or film tearing bond (13 daN/5 cm min. or FTB)
Adhesion (minimum) Heat Welded Seam	ASTM D 751 Dielectric Weld	40 lbs./2in. RF weld min. (17.5 daN/5 cm min.)	40 lbs./2in. RF weld min. (17.5 daN/5 cm min.)	15 lbs./in. RF weld min. (15 daN/5 cm min.)
Dead Load Seam Strength	ASTM D 751, 4-Hour Test	Pass 220 lbs/in @ 70° F (Pass 980 N/2.54 cm @ 21° C) Pass 120 lbs/in @ 160° F (Pass 534 N/2.54 cm @ 70° C)	Pass 220 lbs/in @ 70° F (Pass 980 N/2.54 cm @ 21° C) Pass 120 lbs/in @ 160° F (Pass 534 N/2.54 cm @ 70° C)	
Bonded Seam Strength	ASTM D 751 Procedure A, Grab Test Method	550 lbs. min. (2,450 N min.)	550 lbs. min. (2,450 N min.)	550 lbs. min. (2,560 N min.)

Abrasion Resistance	ASTM D 3389 H-18 Wheel 1 kg Load	2,000 cycles min. before fabric exposure, 50 mg/100 cycles max. weight loss	2,000 cycles min. before fabric exposure, 50 mg/100 cycles max. weight loss	2,000 cycles min. before fabric exposure, 50 mg/100 cycles max. weight loss
Weathering Resistance	Carbon-Arc ASTM G 153	8,000 hours min. with no appreciable changes or stiffening or cracking of coating	8000 hours min. with no appreciable change or stiffening or cracking of coating	8000 hours min. with no appreciable change or stiffening or cracking of coating
Water Absorption	ASTM D 471, Section 12 7 Days	0.025 kg/m^2 max. @70° F/21° C 0.14 kg/m^2 max at 212° F/100° C	0.025 kg/m^2 max. @70° F/21° C 0.14 kg/m^2 max at 212° F/100° C	0.025 kg/m^2 max. @70° F/21° C 0.14 kg/m^2 max at 212° F/100° C
Wicking	ASTM D 751	1/8" max (0.3 cm max)	1/8" max. (0.3 cm max.)	1/8" max. (0.3 cm max.)
Bursting Strength	ASTM D 751 Ball Tip	750 lbs. min. (3,330 N min.)	750 lbs. min. (3,330 N min.)	750 lbs. min. (3,330 N min.)
Puncture Resistance	ASTM D 4833	275 lbs. min. 1,200 N min.	275 lbs. min. 1,200 N min.	275 lbs. min. 1,200 N min.
Coefficient of Thermal Expansion/ Contraction	ASTM D 696	8 x 10^{-6} in/in/° F max. (1.4 x 10^{-5} cm/cm/° C max.)	8 x 10^{-6} in/in/° F max. (1.4 x 10^{-5} cm/cm/° C max.)	8 x 10^{-6} in/in/° F max. (1.4 x 10^{-5} cm/cm/° C max.)
Environmental/Chemical Resistant Properties		See Chemical Resistance Table, Page 8	See Chemical Resistance Table, Page 8	See Chemical Resistance Table, Page 8
Puncture Resistance	FED-STD-101C Method 2031	350 lbs. (approx.)	350 lbs. (approx.)	
Cold Crack	ASTM D 2136 4 Hrs, 1/8" Mandrel	Pass at -30° F/-34° C	Pass @ -30° F/-34° C	Pass @ -30° F/-34° C

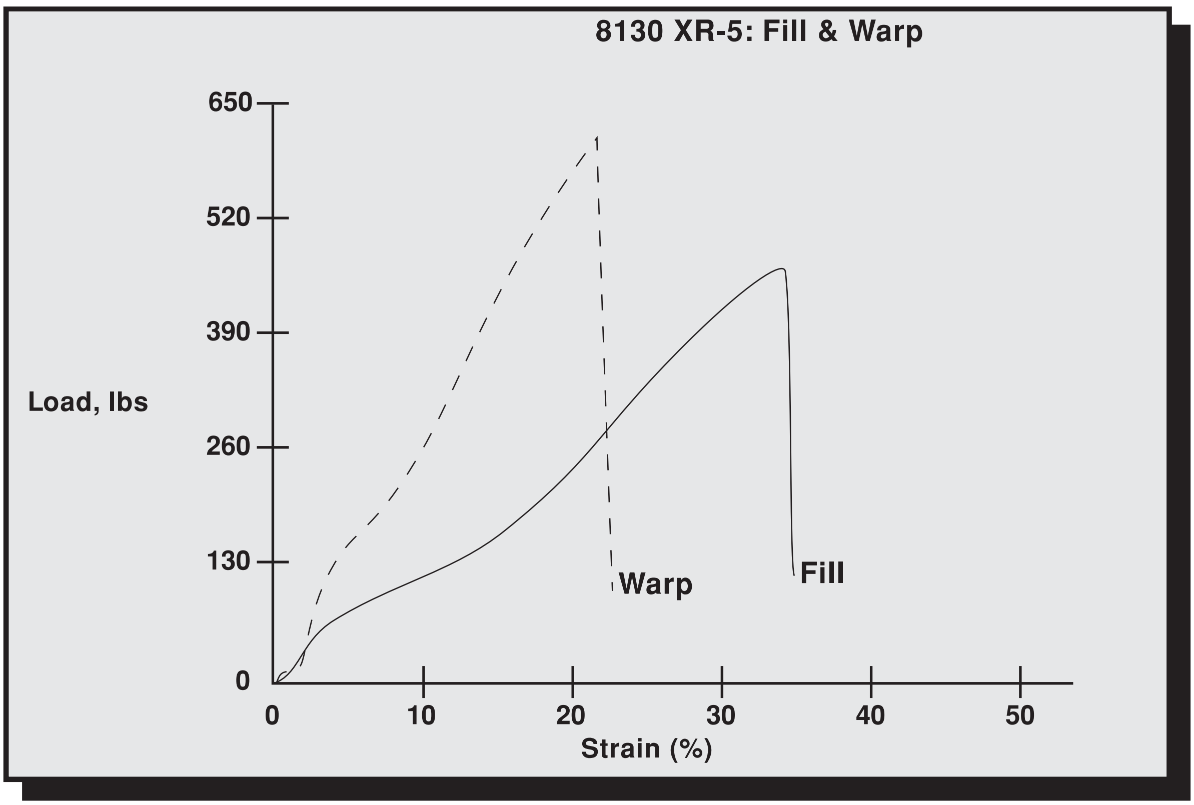
Section 2 - Physical Properties

Part 1- Material Specifications (cont.)

Property	Test Method	8130 XR-3 PW	8228 XR-3
Base Fabric Type Base Fabric Weight	ASTM D 751	Polyester 6.5 oz/yd^2 nominal (220 g/m^2 nominal)	Polyester 3.0 oz/yd^2 nominal (100 g/m^2 nominal)
Thickness	ASTM D 751	30 mils min. (0.76 mm min.)	30 mils min. (0.76 mm min.)
Weight	ASTM D 751	30.0 +- 2 oz./sq. yd. (1017 +- 70 g/sq. m)	28.0 +- 2 oz./sq. yd. (950 +- 70 g/sq. m)
Tear Strength	ASTM D 751 Trap Tear	40/55 lbs. min. (175/245 N min.)	30/30 lbs. nom. (133/133 N nom.)
Breaking Yield Strength	ASTM D 751 Grab Tensile	550/550 lbs. min. (2,447/2447 N min.)	250/200 lbs. min. (1,110/890 N min.)
Low Temperature Resistance	ASTM D 2136 4hrs-1/8" Mandrel	Pass @ -30° F (Pass @ -35° C)	Pass @ -25° F (Pass @ -32° C)
Dimensional Stability	ASTM D 1204 100° C-1 hr.	0.5% max. each direction	5% max. each direction
Hydrostatic Resistance	ASTM D 751 Method A	800 psi min. (5.51 MPa min.)	300 psi min. (2.07 MPa min.)
Blocking Resistance	ASTM D 751 180° F	#2 Rating max.	#2 Rating max.
Adhesion-Ply	ASTM D 413 Type A	15 lbs./in. min. or film tearing bond (13 daN/5 cm min. or FTB)	12 lbs./in. (approx.) (10 daN/5 cm approx.)
Adhesion- Heat Welded Seam	ASTM D 751 Dielectrc Weld	40 lbs./2in. min. (17.5 daN/5 cm min.)	10 lbs./in min. (9 daN/5 cm min.)
Dead Load Seam Strength	ASTM D 751, 4-Hour Test	Pass 220 lbs/in. @ 70° F (Pass 980 N/2.54 cm @ 21° C) Pass 120 lbs/in. @ 160° F (Pass 534 N/2.54 cm @ 70° C)	Pass 100 lbs/in @ 70° F (Pass 445 N @ 21° C) Pass 50 lb @ 160° F (Pass 220 N @ 70° C)
Bonded Seam Strength	ASTM D 751 Procedure A, Grab Test Method	550 lbs. min. (2,450 N min.)	250 lbs. (approx.) (1,112 N min.)

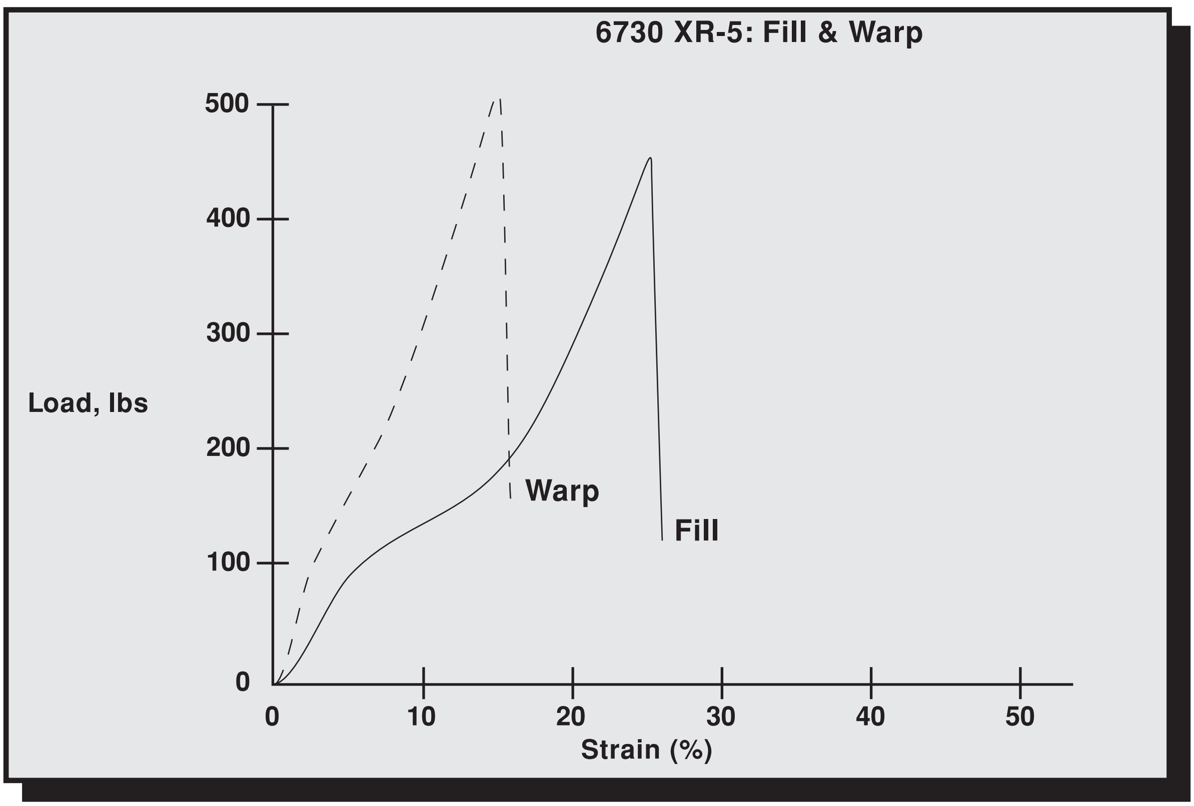
Abrasion Resistance	ASTM D 3389 H-18 Wheel 1 kg Load	2000 cycles min. before fabric exposure, 50 mg/100 cycles max. weight loss	2000 cycles min.
Weathering Resistance	ASTM G 153	8000 hours min. with no appreciable change or stiffening or cracking of coating	8000 hours min.
Water Absorption	ASTM D 471, Section 12 7 Days	0.025 kg/m^2 max. @ 70° F/21° C 0.14 kg/m^2 max @ 212° F/100° C	0.05 kg/m^2 max. @ 70° F/21° C (approx.) 0.28 kg/m^2 max. @ 212° F/100° C (approx.)
Wicking	ASTM D 751	1/8" max. (0.3 cm max.)	1/8" max (0.3 cm max.)
Bursting Strength	ASTM D 751 Ball Tip	750 lbs. min. (3330 N min.)	350 lbs. (approx.) (1557 N min.)
Puncture Resistance	ASTM D 4833	275 lbs. min. 1200 N min.	50 lb typ. (225 N typ.)
Coefficient of Thermal Expansion/ Contraction	ASTM D 696	8×10^{-6} in/in/° F max. (1.4×10^{-5} cm/cm/° C max.)	8×10^{-6} in/in/° F max. (approx.) (1.4×10^{-5} cm/cm/° C max. approx.)
Environmental/Chemical Resistant Properties	ASTM D 741 7-Day Total Immersion With Exposed Edges	NSF 61 approved for potable water	Crude oil 5% max. weight gain Diesel fuel 5% max. weight gain
Puncture Resistance	FTMS 101C Method 2031	350 lbs. (approx.)	205 lbs. (approx.)
Tongue Tear	ASTM D 751		50 lbs. (approx.)

8130 XR-5

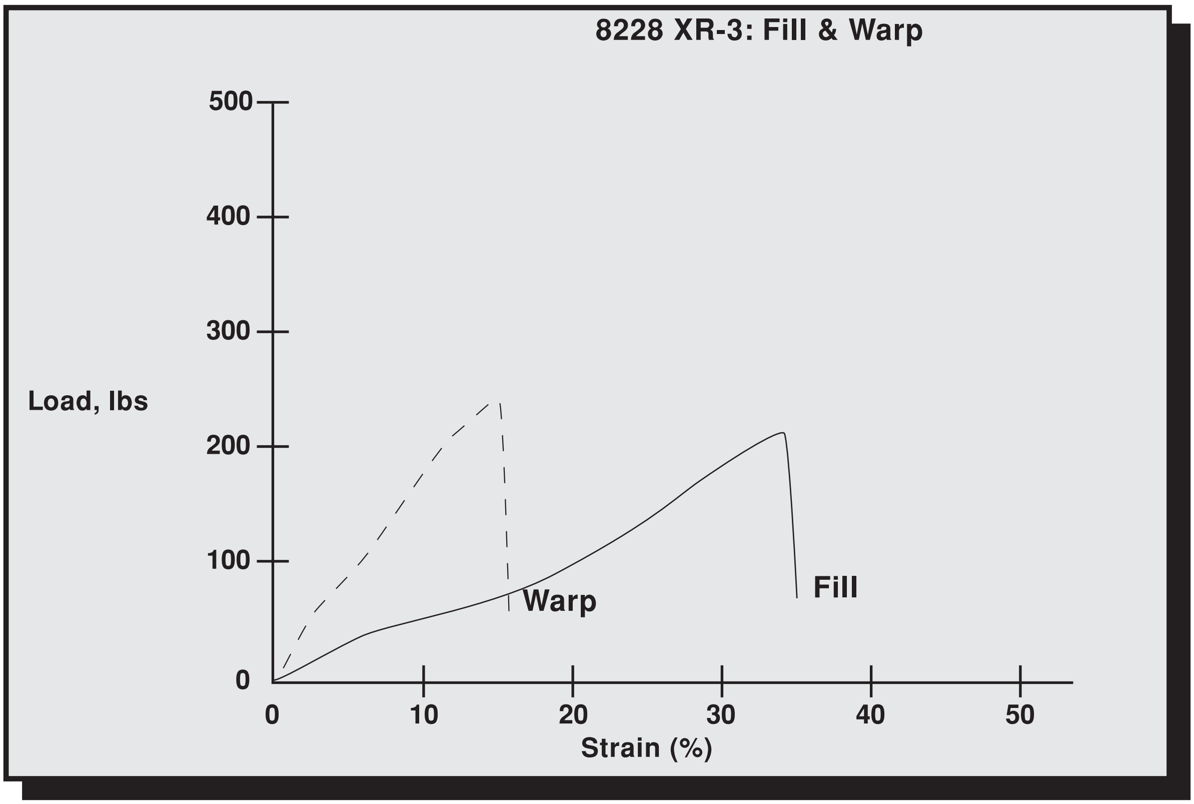

8130 XR-5: Fill & Warp
Load, lbs
650
520
390
260
130
0
0
10
20
30
40
50
Strain (%)
Warp
Fill

Part 2 - Elongation Properties Test

6730 XR-5



8228 XR-3


8228 XR-3: Fill & Warp
Load, lbs
500
400
300
200
100
0
Warp
Fill
0
10
20
30
40
50
Strain (%)

Section 3 - Chemical/Environmental Resistance

Part 1 - XR-5® Fluid Resistance Guidelines

The data below is the result of laboratory tests and is intended to serve only as a guide. No performance warranty is intended or implied. The degree of chemical attack on any material is governed by the conditions under which it is exposed. Exposure time, temperature, and size of the area of exposure usually varies considerably in application, therefore, this table is given and accepted at the user's risk. Confirmation of the validity and suitability in specific cases should be obtained. Contact a Seaman Corporation Representative for recommendation on specific applications.

When considering XR-5 for specific applications, it is suggested that a sample be tested in actual service before specification. Where impractical, tests should be devised which simulate actual service conditions as closely as possible.

EXPOSURE	RATING	EXPOSURE	RATING
AFFF	A	JP-4 Jet Fuel	A
Acetic Acid (5%)	B	JP-5 Jet Fuel	A
Acetic Acid (50%)	C	JP-8 Jet Fuel	A
Ammonium Phosphate	T	Kerosene	A
Ammonium Sulfate	T	Magnesium Chloride	T
Antifreeze (Ethylene Glycol)	A	Magnesium Hydroxide	T
Animal Oil	A	Methanol	A
Aqua Regia	X	Methyl Alcohol	A
ASTM Fuel A (100% Iso-Octane)	A	Methyl Ethyl Ketone	X
ASTM Oil #2 (Flash Pt. 240° C)	A	Mineral Spirits	A
ASTM Oil #3	A	Naphtha	A
Benzene	X	Nitric Acid (5%)	B
Calcium Chloride Solutions	T	Nitric Acid (50%)	C
Calcium Hydroxide	T	Perchloroethylene	C
20% Chlorine Solution	A	Phenol	X
Clorox	A	Phenol Formaldehyde	B
Conc. Ammonium Hydroxide	A	Phosphoric Acid (50%)	A
Corn Oil	A	Phosphoric Acid (100%)	C
Crude Oil	A	Phthalate Plasticizer	C
Diesel Fuel	A	Potassium Chloride	T
Ethanol	A	Potassium Sulphate	T
Ethyl Acetate	C	Raw Linseed Oil	A
Ethyl Alcohol	A	SAE-30 Oil	A
Fertilizer Solution	A	Salt Water (25%)	B
#2 Fuel Oil	A	Sea Water	A
#6 Fuel Oil	A	Sodium Acetate Solution	T
Furfural	X	Sodium Bisulfite Solution	T
Gasoline	B	Sodium Hydroxide (60%)	A
Glycerin	A	Sodium Phosphate	T
Hydraulic Fluid- Petroleum Based	A	Sulphuric Acid (50%)	A
Hydraulic Fluid- Phosphate		Tanic Acid (50%)	A
Ester Based	C	Toluene	C
Hydrocarbon Type II (40% Aromatic)	C	Transformer Oil	A
Hydrochloric Acid (50%)	A	Turpentine	A
Hydrofluoric Acid (5%)	A	Urea Formaldehyde	A
Hydrofluoric Acid (50%)	A	UAN	A
Hydrofluosilicic Acid (30%)	A	Vegetable Oil	A
Isopropyl Alcohol	T	Water (200°F)	A
Ivory Soap	A	Xylene	X
Jet A	A	Zinc Chloride	T

Ratings are based on visual and physical examination of samples after removal from the test chemical after the samples of Black XR-5 were immersed for 28 days at room temperature. Results represent ability of material to retain its performance properties when in contact with the indicated chemical.

Rating Key:
A – Fluid has little or no effect
B – Fluid has minor to moderate effect
C – Fluid has severe effect
T – No data - likely to be acceptable
X – No data - not likely to be acceptable

Vapor Transmission Data

Tested according to ASTM D814-55 Inverted Cup Method

Perhaps a more meaningful test is determination of the diffusion rate of the liquid through the membrane. The vapor transmission rate of Style 8130 XR-5® to various chemicals was determined by the ASTM D814-55 inverted cup method. All tests were run at room temperature and results are shown in the table.

Chemical	8130 XR-5 Black g/hr/m2
Water	0.11
#2 Diesel Fuel	0.03
Jet A	0.11
Kerosene	0.15
Hi-Test Gas	1.78
Ohio Crude Oil	0.03
Low-Test Gas	5.25
Raw Linseed Oil	0.01
Ethyl Alcohol	0.23
Naphtha	0.33
Perchlorethylene	38.58
Hydraulic Fluid	0.006
100% Phosphoric Acid	7.78
50% Phosphoric Acid	0.43
Ethanol (E-96)	0.65
Transformer Oil	0.005
Isopropyl Alcohol	0.44
JP4 (E-96)	0.81
JP8 (E-96)	0.42
Fuel B (E-96)	6.28
Fuel C (E-96)	7.87

Note: The tabulated values are measured Vapor Transmission Rates (VTR). Normal soil testing methods to determine permeability are impractical for synthetic membranes. An "equivalent hydraulic" permeability coefficient can be calculated but is not a direct units conversion. Contact Seaman Corporation for additional technical information.

Seam Strength

Style 8130 XR-5 Black Seam Strength After Immersion

Two pieces of Style 8130 were heat sealed together (seam width 1 inch overlap) and formed into a bag. Various oils and chemicals were placed in the bags so that the seam area was entirely covered. After 28 days at room temperature, the chemicals were removed and one inch strips were cut across the seam and the breaking strength immediately determined. Results are listed below.

Chemical	Seam Strength
None	340 Lbs. Fabric Break- No Seam Failure
Kerosene	355 Lbs. Fabric Break- No Seam Failure
Ohio Crude Oil	320 Lbs. Fabric Break- No Seam Failure
Hydraulic Fluid- Petroleum Based	385 Lbs. Fabric Break- No Seam Failure
Toluene	0 Lbs. Adhesion Failure
Naphtha	380 Lbs. Fabric Break- No Seam Failure
Perchloroethylene	390 Lbs. Fabric Break- No Seam Failure

Even though 1-inch overlap seams are used in the tests to study the accelerated effects, it is recommended that XR-5 be used with a 2-inch nominal overlap seam in actual application. In some cases where temperatures exceed 160ºF and the application demands extremely high seam load, it may be necessary to use a wider width seam.

Long Term Seam Adhesion

11 Years Immersion
ASTM D 751
Lbs./In.

Seam samples of 8130 XR-5® were dielectrically welded together and totally immersed in the liquids for 11 years. The samples were taken out, dried for 24 hours and visually observed for any signs of swelling, cracking, stiffening or degradation of the coating. The coating showed no appreciable degradation and no stiffening, swelling, cracking or peeling.

The adhesion, or resistance to separation of the coating from the base cloth, was then measured by ASTM D 751. Results show 8130 XR-5 maintains seam strength over this long period (11 years).

	Control	Crude Oil	JP-4 Jet Fuel	Diesel Fuel	Kerosene	Naphtha
8130 XR-5	20+	18	33	25	40	33*

Values in lbs./in.

*The naphtha sample was sticky.

We believe this information is the best currently available on the subject. We offer it as a suggestion in any appropriate experimentation you may care to undertake. It is subject to revision as additional knowledge and experience are gained. We make no guarantee of results and assume no obligation or liability whatsoever in connection with this information.

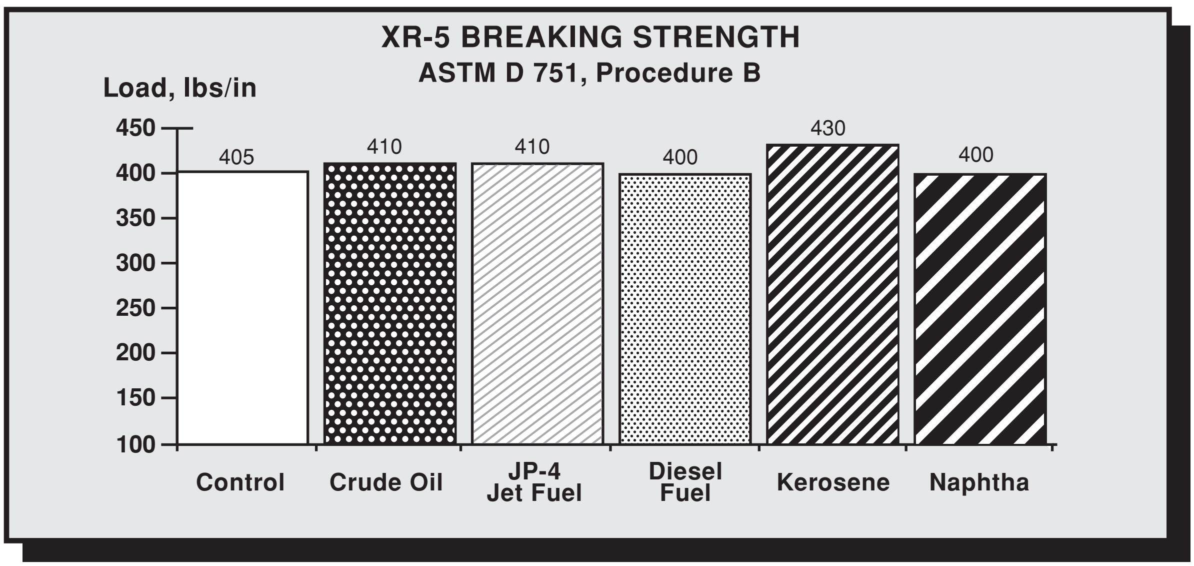
Fuel Compatibility - Long Term Immersion

Test: Samples of 8130 XR-5® Black were immersed in Diesel Fuel, JP-4 Jet Fuel, Crude Oil, Kerosene, and Naphtha for 6 1/2 years.

The samples were then taken out of the test chemicals, blotted and dried for 24 hours. The samples were observed for blistering, swelling, stiffening, cracking or delamination of the coating from the fiber.

Results: It was found in all cases that the 8130 XR-5, after immersion for six years, maintained its strength and there was no evidence of blistering, swelling, stiffening, cracking or delamination.

The strip tensile strength, or breaking strength, of the samples was measured after six years of immersion and the following are the results.



XR-3 Chemical Resistance Statement (Summary)

XR-3® is recommended for moderate chemical resistant applications such as stormwater and municipal wastewater and is not recommended for prolonged contact with pure solutions. XR-3 PW® membranes are recommended only for contact with drinking water and are resistant to low levels of chlorine found in drinking water. XR-5 has a broad range of chemical resistance which is detailed in this section.

Part 2: XR-5® Comparative Chemical Resistance

Chemical Resistance Chart

Comparative Chemical Resistance

	XR-5	HDPE	PVC	Hypalon	Polypropylene
Kerosene	A	B	C	C	C
Diesel Fuel	A	A	C	C	C
Acids (General)	A	A	A	B	A
Naphtha	A	A	C	B	C
Jet Fuels	A	A	C	B	C
Saltwater, 160° F	A	A	C	B	A
Crude Oil	A	B	C	B	C
Gasoline	B	B	C	C	C

A= Excellent **B= Moderate** **C= Poor**

Source: Manufacturer's Literature

XR-5 data based on conditions detailed in Section 3, Part 1.

Part 3: Weathering Resistance

Accelerated Weathering Test

XR-5 has been tested in the carbon arc weatherometer for over 10,000 hours of exposure and in the Xenon weatherometer for over 12,000 hours of exposure. The sample showed no loss in flexibility and no significant color change. Based on field experience of Seaman Corporation products and similar weatherometer exposure tests, XR-5 should have an outdoor weathering life significantly longer than competitive geomembranes, particularly in tropical or subtropical applications.

EMMAQUA Testing: ASTM E-838-81 was performed on a modified form of XR-5, FiberTite, used in the single-ply roofing industry. After 3 million Langleys in Arizona, no signs of degradation were noted with no evidence of cracking, blistering, swelling or adhesion delamination failure of the coating.

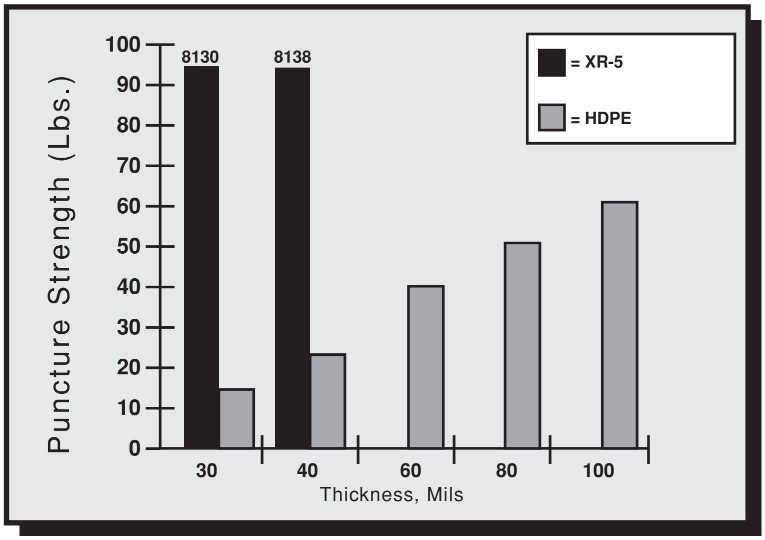
Natural Exposure

After over 17 years as a holding basin at a large oil company in the Texas desert, XR-5 showed no signs of environmental stress cracking, thermal expansion/contraction, or low yield strength problems. Temperature ranges from near zero to over 100º F.

In service approximately 17 years in a solar pond application at a research facility in Ohio, UV exposed samples, as well as immersed samples, retained over 90% of the tensile strength. Examination of the material determined there was little effect on the coating compound. The solar pond was exposed to temperatures from below zero to over 100° F.

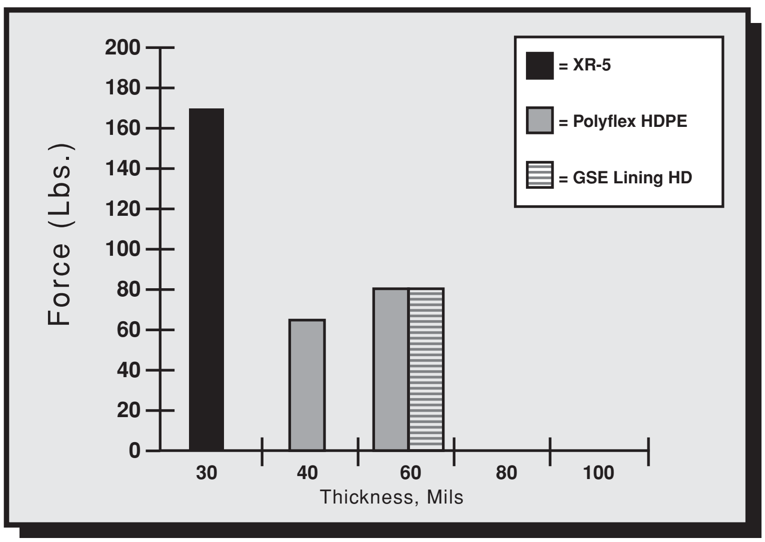
XR5 was exposed for 12½ years in Sarasota, Florida, on a weathering rack, facing the southern direction at 45°. No significant color loss, cracking, crazing, blistering, or adhesion delamination failure of the coating was noted.

XR-5/HDPE Comparative Properties

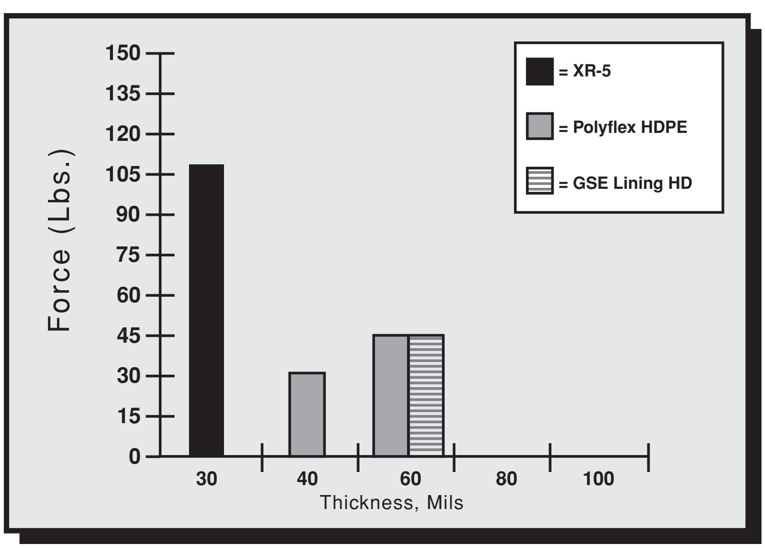


Puncture Resistance

1. ASTM D 751, Screwdriver Tip, 45° Angle (Room Temperature) Puncture Resistance, XR5 vs. HDPE



2. FED-STD-101C Method 2065 (Room Temperature)*



3. FED-STD-101C Method 2065 (70°C)*

* Data provided by E.I. DuPont de Nemours & Co. Wilmington, Delaware

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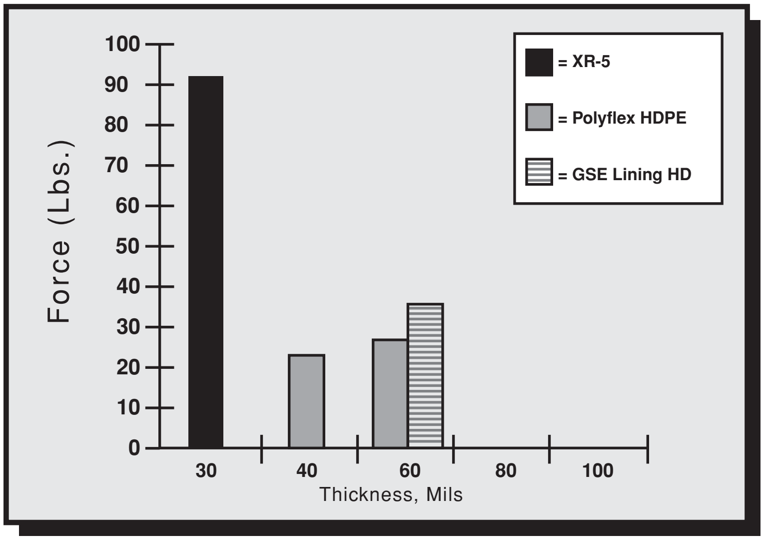
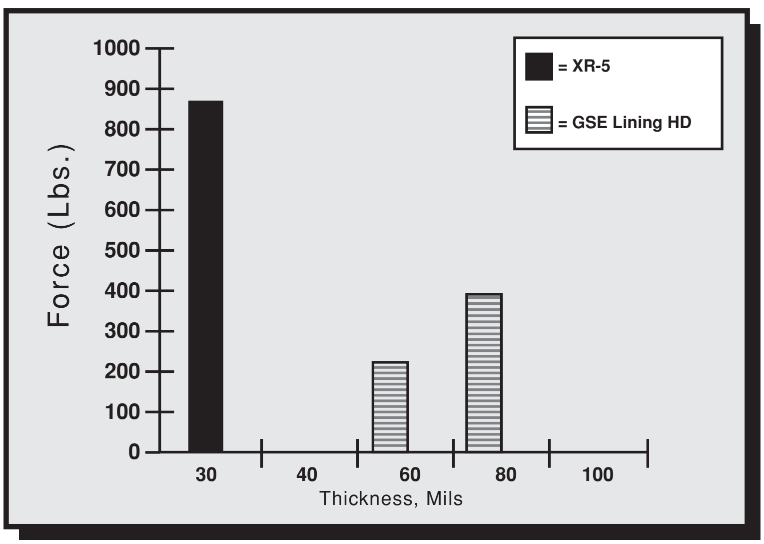
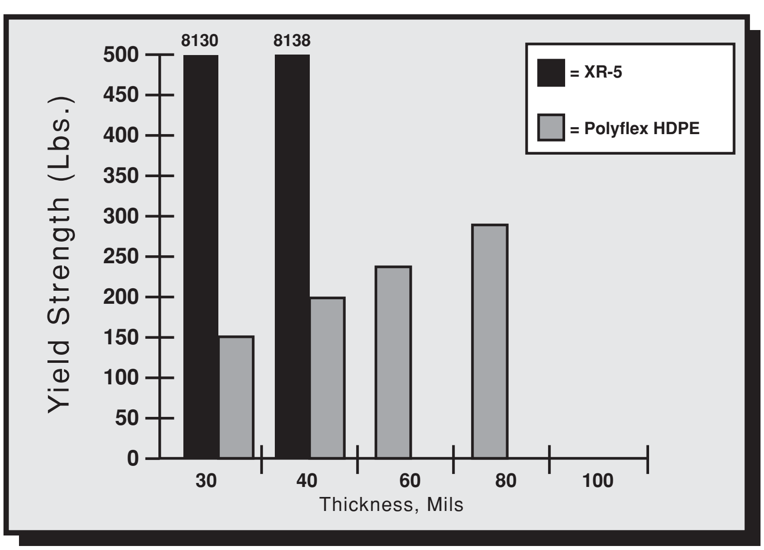


4. FED-STD-101C Method 2065 (100°C)*



5. ASTM D 751 Ball Burst Puncture

Yield Strength



1. Yield Strength, XR-5 vs. HDPE

Test Method: Grab Tensile, ASTM D 751, 70° C

* Data provided by E.I. DuPont de Nemours & Co. Wilmington, Delaware

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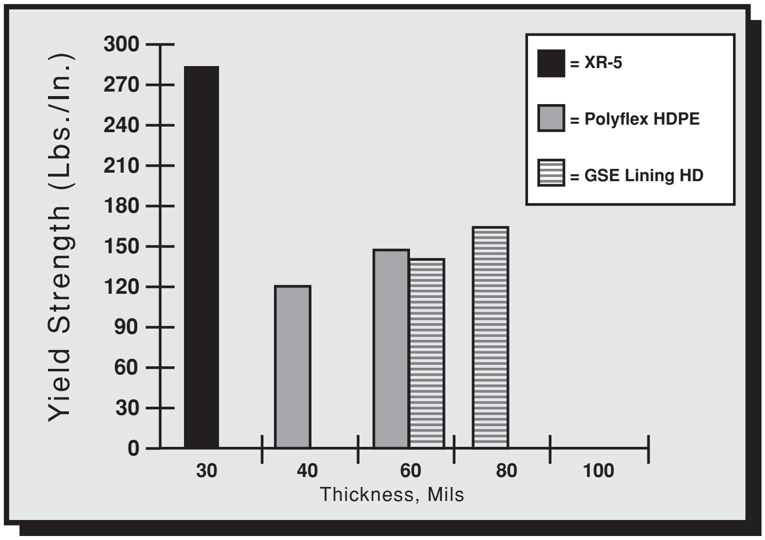
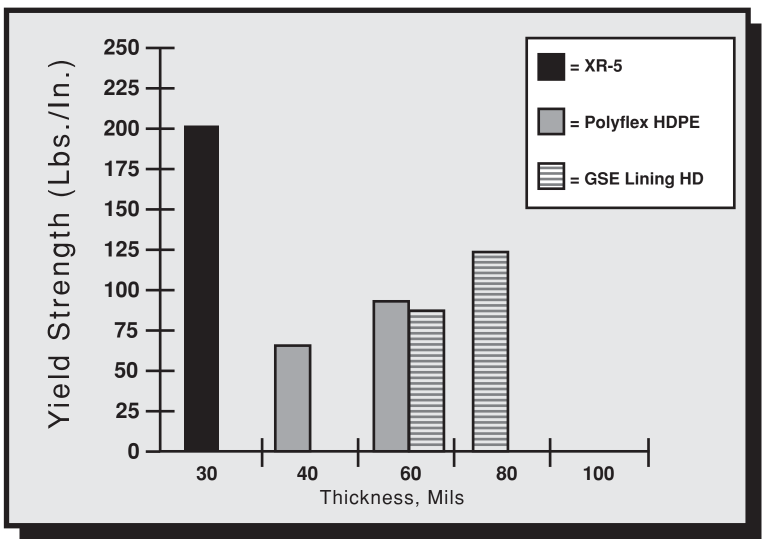
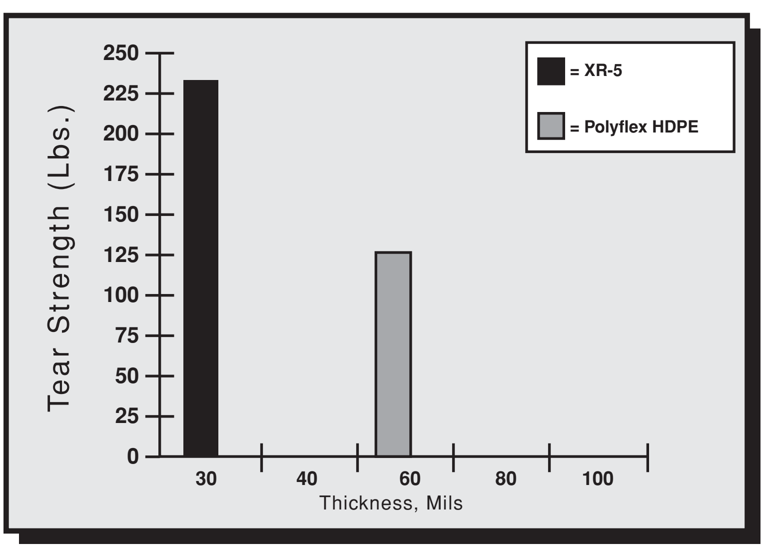


2. Strip Tensile, ASTM D 751, Room Temperature*



3. Strip tensile, ASTM D 751, 70ºC*

Tear Strength



1. Tongue Tear (8" x 10" Specimens), ASTM D 751, Room Temperature*

* Data provided by E.I. DuPont de Nemours & Co. Wilmington, Delaware

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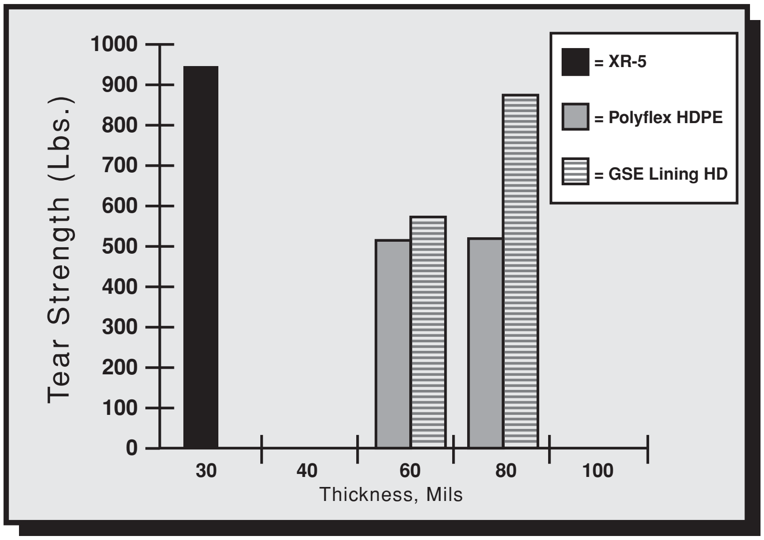
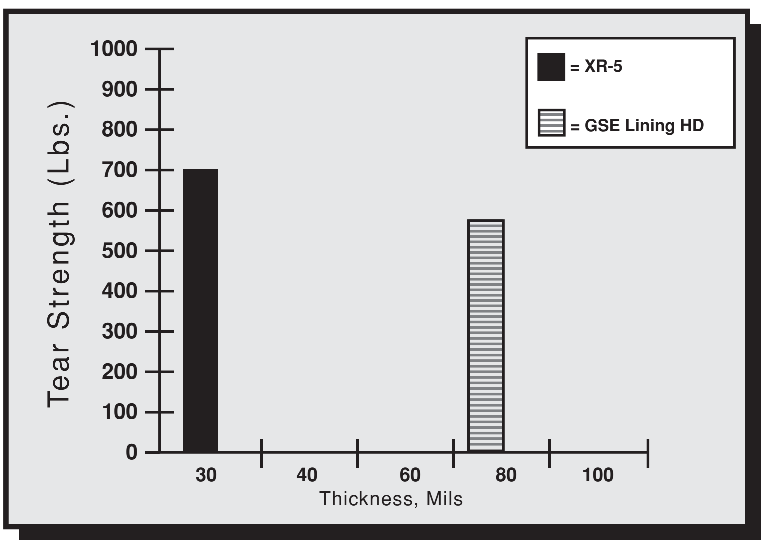


1. Graves Tear, ASTM D 624, Die C, Room Temperature*

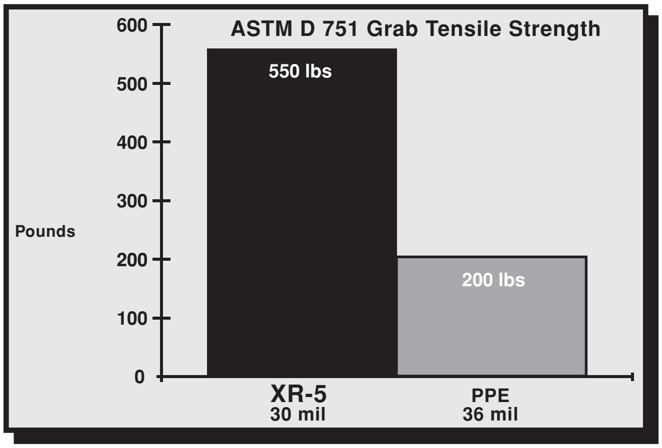


2. Graves Tear, ASTM D 624, Die C, 70°C*

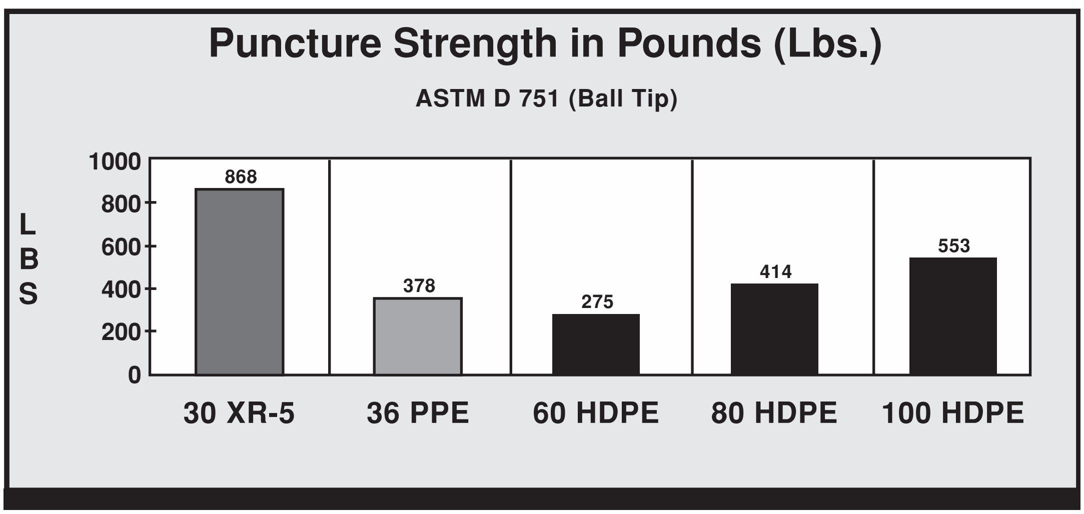
* Data provided by E.I. DuPont de Nemours & Co. Wilmington, Delaware

GSE is a registered trademark of GSE Lining Technology, Inc.

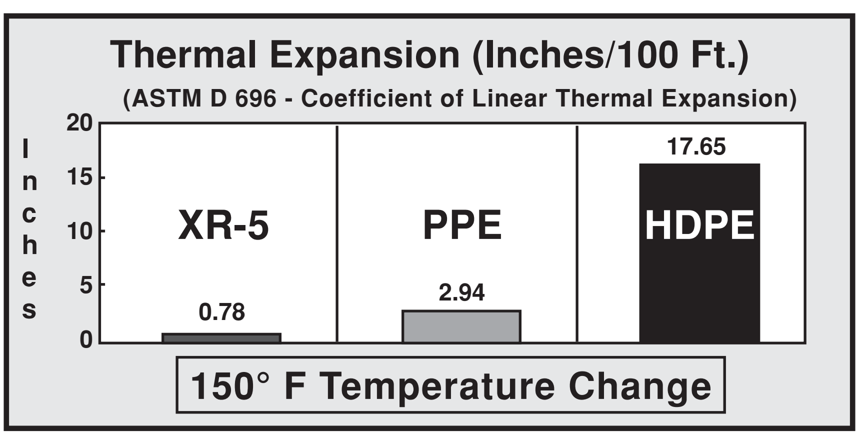
Grab Strength – XR-5® vs. Polypropylene Tensile



Puncture Strength Comparison



Coated Fabric Thermal Stability



Specification For Geomembrane Liner

(Sample specification: 8130 XR-5®. For other product specifications, go to www.xr-5.com)

General

1.01 Scope Of Work
Furnish and install flexible membrane lining in the areas shown on the drawings. All work shall be done in strict accordance with the project drawings, these specifications and membrane lining fabricator's approved shop drawings.

Geomembrane panels will be supplied sufficient to cover all areas, including appurtenances, as required in the project, and shown on the drawings. The fabricator/installer of the liner shall allow for shrinkage and wrinkling of the field panels.

1.02 Products
The lining material shall be 8130 XR-5 as manufactured by Seaman Corporation (1000 Venture Boulevard, Wooster, OH 44691; 330-262-1111), with the following physical specifications:

Base- (Type)	Polyester
Fabric Weight (ASTM D 751)	6.5 oz./sq. yd.
Finished Coated Weight (ASTM D 751)	30 ± 2 oz./sq. yd.
Trapezoid Tear (ASTM D 751)	40/55 lbs. min.
Grab Yield Tensile (ASTM D 751, Grab Method Procedure A)	550/550 lbs. min.
Elongation @ Yield (%)	20% min.
Adhesion- Heat Seam (ASTM D 751, Dielectric Weld)	40 lbs./2in. weld min.
Adhesion- Ply (ASTM D 413, Type A)	15 lbs./in. or film tearing bond
Hydrostatic Resistance (ASTM D 751, Method A)	800 psi min.
Puncture Resistance (ASTM D 4833)	275 lbs. min.
Bursting Strength (ASTM D 751 Ball Tip)	750 lbs. min.
Dead Load (ASTM D 751) Room Temperature	220 lbs. min.
(2" overlap seam, 4 hours) 160ºF	120 lbs. min.
Bonded Seam Strength (ASTM D 751 Grab Test Method, Procedure A)	575 lbs. min.
Low Temperature (ASTM D 2136, 4 hours- 1/8" Mandrel)	Pass @ -30ºF
Weathering Resistance ASTM G 153 Carbon Arc	8,000 hours min. With no appreciable changes or stiffening or cracking of coating
Dimensional Stability (ASTM D 1204, 212ºF 1 Hour, Each Direction)	0.5% max.
Water Absorption (ASTM D 471, 7 Days)	0.025 kg/m^2 max. @ 70ºF 0.14 kg/m^2 max. @ 212ºF
Abrasion Resistance ASTM D 3389,	2000 cycles before fabric exposure;
H-18 Wheel, 1000 g load	50 mg/100 cycles max. wgt. Loss
Coefficient of Thermal Expansion/Contraction (ASTM D 696)	8 x 10^{-6} in/in/º F max.

1.03 Submittals
The fabricator of panels used in this work shall prepare shop drawings with a proposed panel layout to cover the liner area shown in the project plans. Shop drawings shall indicate the direction of factory seams and shall show panel sizes consistent with the material quantity requirements of 1.01.

Details shall be included to show the termination of the panels at the perimeter of lined areas, the methods of sealing around penetrations, and methods of anchoring.

Placement of the lining shall not commence until the shop drawings and details have been approved by the owner, or his representative.

1.04 Factory Fabrication
The individual XR-5® liner widths shall be factory fabricated into large sheets custom designed for this project so as to minimize field seaming. The number of factory seams must exceed the number of field seams by a factor of at least 10.

A two-inch overlap seam done by heat or RF welding is recommended. The surface of the welded areas must be dry and clean. Pressure must be applied to the full width of the seam on the top and bottom surface while the welded area is still in a melt-type condition. The bottom welding surface must be flat to insure that the entire seam is welded properly. Enough heat shall be applied in the welding process that a visible bead is extruded from both edges being welded. The bead insures that the material is in a melt condition and a successful chemical bond between the two surfaces is accomplished.

Two-inch overlapped seams must withstand a minimum of 240 pounds per inch width dead load at 70° F. and 120 pounds per inch width at 160° F. as outlined in ASTM D 751. All seams must exceed 550 lbs. bonded seam strength per ASTM D 751 Bonded Seam Strength Grab Test Method, Procedure A.

1.05 Inspection And Testing Of Factory Seams
The fabricator shall monitor each linear foot of seam as it is produced. Upon discovery of any defective seam, the fabricator shall stop production of panels used in this work and shall repair the seam, and determine and rectify the cause of the defect prior to continuation of the seaming process.

The fabricator must provide a Quality Control procedure to the owner or his representative which details his method of visual inspection and periodic system checks to ensure leak-proof factory fabrication.

1.06 Certification and Test Reports
Prior to installation of the panels, the fabricator shall provide the owner, or his representative, with written certification that the factory seams were inspected in accordance with Section 1.05.

1.07 Panel Packaging and Storage
Factory fabricated panels shall be accordian-folded, or rolled, onto a sturdy wooden pallet designed to be moved by a forklift or similar equipment. Each factory fabricated panel shall be prominently and indelibly marked with the panel size. Panels shall be protected as necessary to prevent damage to the panel during shipment.

Panels which have been delivered to the project site shall be stored in a dry area.

1.08 Qualifications of Suppliers
The fabricator of the lining shall be experienced in the installation of flexible membrane lining, and shall provide the owner or his representative with a list of not less than five (5) projects and not less than 500,000 square feet of successfully installed XR-5 synthetic lining. The project list shall show the name, address, and telephone number of an appropriate party to contact in each case. The manufacturer of the sheet goods shall provide similar documentation with a 10 million square foot minimum, with at least 5 projects demonstrating 10+ years service life.

The installer shall provide similar documentation to that required by the fabricator.

1.09 Subgrade Preparation By Others
Lining installation shall not begin until a proper base has been prepared to accept the membrane lining. Base material shall be free from angular rocks, roots, grass and vegetation. Foreign materials and protrusions shall be removed, and all cracks and voids shall be filled and the surface made level, or uniformly sloping as indicated

on the drawings. The prepared surface shall be free from loose earth, rocks, rubble and other foreign matter. Generally, no rock or other object larger than USCS sand (SP) should remain on the subgrade in order to provide an adequate safety factor against puncture. Geotextiles may be used to compensate for irregular subgrades. The subgrade shall be uniformly compacted to ensure against settlement. The surface on which the lining is to be placed shall be maintained in a firm, clean, dry and smooth condition during lining installation.

1.10 Lining Installation
Prior to placement of the liner, the installer will indicate in writing to the owner or his representative that he believes the subgrade to be adequately prepared for the liner placement.

The lining shall be placed over the prepared surface in such a manner as to assure minimum handling. The sheets shall be of such lengths and widths and shall be placed in such a manner as to minimize field seaming.

In areas where wind is prevalent, lining installation should be started at the upwind side of the project and proceed downwind. The leading edge of the liner shall be secured at all times with sandbags or other means sufficient to hold it down during high winds.

Sandbags or rubber tires may be used as required to hold down the lining in position during installation. Materials, equipment or other items shall not be dragged across the surface of the liner, or be allowed to slide down slopes on the lining. All parties walking or working upon the lining material shall wear soft-sole shoes.

Lining sheets shall be closely fit and sealed around inlets, outlets and other projections through the lining. Lining to concrete seals shall be made with a mechanical anchor, or as shown on the drawings. All piping, structures and other projections through the lining shall be sealed with approved sealing methods.

1.11 XR-5 Field Seaming
All requirements of Section 1.04 and 1.05 apply. A visible bead should be extruded from the hot air welding process.

Field fabrication of lining material will not be allowed.

1.12 Inspection
All field seams will be tested using the Air Lance Method. A compressed air source will deliver 55 psi minimum to a 3/16 inch nozzle. The nozzle will be directed to the lip of the field seam in a near perpendicular direction to the length of the field seam. The nozzle will be held 4 inches maximum from the seam and travel at a rate not to exceed 40 feet per minute. Any loose flaps of 1/8" or greater will require a repair.

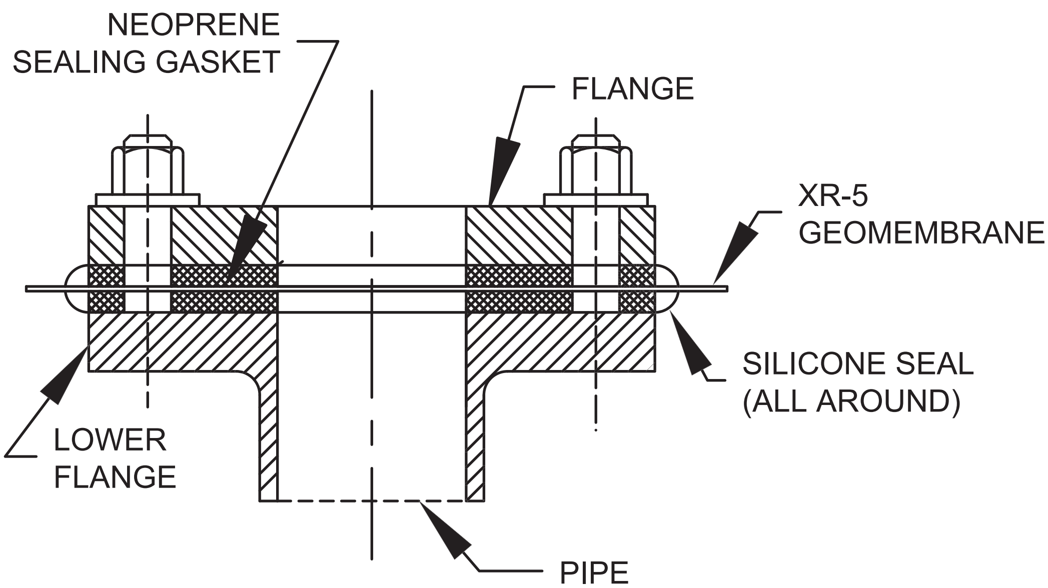
Alternatively all field seams should also be inspected utilizing the Vacuum Box Technique as described in Standard Practice for Geomembrane Seam Evaluation by Vacuum Chamber (ASTM D 5641-94 (2006)), using a 3 to 5 psi vacuum pressure. All leaks shall be repaired and tested.

All joints, on completion of work, shall be tightly bonded. Any lining surface showing injury due to scuffing, penetration by foreign objects, or distress from rough subgrade, shall as directed by the owner or his representative be replaced or covered, and sealed with an additional layer of lining of the proper size, in accordance with the patching procedure.

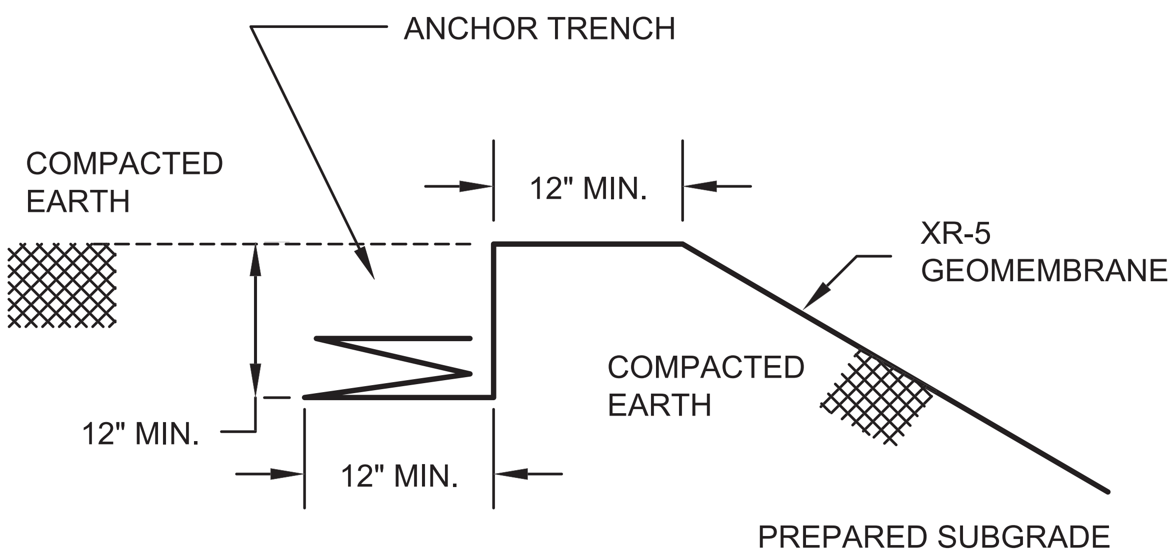
1.13 Patching
Any repairs to the lining shall be patched with the lining material. The patch material shall have rounded corners and shall extend a minimum of four inches (4") in each direction from the damaged area.

Seam repairs or seams which are questionable should be cap stripped with a 1" wide (min.) strip of the liner material. The requirements of Section 1.11 apply to this cap stripping.

1.14 Warranty
The lining material shall be warranted on a pro-rated basis for 10 years against both weathering and chemical compatibility in accordance with Seaman Corporation warranty for XR-5® Style 8130. A test immersion will be performed by the owner and the samples evaluated by the manufacturer. Workmanship of installation shall be warranted for one year on a 100% basis.



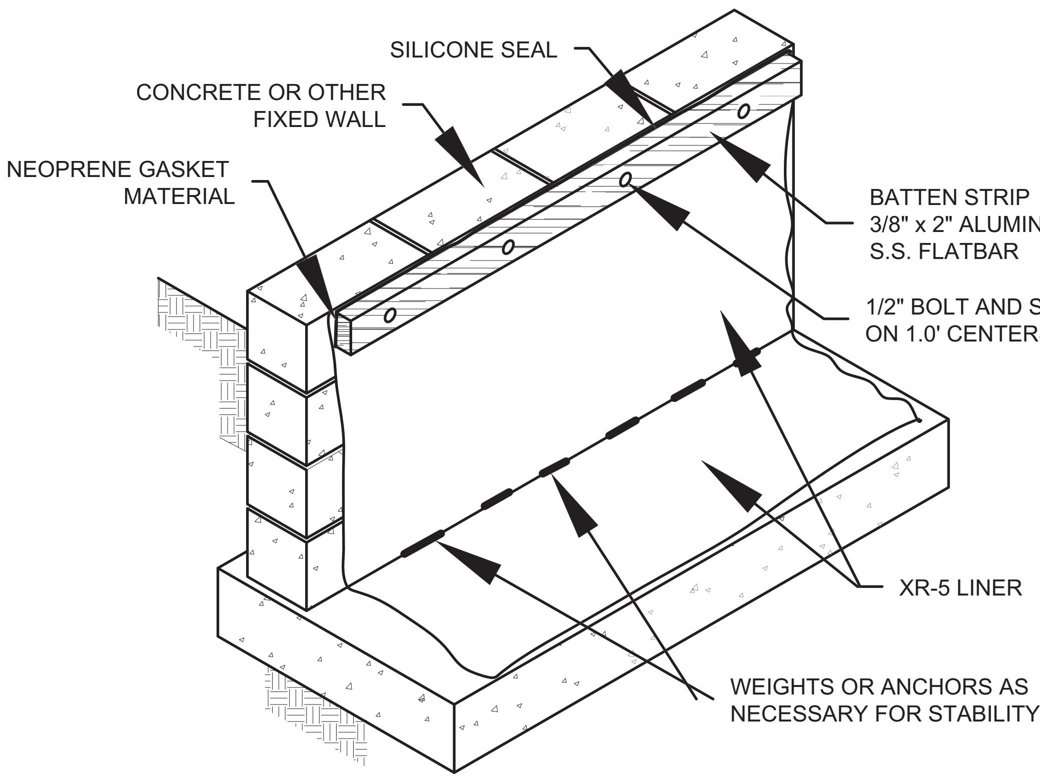
Seaman Corporation ENGINEERED PRODUCTS GROUP 1000 Venture Blvd.,Wooster, Ohio 44691	
FLANGE CONNECTION TO PIPE SECTION	
SCALE: NONE	
SHEET 1 of 1	DRAW NO. XRD-019



Seaman Corporation
ENGINEERED PRODUCTS GROUP
1000 Venture Blvd.,Wooster, Ohio 44691

ELEVATION VIEW
TYPICAL ANCHOR DETAILS
XR-5 LINER

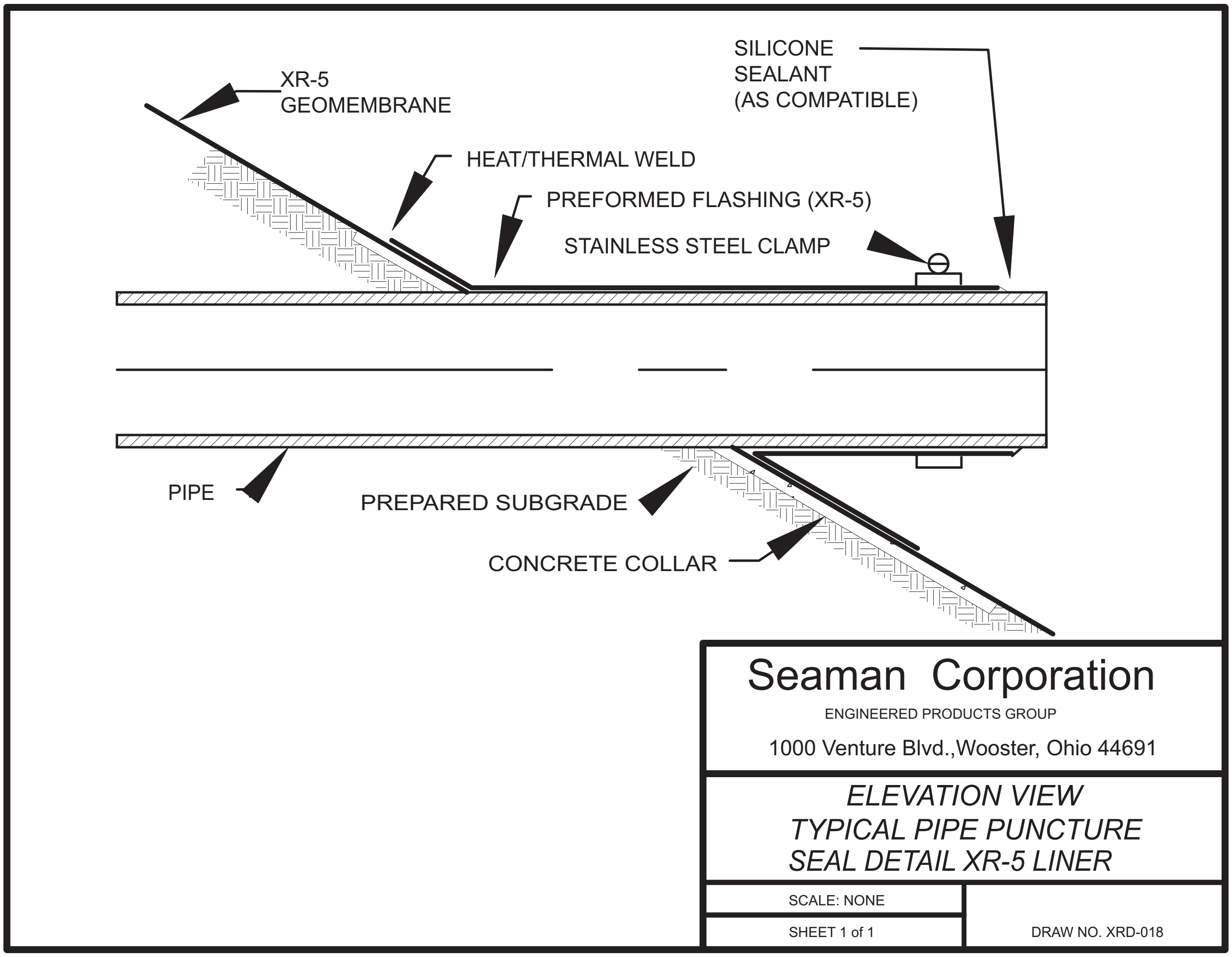
SCALE: NONE	
SHEET 1 of 1	DRAW NO. XRD-001



Seaman Corporation
ENGINEERED PRODUCTS GROUP
1000 Venture Blvd.,Wooster, Ohio 44691

ANCHORING DETAIL
XR-5 LINER TO FIXED WALL

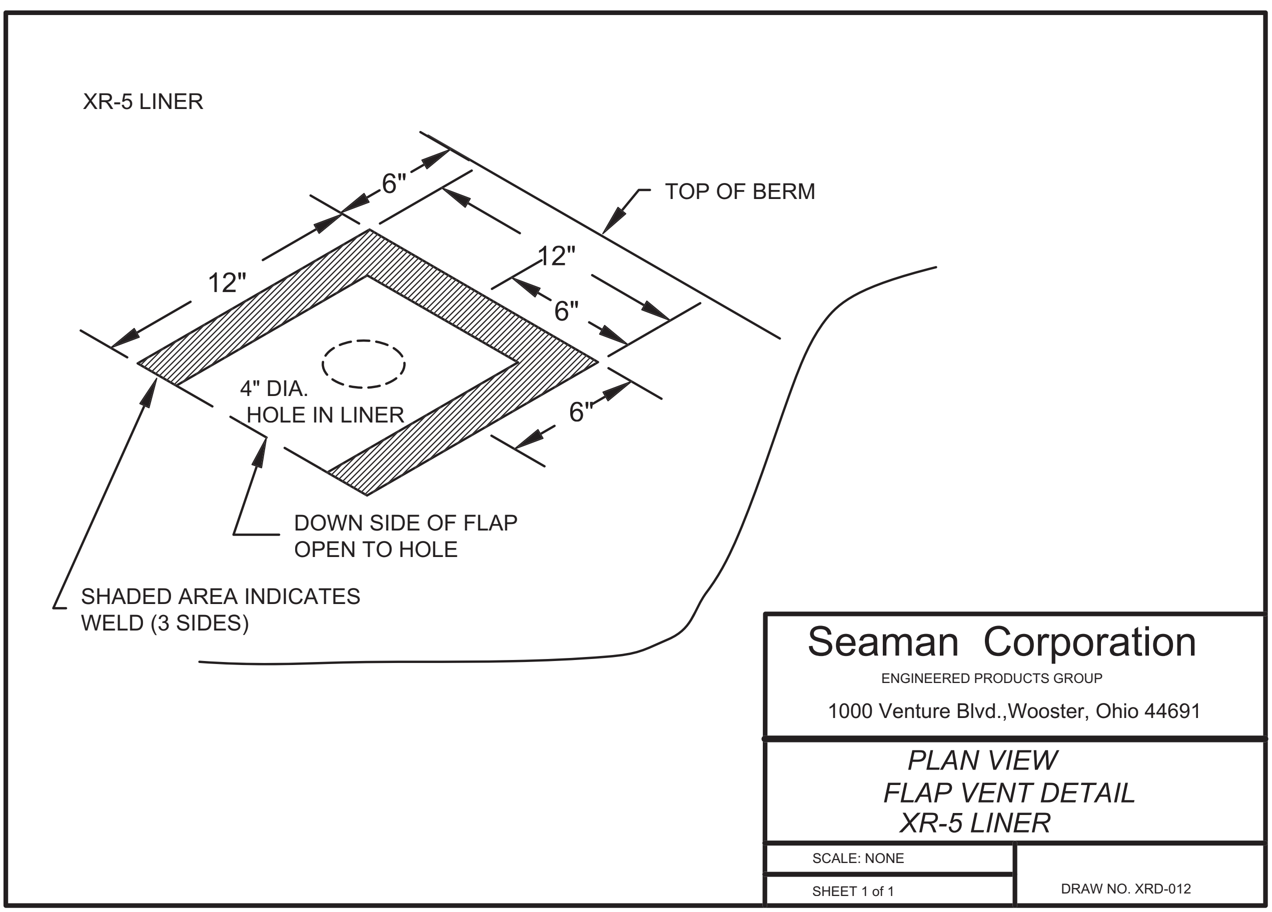
SCALE: NONE
SHEET 1 of 1
DRAW NO. XRD-023



Seaman Corporation
ENGINEERED PRODUCTS GROUP
1000 Venture Blvd.,Wooster, Ohio 44691

ELEVATION VIEW
TYPICAL PIPE PUNCTURE
SEAL DETAIL XR-5 LINER

SCALE: NONE
SHEET 1 of 1
DRAW NO. XRD-018



Seaman Corporation
ENGINEERED PRODUCTS GROUP
1000 Venture Blvd.,Wooster, Ohio 44691

PLAN VIEW
FLAP VENT DETAIL
XR-5 LINER

SCALE: NONE
SHEET 1 of 1
DRAW NO. XRD-012

Section 6 - Warranty Information

Warranty

XR-5® is offered with Seaman Corporation standard warranty which addresses weathering and chemical compatibility for a 10-year period. A test immersion is required with subsequent testing and approval by Seaman Corporation.

Instructions for XR-5 Test Immersions and Warranty Requests

1. Completely immerse six Style 8130 XR-5 samples (8-1/2" x 11" size) in the liquid to be contained.

2. At the end of approximately thirty days, retrieve three of the samples. The samples should be rinsed with fresh water and dried.

3. Send the three samples to:
   Attn: Geomembrane Department
   Seaman Corporation
   1000 Venture Blvd.
   Wooster, OH 44691

4. Keep the other three samples immersed until further notice in case longer immersion data is required.

5. Complete and return the information form on the liner application.

8228 XR-3® and all PW Geomembranes are offered with a standard 10-year warranty for weathering. The attached information form should be completed.

XR® Membrane Application and Utilization Form

Installation Owner and Address:

Physical Location of Installation:

Expected Date of Installation:

Expected Beginning Date of Service:

Description of Application:
(Example: impoundment used to contain brine on an emergency basis.)

Physical Features of Application:
(Example: 1.3 million gallon earthen impoundment with overall top dimensions of 160’ x 160’ with 3:1 slopes and 10’ deep.)

Description of Liquid:

(Describe content of liquid including pollutants and expected temperature extremes in basin and at application point. Attach analysis of liquid chemistry, composition taken on a representative basis.)

Operational Characteristics:

(Describe the operation of the facility such as filling schedules, fluctuating liquid levels, operating temperatures, etc.)

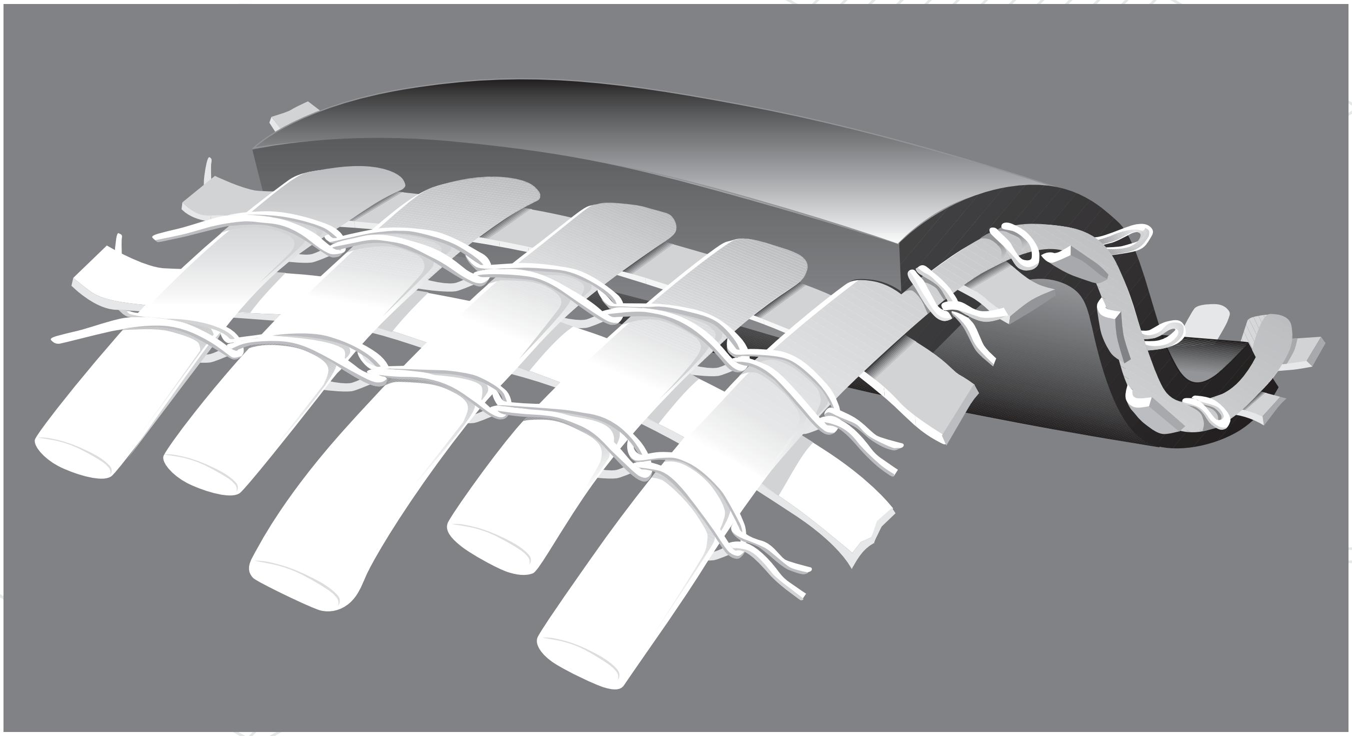
Performance Requirements, Etc:

(State any other requirements, such as rate of permeability required.)

Owner represents the information herein is complete and accurate, and understands and agrees that issuance of Seaman Corporation Warranty for XR products are conditioned upon such completeness and accuracy.

OWNER'S SIGNATURE

Reference Materials:



XR-5®: High Performance Composite Geomembrane

Seaman Corporation
1000 Venture Blvd.
Wooster, Ohio 44691
(330) 262-1111
www.xr-5.com

ATTACHMENT III.1.I

SMOOTH HDPE GEOMEMBRANE

SMOOTH HDPE GEOMEMBRANE
ENGLISH UNITS

Property	Test Method	Minimum Average Values 30 mil	40 mil	60 mil	80 mil	100 mil
Thickness, mils	ASTM D 5199					
minimum average		30	40	60	80	100
lowest individual reading		27	36	54	72	90
Sheet Density, g/cc	ASTM D 1505/D 792	0.940	0.940	0.940	0.940	0.940
Tensile Properties[1]	ASTM D 6693					
1. Yield Strength, lb/in		63	84	126	168	210
2. Break Strength, lb/in		114	152	228	304	380
3. Yield Elongation, %		12	12	12	12	12
4. Break Elongation, %		700	700	700	700	700
Tear Resistance, lb	ASTM D 1004	21	28	42	56	70
Puncture Resistance, lb	ASTM D 4833	54	72	108	144	180
Stress Crack Resistance[2], hrs	ASTM D 5397 (App.)	300	300	300	300	300
Carbon Black Content[3], %	ASTM D 1603	2.0 - 3.0	2.0 - 3.0	2.0 - 3.0	2.0 - 3.0	2.0 - 3.0
Carbon Black Dispersion	ASTM D 5596			--Note 4--		
Oxidative Induction Time (OIT) Standard OIT, minutes	ASTM D 3895	100	100	100	100	100
Oven Aging at 85°C High Pressure OIT - % retained after 90 days	ASTM D 5721 ASTM D 5885	60	60	60	60	60
UV Resistance[5] High Pressure OIT[6] - % retained after 1600 hrs	GRI GM11 ASTM D 5885	50	50	50	50	50
Seam Properties	ASTM D 6392 (@ 2 in/min)					
1. Shear Strength, lb/in		57	80	120	160	200
2. Peel Strength, lb/in - Hot Wedge		45	60	91	121	151
- Extrusion Fillet		39	52	78	104	130
Roll Dimensions						
1. Width (feet):		23	23	23	23	23
2. Length (feet)		1000	750	500	375	300
3. Area (square feet):		23,000	17,250	11,500	8,625	6,900
4. Gross weight (pounds, approx.)		3,470	3,470	3,470	3,470	3,470

1 Machine direction (MD) and cross machine direction (XMD) average values should be on the basis of 5 test specimens each direction. Yield elongation is calculated using a gauge length of 1.3 inches; Break elongation is calculated using a gauge length of 2.0 inches.
2 The yield stress used to calculate the applied load for the SP-NCTL test should be the mean value via MQC testing.
3 Other methods such as ASTM D 4218 or microwave methods are acceptable if an appropriate correlation can be established.
4 Carbon black dispersion for 10 different views: Nine in Categories 1 and 2 with one allowed in Category 3.
5 The condition of the test should be 20 hr. UV cycle at 75°C followed by 4 hr. condensation at 60°C.
6 UV resistance is based on percent retained value regardless of the original HP-OIT value.

ATTACHMENT III.1.J

COMPUTER AIDED EARTHMOVING SYSTEM

Computer Aided Earthmoving System

CAES for Landfills

Landfill Compactors
Track-Type Tractors
Wheel Tractor Scrapers
Motor Graders

System Components	
Communications Radio	TC900B
GPS Antenna	L1/L2
GPS Receiver	MS840
In-Cab Display	CAES Touch Screen Display
CAESoffice™/METSmanager	

Computer Aided Earthmoving System for Landfills

Advanced GPS technologies for earthmoving equipment improve machine efficiency, maximize air space utilization, and extend landfill life.

Caterpillar is helping customers revolutionize the way they compact trash, grade slopes and manage their operation with new technology solutions for landfills. Solutions that provide greater accuracy, higher productivity, lower operating costs, more profitability and longer landfill life.

The Computer Aided Earthmoving System (CAES) is a high technology earthmoving tool that allows machine operators to achieve maximum landfill compaction, desired grade/slope, and conserve and ensure even distribution of valuable cover soil with increased accuracy without the use of traditional survey stakes and crews. Using global positioning system (GPS) technology, machine-mounted components, a radio network, and office management software, this state-of-the-art machine control system delivers real-time elevation, compaction and grade control information to machine operators on an in-cab display. By monitoring grade and compaction progress, operators have the information they need to maximize the efficiency of the machine, resulting in proper drainage and optimum airspace utilization.

This advanced technology tool also aids in the identification of site-specific storage areas for hazardous, medical, industrial, and organic waste requiring special handling and placement records.

Applications

CAES is an ideal tool for landfill planning, engineering, surveying, grade control, and production monitoring applications in dump areas. CAES is specifically designed for use on landfill compactors, track-type tractors, wheel tractor scrapers, and motor graders.

On-Board Components

- CAES Touch Screen Display
- GPS Receiver
- GPS Antenna (L1/L2)
- Communications Radio

Off-Board Components

- GPS Reference Station
- Radio Network
- CAESoffice/METSmanager

Operation

CAES uses GPS technology, a wireless radio communications network, and office software to map landfills, create site plans, locate a machine's position, and track compaction and earthmoving progress with complete accuracy.

The receiver uses signals from GPS satellites to determine precise machine positioning. Two receivers are used to capture and collect satellite data – one located at a stationary spot on the landfill site, and another located on the machine. Signals from the ground-based reference station and on-board computer are used to remove errors in satellite measurements for centimeter accuracy.

The CAES-enabled machine is driven over the site to create a digital terrain design file. Using the radio network and office software, landfill terrain data is transmitted from the machine to the landfill office. Landfill managers can then send the work plan from the office to the in-cab display to show operators the work to be done.

The in-cab display provides the operator with an overhead and cross-sectional three-dimensional surface view of the color-coded work plan and precise machine location. The software continuously updates terrain and machine position information as the machine traverses the site.

CAES gives the operator the ability to control grade by monitoring progress on the in-cab display, which shows a graphical representation of lift thickness and compaction density. Cut/fill numbers are displayed in real-time as the machine moves across the site, which allows the operator to know precise elevation, material spread, compaction passes, and required cut or fill at any point on the job.

The *compactor* display shows colored grids representing the number of compaction passes the machine has made across each area. As the compactor wheel travels over an area, the screen changes color to acknowledge the pass. Green areas indicate when optimum compaction has been reached. The system also monitors thick lift information and visually displays when a lift exceeds maximum site parameters.

In *tractor, scraper and motor grader* applications, the color display graphically shows the operator cut, fill, and grade work to be done according to plan. As the machine works, the screen changes color. Green indicates when the operator has achieved plan grade.

By providing immediate feedback on the accuracy of each pass, CAES operators have the information and confidence they need to work more efficiently, productively and profitably.

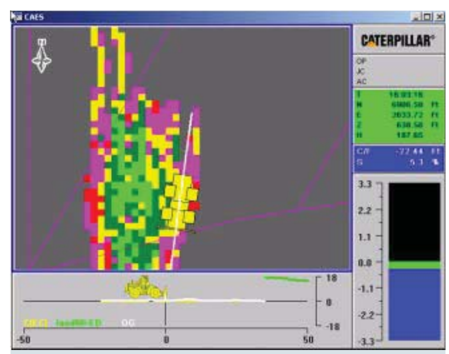
On-Board Components

Communications Radio. The rugged radio, mounted on the roof of the machine, is used for transmitting, repeating and receiving real-time data from GPS receivers. The radio broadcasts real-time, high-precision data for GPS applications. Under normal conditions, the 900 MHz radio broadcasts data up to 10 km (6.2 miles) line-of-sight. Coverage can be enhanced with a network of repeaters, which allows coverage over a broader area. Optimized for GPS with increased sensitivity and jamming immunity, the radio features error correction and high-speed data transfer, ensuring optimum performance. A 450 MHz radio solution is also available.

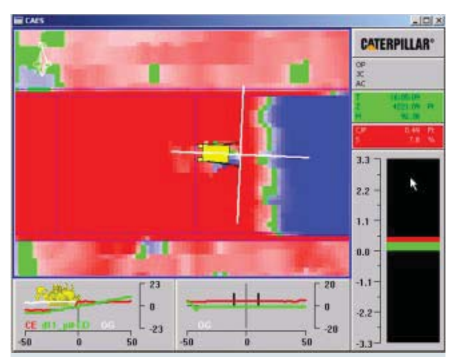
GPS Antenna (L1/L2). The dual frequency external antenna, mounted on the roof of the machine and reference station, is used to pick up the signals from the GPS satellites to determine the machine's position for high precision, real-time machine guidance and control. A low-noise amplifier provides sensitive performance in demanding applications. The compact, low profile design and sealed housing ensure reliable performance in harsh weather conditions.

GPS Receiver. The dual frequency real-time kinematic (RTK) GPS receiver is used to send and receive data simultaneously across the radio network. The system computes differential corrections for real-time positioning with centimeter accuracies, to ensure precise machine guidance and control.

CAES Touch Screen Display. The in-cab graphical display provides real-time operating information to the operator. Designed for simple operation, the 264 mm (10.4 in) custom configurable, integrated touch screen display allows operators to easily interface with the CAES system. The display utilizes the latest infrared touch and transflective backlight technology for superior viewing in bright light conditions and a broad-range dimmable backlight for viewing in low light conditions. Designed for reliable performance in extreme operating conditions, the unit is guarded against shock and sealed to keep out dust and moisture.



Compactor Screen



Dozer Screen

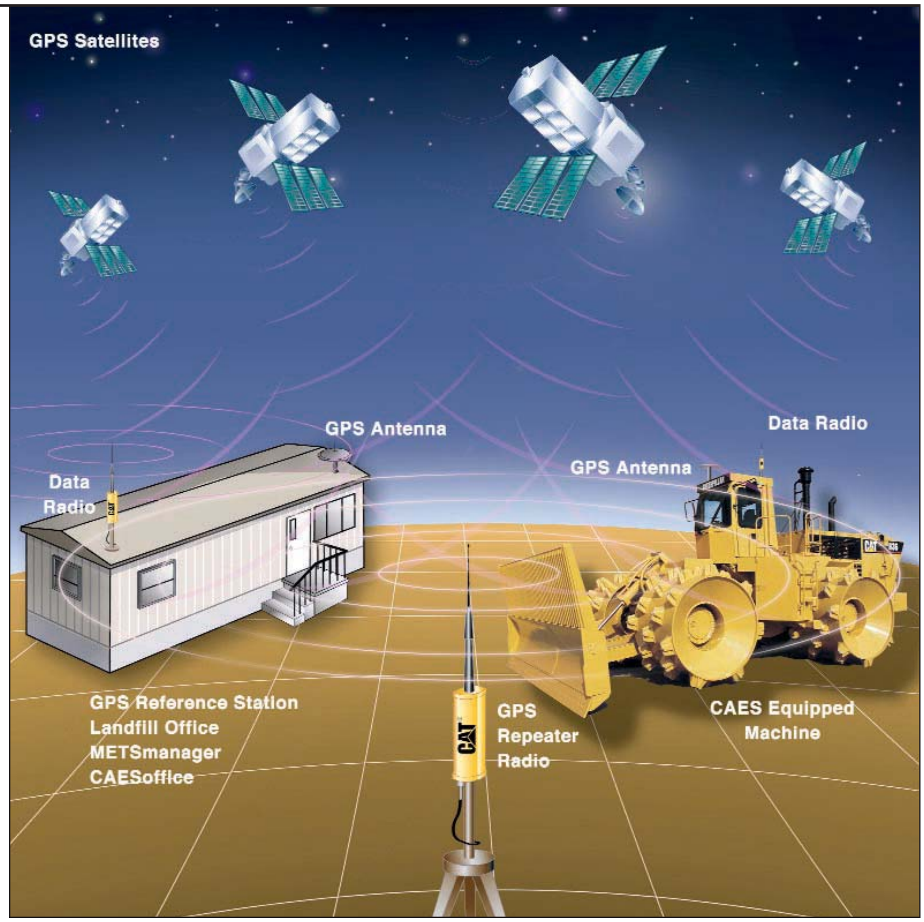
Off-Board Components

GPS Technology. Global Positioning System (GPS) technology uses 24+ satellites that orbit above the earth and constantly transmit their positions, identities and times of signal broadcasts to earth-based satellite sensors. The GPS receiver is an electronic box, which measures the distance to each visible satellite from an antenna on the ground. Through trilateralization, the receiver determines where the satellite is in respect to the center of the earth. The GPS receiver uses its own position and GPS satellite positions to calculate errors and corrections for computing exact location and precise positioning with centimeter accuracy.

GPS Reference Station. A GPS reference station is used to achieve the centimeter level accuracy needed in a landfill application. The reference station sends GPS information over a radio link to the GPS receiver on the CAES-enabled machine. The receiver combines the information with its own observations to compute precise positioning.

Radio Network. The radio network for CAES has two channels. GPS correction data is transmitted over one channel, while the other channel is used to send site planning and production data to the machine and from the machine back to the site office. By utilizing the same radio as a repeater the range can be extended to provide seamless coverage around local obstacles such as hills or large buildings. Up to four radio repeaters may be used to provide extended coverage.

Landfill Planning Software. Site planning and surveying begins with the landfill planning software. CAES is compatible with most third party CAD planning software packages. Data formats used between the CAES software and the planning software are industry standard .DXF and ASCII.



CAESoffice™. The powerful Caterpillar-designed CAESoffice software enables landfill management to monitor CAES-equipped machines and work progress throughout the site in near real-time. The data is stored in a database format for easy customized access, reporting and editing.

METSmanager. This software package allows for integration of the landfill planning system and the machine. It provides the user interface for CAES and controls all communications over the wireless radio network. METSmanager reads design files in standard .DXF formats, converts them to CAES format (.CAT), and sends the design files to the on-board display on the machine over the radio network. This program continually updates the site model by regularly requesting data transmissions from the machine to the office.

- **File Window.** Displays design files (.DXF) created using the site planning package, and holds application configuration files for GPS receivers and files converted from .DXF to the CAES on-board software format (.CAT).
- **Machines Window.** Shows icons of each machine equipped with CAES on-board software. Allows multiple machines to be monitored at the same time.
- **Messages Window.** Contains a list of recent error, warning, confirmation, or information messages generated by METSmanager.
- **Communications Queue Window.** Lists all file transmissions scheduled to occur over the radio network and displays transmission status for all files.

Specifications

TC900B Communications Radio
- Technology: Spread spectrum
- Modes: Base, repeater, rover
- Optimal Range: 10 km (6 miles), line-of-sight
- Typical Range: 3-5 km (2-3 miles) varies w/terrain and operating conditions. Repeaters may be used to extend range
- Frequency Range: 902-928 MHz
- Networks: Ten, user selectable
- Transmit Power: Meets FCC requirements, 1 watt max.
- License Free (U.S. and Canada)
- Wireless Data Rates: 128 Kbps[2]
- Operating Temperature: –40° C to 70° C (–40° F to 158° F)
- Storage Temperature: –40° C to 85° C (–40° F to 185° F)
- Humidity: 100%
- Sealing: Exceeds MIL-STD-810E, sealed to ±34.5 kPa (±5 psi), immersible to 1 m (39 in)
- Vibration: 8 gRMS, 20-2000 Hz
- Operational Shock: ±40 g, 10 msec
- Survival Shock: ±75 g, 6 msec
- Electrical Input: 10.5 to 20V DC
- Nominal Current: 250 mA (3 W)1
- Transmit Current: 1000 mA (12 W)1
- Protection: Reverse polarity
- Control Interface: SAE J1939 CAN
- Emissions and Susceptibility: CE compliant, exceeds ISO 13766
- Input Connector: 8-pin
- Network Connector: 8-pin
- Height: 250 mm (10 in)
- Width: 85 mm (3.4 in)
- Weight: 0.9 kg (2.0 lb)

Radios outside of U.S. and Canada operate on different frequencies. Please contact your Cat Dealer for specifics.

L1/L2 GPS Antenna
- Operating Temperature: –40° C to 70° C (–40° F to 158° F)
- Storage Temperature: –55° C to 85° C (–67° F to 185° F)
- Height: 151mm (6 in)
- Width: 330 mm (13 in)
- Depth: 72 mm (2.8 in)
- Weight: 1.695 kg (3.8 lb)

MS840 GPS Receiver
- Tracking: 9 channels L1 C/A code, L1/L2 full cycle carrier, fully operational during P-code encryption
- Signal Processing: Supertrak multibit technology, Everest multipath suppression
- Positioning Mode –
- Synchronized RTK: 1 cm + 2 ppm horizontal accuracy/2 cm + 2 ppm vertical accuracy, 300 ms latency, 5 Hz std. maximum rate
- Low Latency: 2 cm + 2 ppm horizontal accuracy/3 cm + 2 ppm vertical accuracy, <20 ms latency, 20 Hz maximum rate
- DPGS: <1m accuracy, <20 ms latency, 20 Hz maximum rate
- Range: Up to 20 km from base for RTK
- Communication: 3x RS-232 ports, baud rates up to 115,200
- Control Interface: SAE J1939 CAN
- Configuration: RS-232 Serial connection
- Operating Temperature: –20° C to 60° C (–4° F to 140° F)
- Storage Temperature: –30° C to 80° C (–22° F to 176° F)
- Humidity: 100%
- Operational Vibration: 3 gRMS
- Survival Vibration: 6.2 gRMS
- Operational Shock: ±40 g
- Survival Shock: ±75 g
- Electrical Input: 12/24V DC, 9 watts
- Height: 5.1 cm (2.0 in)
- Width: 14.5 cm (5.7 in)
- Depth: 23.9 cm (9.4 in)
- Weight: 1.0 kg (2.25 lb)

CAES Touch Screen Display
- LCD Display: 264 mm (10.4 in) 640 × 480 transflective color VGA
- Buttons: touch screen
- Touch Screen: 3.17 mm (0.125 in) resolution infrared high light rejection
- Back Light: 200 cd/m2, 200:1 dimming ratio
- Processor: Intel Pentium CPU
- Memory: 64 MB Ram
- Solid State Disk: Internal 128 MB, external compact flash
- Operating Environment: Embedded WinNT
- Operating Temperature: –20° C to 70° C (–4° F to 158° F)
- Storage Temperature: –50° C to 85° C (–58° F to 185° F)
- Sealing: IP68 sealed to ±5 psi
- Humidity: 100%
- Electrical Input: 9-32V DC
- Power Supply: 5 amp @ 40W load dump, reverse voltage, ESD, over voltage protection
- Connector: 70-pin
- Discrete I/O: 8 digital ports; 5 PMW inputs
- Mounting: bracket or panel
- Height: 261 mm (10.28 in)
- Width: 315 mm (12.4 in)
- Depth: 93 mm (3.66 in)
- Weight: 3.17 kg (8.5 lb)

CAESoffice/METSmanager PC Requirements
- Pentium II/III processor w/ 128 MB memory
- 21 in. monitor (SVGA color 1024 × 768 resolution) with 2MB video memory
- Windows NT 4.0 or higher with latest service pack
- Modem- internal or external (required for remote support)
- Required ports: serial (suggest 2 serial, 1 parallel)
- CD ROM drive
- 3.5 in disk drive
- Mouse or suitable pointing device
- Hard Drive Space: 200 MB min.

Customer Support. For over 25 years, Caterpillar has been providing electronic and electrical components and systems for the earthmoving industry – real world technology solutions that enhance the value of Cat products and make customers more productive and profitable. Your Cat Dealer is ready to assist you with matching machine systems to the application or obtaining responsible, knowledgeable support. For additional information, please contact us at LANDFILLGPS@CAT.com

Computer Aided Earthmoving System for Landfills

Landfill Compactors
Track-Type Tractors
Wheel Tractor Scrapers
Motor Graders

www.CAT.com

AEHQ5549 (9-03)

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1.0 INTRODUCTION

Lea Land LLC (the Facility) is an existing Surface Waste Management Facility (SWMF) providing oil field waste solids (OFWS) disposal services. The existing Lea Land SWMF is subject to regulation under the New Mexico Oil and Gas Rules, specifically 19.15.9.711 and 19.15.36 NMAC, administered by the Oil Conservation Division (OCD) of the NM Energy, Minerals, and Natural Resources Department (NMEMNRD). This document is a component of the “Application for Permit Modification” that proposes continued operations of the existing approved waste disposal unit; lateral and vertical expansion of the landfill via the construction of new double-lined cells; and the addition of waste processing capabilities. The proposed Facility is designed in compliance with 19.15.36 NMAC, and will be constructed and operated in compliance with a Surface Waste Management Facility Permit issued by the OCD. The Facility is owned by, and will be constructed and operated by, Lea Land LLC.

The Lea Land SWMF is one of the most recently designed facilities to meet the new more stringent standards that, for instance, mandate double liners and leak detection for land disposal. The new services that Lea Land will provide needed resources to fill an existing void in the market for technologies that exceed current OCD requirements.

The existing Lea Land Landfill is equipped with a composite liner design with an inclined leachate collection geopipe system and extraction point in the northeast corner. Liner Installation Records and Engineering Certification/CQA Reports document that the liner segments were constructed in compliance with current industry and engineering standards. Routine attempts to monitor and collect leachate flow from “Unit I” have demonstrated that oil field waste solids do not generate fluids, as no free liquids are allowed, and does not produce water.

1.1 Site Location

The Lea Land site is located approximately 27 miles northeast of Carlsbad, straddling US Highway 62-180 (Highway 62) in Lea County, NM. The Lea Land site is comprised of a 642-acre ± tract of land encompassing Section 32, Township 20 South, Range 32 East, Lea County, NM. Site access is currently provided on the south side of US Highway 62. The coordinates for the approximate center of the Lea Land site are Latitude 32°31’46.77” and Longitude -103°47’18.25”.

1.2 Facility Description

The Lea Land SWMF comprises approximately 463 acres± of the 642-acre ± site, and will include two main components: an oil field waste Processing Area and an oil field waste solids Landfill, as well as related infrastructure (i.e., access, waste receiving, stormwater management, etc.). Oil field wastes are delivered to the Lea Land SWMF from oil and gas exploration and production operations in southeastern NM and west Texas. The Permit Plans (**Attachment III.1.A**) identify the locations of the Processing Area and Landfill Disposal facilities. The proposed facilities are detailed in **Table II.1.2** (**Volume II.1**), and are anticipated to be developed in four primary phases as described in **Table II.1.3** (**Volume II.1**).

2.0 LANDFILL VOLUMETRIC CALCULATIONS

Landfill volumetric calculations were completed for the Lea Land SWMF Landfill corresponding to the design shown on the **Permit Plans** (**Volume III.1**). Landfill volumetric calculations include waste capacity analysis and the soil material balance. The capacity analysis for the Lea Land SWMF Landfill is presented in **Table III.2.1**. The minimum gross airspace computed for the balance of Unit I and for Units II - V is approximately 14,626,216 cubic yards (yd^3); with approximately 12,520,079 yd^3 (12,520,079 tons assuming an average waste density of 2,000 lbs/yd^3) of net airspace (i.e., minimum waste capacity). The projected longevity is approximately 96.3 years assuming 500 tons per day (tpd) incoming waste volume; 48.5 years assuming 1,000 tpd incoming waste volume and 32.1 years assuming 1,500 tpd incoming waste volume. A materials balance was also completed for the Landfill and is presented in **Table III.2.2**. Lea Land has more than sufficient soils from on-site excavations for the protective soil layer, daily and intermediate cover soils, and final cover for the balance of Unit I and for Units II - V.

TABLE III.2.1 - Capacity Analysis

Description	Plan Fill Area (± acres)	Gross Airspace (yd^3)	Cover[2] (yd^3)	Waste Capacity[3,4] Airspace (yd^3)	Waste Capacity Airspace (tons)[4]	Longevity Estimate (years)[5,6]		
						@ 500 tpd	@ 1,000 tpd	@ 1,500 tpd
Unit I (future balance)	20.0	1,024,316	181,120	843,197	843,197	6.5	3.2	2.2
Units II - V	70.0	13,602,384	1,925,502	11,676,882	11,676,882	89.8	44.9	29.9
Landfill Total	**90.0**	**14,626,700**	**2,106,622**	**12,520,079**	**12,520,079**	**96.3**	**48.2**	**32.1**

Notes:

1. *The calculations presented in this table provide the proposed capacity and longevity for the site. Estimated waste rates include stabilized and solidified materials from the Processing Area, and are subject to change.*
2. *yd^3 = cubic yards. Cover includes protective soil cover, daily, and intermediate cover; and final cover (collectively called total cover soil) [see **Table III.2.2**].*
3. *Waste capacity airspace = (gross airspace - cover soils); see **Table III.2.2**.*
4. *In-place waste density: Oil Field Waste = 2,000 lbs/yd^3; [tons = ((waste capacity airspace (yd^3) x in-place waste density)/2,000 lbs/ton)]. 1 yd^3 = 1 ton.*
5. *Longevity = [waste capacity airspace (tons)/daily incoming waste rate (tons/day)] / (260 operating days/year).*
6. *Tons per day = tpd (based on 5 days per week).*

TABLE III.2.2 - Materials Balance (yd^3)

Description	Surface Fill Area (± acres)	Protective Soil/Drainage Layer[1]	Cover Soil[2] (Daily & Intermediate)	Final Cover[3,6]	Total Soil Required[4]	Excavation Volume	Soil Balance
Unit I (future balance)	20.0	0	84,320	96,800	181,120	0	-181,120
Units II - V	70.0	225,867	1,297,431	402,204	1,925,502	2,867,965	942,463
Landfill Total	**90.0**	**225,867**	**1,381,751**	**499,004**	**2,106,622**	**2,867,965**	**761,343**

Notes:

1. *Volume of protective soil layer assumes 2-foot depth over future liner area.*
2. *Cover Soil for Landfill assumes approximately 10% of effective airspace (gross airspace - protective soil/drainage layer - final cover).*
3. *Volume of final cover conservatively assumes 3-foot depth over lined area (6 inch erosion layer and 30 inch protective layer).*
4. *Includes protective cover/drainage layer soil, cover soil, and final cover.*

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1.0 INTRODUCTION

Lea Land LLC (the Facility) is an existing Surface Waste Management Facility (SWMF) providing oil field waste solids (OFWS) disposal services. The existing Lea Land SWMF is subject to regulation under the New Mexico Oil and Gas Rules, specifically 19.15.9.711 and 19.15.36 NMAC, administered by the Oil Conservation Division (OCD) of the NM Energy, Minerals, and Natural Resources Department (NMEMNRD). This document is a component of the “Application for Permit Modification” that proposes continued operations of the existing approved waste disposal unit; lateral and vertical expansion of the landfill via the construction of new double-lined cells; and the addition of waste processing capabilities. The proposed Facility is designed in compliance with 19.15.36 NMAC, and will be constructed and operated in compliance with a Surface Waste Management Facility Permit issued by the OCD. The Facility is owned by, and will be constructed and operated by, Lea Land LLC.

The Lea Land SWMF is one of the most recently designed facilities to meet the new more stringent standards that, for instance, mandate double liners and leak detection for land disposal. The new services that Lea Land will provide needed resources to fill an existing void in the market for technologies that exceed current OCD requirements.

1.1 Site Location

The Lea Land site is located approximately 27 miles northeast of Carlsbad, straddling US Highway 62-180 (Highway 62) in Lea County, NM. The Lea Land site is comprised of a 642-acre ± tract of land encompassing Section 32, Township 20 South, Range 32 East, Lea County, NM. Site access is currently provided on the south side of US Highway 62. The coordinates for the approximate center of the Lea Land site are Latitude 32°31’46.77” and Longitude -103°47’18.25”.

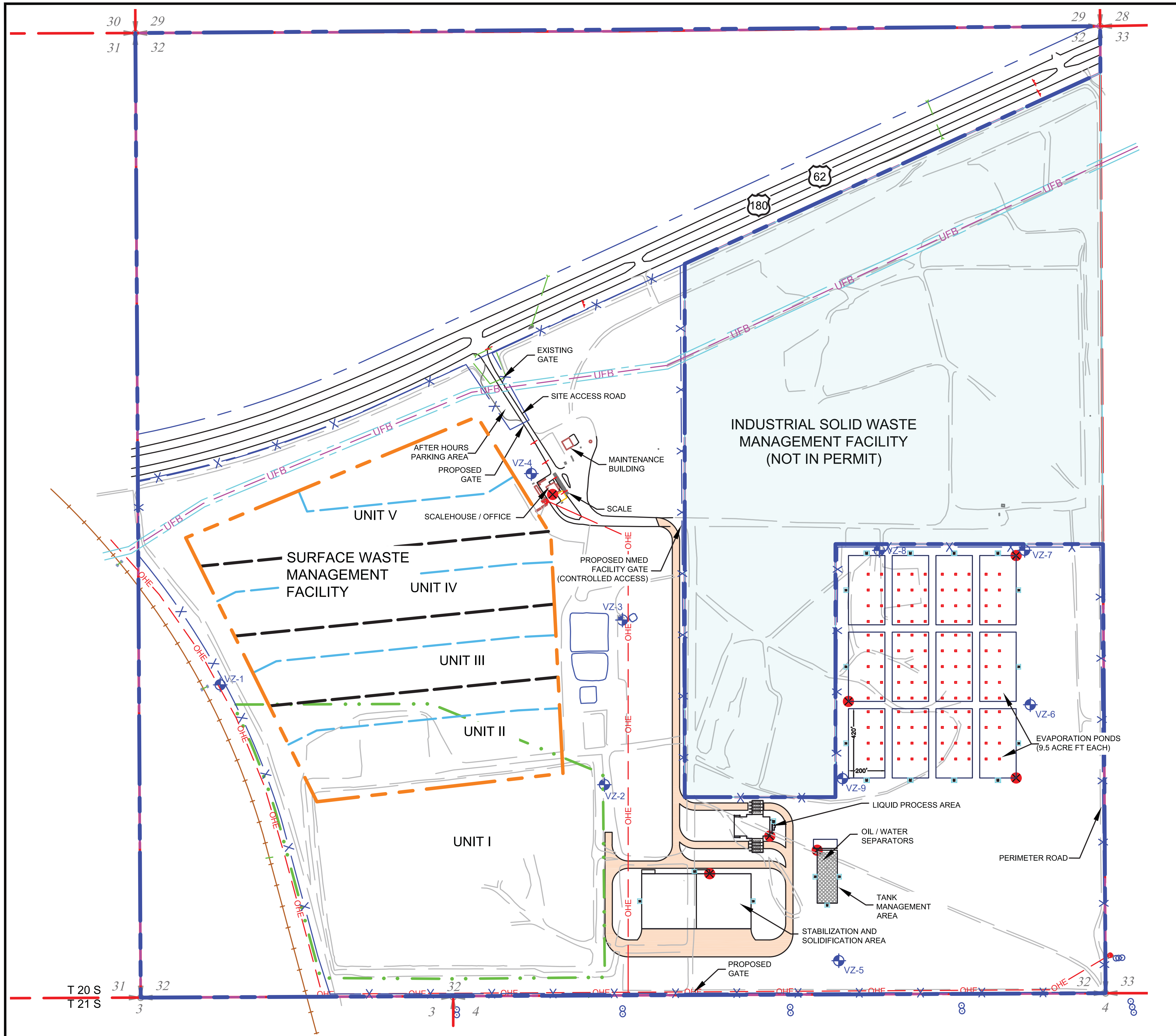
1.2 Facility Description

The Lea Land SWMF comprises approximately 463 acres ± of the 642-acre ± site, and will include two main components: an oil field waste Processing Area and an oil field waste solids Landfill, as well as related infrastructure (i.e., access, waste receiving, stormwater management, etc.). Oil field wastes are delivered to the Lea Land SWMF from oil and gas exploration and production operations in southeastern NM and west Texas. The Site Plan provided as **Figure III.3.1** identify the locations of the Processing Area and Land Disposal facilities, which are further detailed on the **Permit Plans** (**Attachment III.1.A**). The proposed facilities are detailed in **Table II.1.2** (**Volume II.1**), and are anticipated to be developed in four primary phases as described in **Table II.1.3** (**Volume II.1**).

1.3 Interim Drainage Plan

While the design shown on the Permit Plans primarily addresses landfill completion conditions, it is anticipated that site development will take place as a sequence of "Units" (generally south to north) that each consist of a sequential "Cells" (generally east to west). Interim drainage during initial site development captures and controls run-on that enters the site from the south and west. "Unit II" (see **Permit Plans**, **Volume III.1**) excavation includes construction of berms along portions of the north, east and west landfill boundary to divert stormwater away from the first unit excavation and to provide a lined eastern bulwark for the planned landfill stormwater runoff evaporation ponds. Within a Unit, there will be temporary perimeter Cell stormwater berms that serve to separate stormwater from leachate. As a Unit is developed, base grade elevations remain below the site natural flow paths.

Pumping will be required to evacuate the accumulated stormwater within the excavation to the stormwater evaporation basins. As units and cells develop along with the site perimeter channel systems, the retention and detention basins will be developed incrementally as the operation progresses. Channel configurations and temporary stormwater and erosion control measures that may be implemented during interim construction are shown on the **Permit Plans** (**Volume III.1**). The permanent stormwater management designs, including planned lined and unlined retention/detention basins, are not anticipated to be necessary for decades into the future (see Volumetric Analysis, **Volume III.2**). All interim (temporary) and permanent (Landfill Completion Plan) installations will be subject to routine maintenance and silt removal.



LEGEND

- SECTION BOUNDARY
- PROPERTY BOUNDARY
- SURFACE WASTE MANAGEMENT BOUNDARY
- EXISTING OCD PERMIT BOUNDARY
- PROPOSED SURFACE WASTE MANAGEMENT BOUNDARY
- INDUSTRIAL SOLID WASTE FACILITY BOUNDARY
- CELL BOUNDARY
- EXISTING PAVED ROAD
- EXISTING UNPAVED ROAD
- EXISTING FENCE
- PROPOSED FENCE
- EXISTING RAILROAD
- PROPOSED UNDERGROUND FIBER OPTIC
- PROPOSED OVERHEAD POWER LINE
- EXISTING CULVERT
- PROPOSED UNPAVED ROAD (GRAVEL)
- NOT IN OCD PERMIT
- EXISTING TRANSMISSION LINE
- H_2S MONITORING LOCATION
- PROPOSED VADOSE ZONE MONITORING WELL
- PPE AND EMERGENCY EQUIPMENT

NORTH

0 300' 600'

SITE PLAN

LEA LAND LLC SURFACE WASTE MANAGEMENT FACILITY
LEA COUNTY, NEW MEXICO

GORDON ENVIRONMENTAL PSC
333 Rio Rancho Blvd. NE
Rio Rancho, NM 87124
Phone: 505-867-6990
Fax: 505-867-6991

DATE: 03/27/2019	CAD: SITE PLAN.dwg	PROJECT #: 0416.18
DRAWN BY: DMI	REVIEWED BY: CRK	FIGURE III.3.1
APPROVED BY: CWF	www.team-psc.com	

Drawing:X:\2018\0416.18\02_DSGN\02_DWG\050_CIVIL\02_CONTENT\PERMIT FIGURES\RAI1\SITE PLAN.dwg
Date/Time:Jun. 27, 2019-09:12:40 ; LAYOUT: B (LS)

2.0 DESIGN CRITERIA

The stormwater management systems for the Lea Land SWMF Landfill and Processing Facility are designed to meet the requirements of the regulatory standards identified in the New Mexico Oil Conservation Department Rules 19.15.36 NMAC. More specifically, closure standards in 19.15.36.13.M specifies:

Each operator shall have a plan to control run-on water onto the site and runoff water from the site, such that:

(1) the run-on and runoff control system shall prevent flow onto the surface waste management facility's active portion during the peak discharge from a 25-year storm; and

(2) runoff from the surface waste management facility's active portion shall not be allowed to discharge a pollutant to the waters of the state or United States that violates state water quality standards.

19.15.36.18.D(2)(a) NMAC requires:

"…soil contoured to promote drainage of precipitation…" and "...prevent the ponding of water…"

3.0 METHODOLOGY

The methodology for the calculation of runoff stormwater flows is based on the guidelines set forth in the New Mexico State Highway and Transportation Department (NMSHTD) Drainage Manual, Section 400: Hydrology (Philips et al., 2018; **Attachment III.3.A**). The total enclosed drainage basin acreage for the project area is determined to be all ±642 acres of Section 32, with an additional ±243 acres of run-on area, totaling ±885 acres or ±1-3/8 square miles (**Figure III.3.1**). Based on the selection criteria for basins of this size and use, the NMSHTD Drainage Manual specifies that the SCS Unit Hydrograph Method, or TR-20, is to be used. The standard government route to utilize the computational model TR-20 is to use the US Army Corps of Engineers' HEC-HMS software. The Engineer has elected to utilize *Autodesk® Inc.'s Storm and Sanitary Analysis* software package to run the TR-20 model computational analysis, as it uses the TR-20 model, among others, and builds on the functionality offered by the Army Corps' HEC-HMS software.

TR-20 is a computerized model for estimating the peak rate of runoff and runoff volume from small to medium watersheds. Infiltration and other losses are estimated using the SCS Curve Number (CN) methodology while Time of Concentration is calculated using the SCS TR-55 iterative method. The TR-20 Method is limited to single basins less than 5 square miles in area and is to be used when the Time of Concentration (T_c) is expected not to exceed 8.0 hours; and where channels will be used to convey runoff. Lea Land meets these criteria, at ±885 acres (i.e., ±1-3/8 square miles) with appropriate channelization.

In addition to modelling the pre-development condition with the SCS TR-20 Method's hydrology model, and calculating SCS TR-55 time-of-concentration, *Autodesk® Inc.'s Storm and Sanitary Analysis* software package was used to analyze existing incidental and deliberate storage areas. The same method and model software package was used in modelling the post-development conditions' runoff and run-on areas, and in an iterative process for projecting the effects and sizing of the run-on collection network including drainage channels and stormwater basins.

4.0 SURFACE WATER CALCULATIONS – PRE-DEVELOPMENT CONDITION

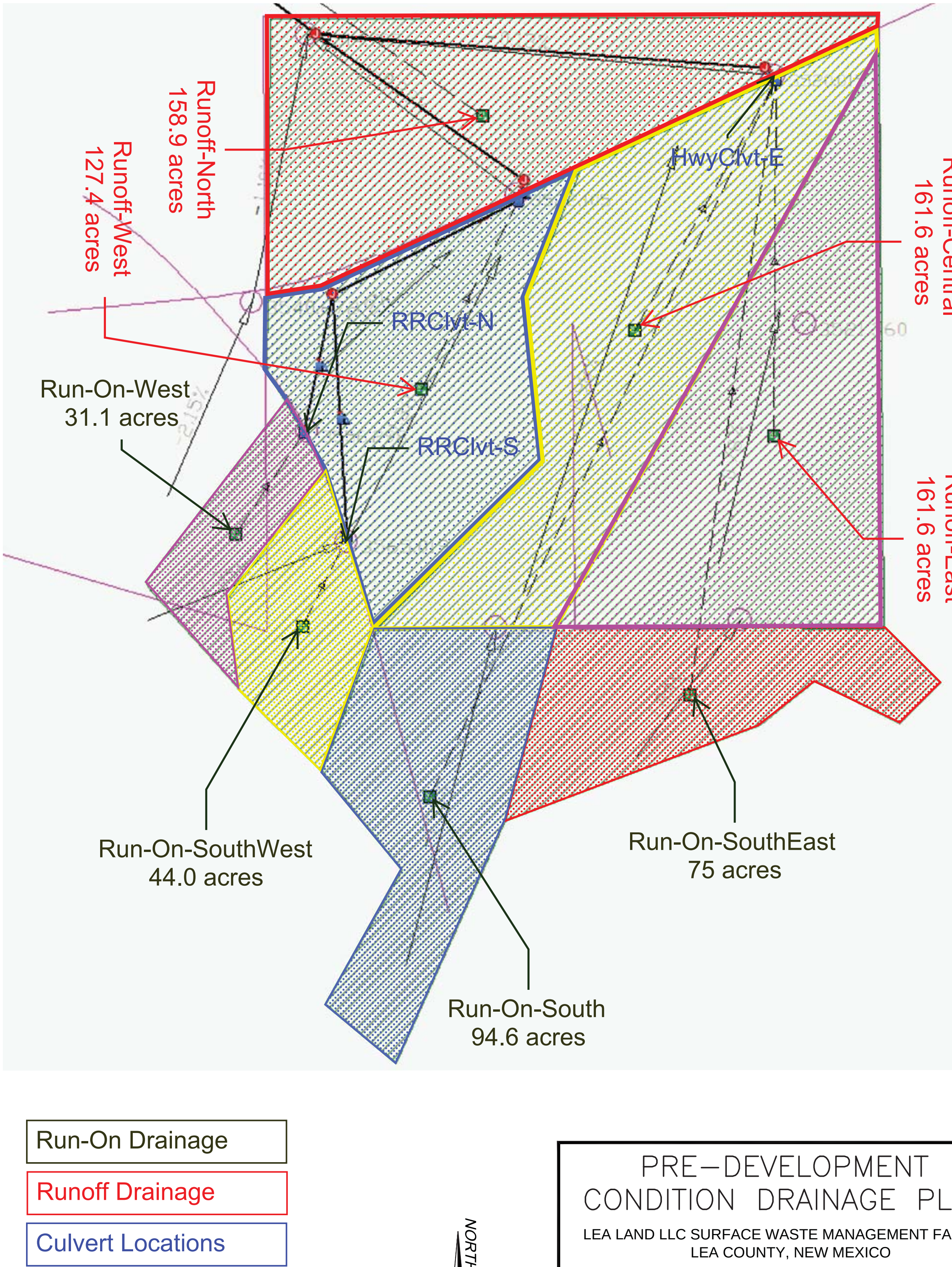
For the pre-development condition, the subbasins used for the SCS TR-20 Method are configured to represent on-site run-on, and contributory run-on. Based on an examination of site conditions, it is evident that the railway drainage structures through the southwest corner of the property cause detention of run-on water west of the tracks, and that the highway drainage structures create significant detention and retention areas south of the highway on the Site. The pre-development drainage areas, run-on controls, and these highway and railroad drainage systems, illustrated in **Figure III.3.2**, have been analyzed utilizing the runoff data acquired via TR-20 Method calculations and runoff flows, and hydrologic routing analyses. The calculations and analysis for the pre-development drainage condition are presented in **Attachment III.3.C**.

Below may be found a general description of the methodology, and a summary of the results of the calculations.

- Obtain the 24-hour rainfall depth directly from the table in **Attachment III.3.B**
  P_{24} = 4.48 inches.
- Calculate the drainage area, A, in acres (**Tables III.3.1** and **III.3.2**):

TABLE III.3.1 – Pre-Development Run-On Drainage Summary

RUN-ON DRAINAGE AREAS				
SUB-BASIN ID	**AREA (ACRES)**	**PEAK DISCHARGE(CFS)**	**VOLUME (ACRE-FT)**	**DISCHARGE TO:**
Run-On-South	94.6	65.68	17.30	HwyClvt-E
Run-On-SouthEast	75.0	30.52	13.71	HwyClvt-E
Run-On-SouthWest	44.0	69.02	8.04	RRClvt-S
Run-On-West	31.1	78.93	5.69	RRClvt-N



PRE-DEVELOPMENT CONDITION DRAINAGE PLAN

LEA LAND LLC SURFACE WASTE MANAGEMENT FACILITY
LEA COUNTY, NEW MEXICO

GORDON ENVIRONMENTAL PSC
333 Rio Rancho Blvd. NE
Rio Rancho, NM 87124
Phone: 505-867-6990
Fax: 505-867-6991

DATE: 03/21/2019	CAD: PRE DRAINAGE.dwg	PROJECT #: 0416.18
DRAWN BY: DMI	REVIEWED BY: CRK	FIGURE III.3.2
APPROVED BY: CWF	www.team-psc.com	

Drawing:X:\2018\0416.18\02_DSGN\02_DWG\050_CIVIL\02_CONTENT\PERMIT FIGURES\PRE-DEVELOPMENT DRAINAGE.dwg
Date/Time:Mar. 21, 2019-13:01:41

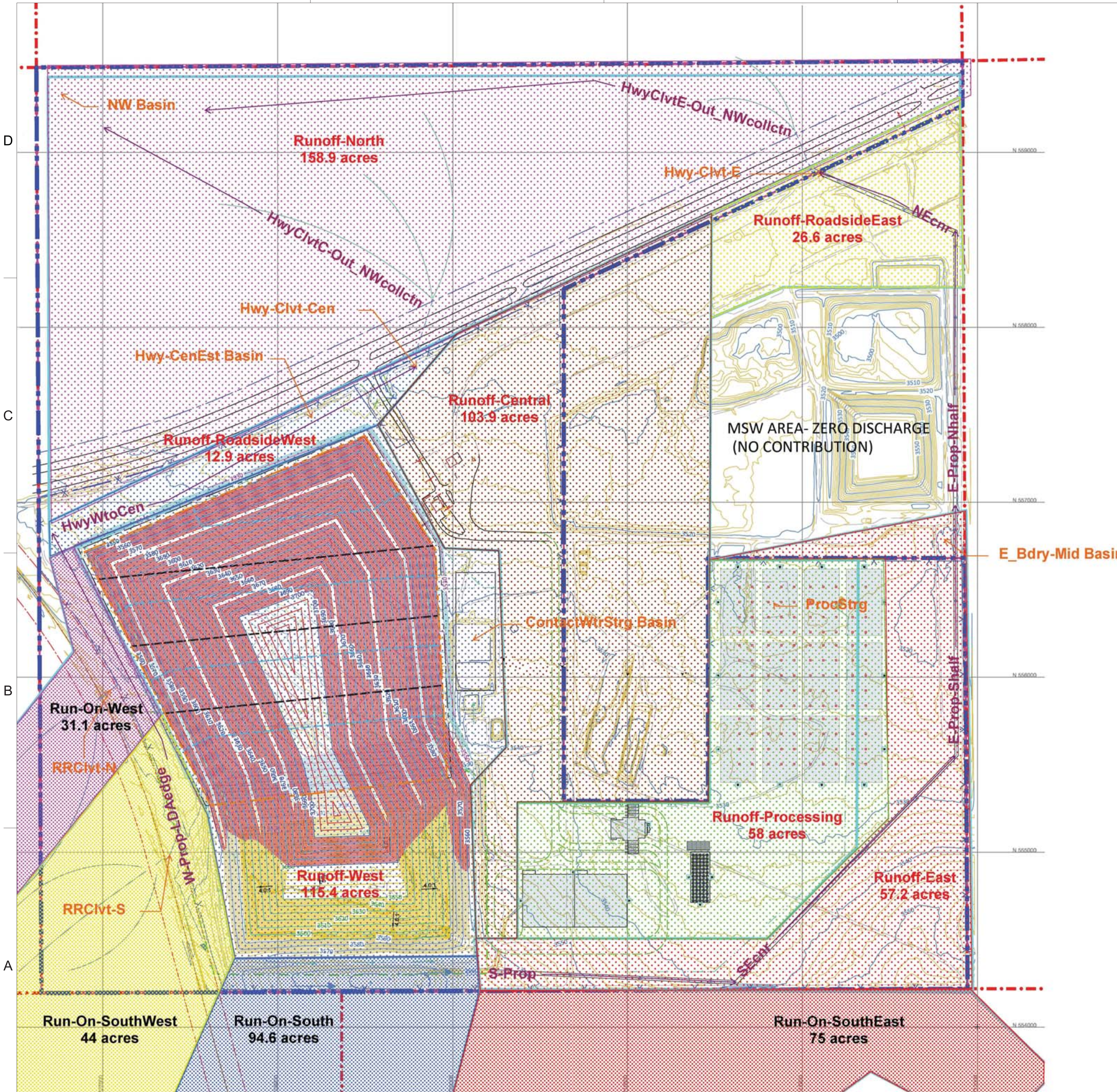
TABLE III.3.2 – Pre-Development Runoff Drainage Summary

RUNOFF DRAINAGE AREAS				
WATERSHED	**DRAINAGE AREA (ACRES)**	**PEAK DISCHARGE(CFS)**	**VOLUME (ACRE-FT)**	**DISCHARGE TO**
Runoff-Central	161.6	148.97	29.55	HwyClvt-E
Runoff-East	161.2	232.63	29.48	HwyClvt-E
Runoff-North	158.9	200.79	29.05	NWBasin
Runoff-West	127.4	176.25	23.30	HwyClvt-Cen

- Determine curve number "CN": From Table 3-1 "Runoff Curve Numbers for Arid and Semiarid Rangelands" in **Attachment III.3.A pg. 4-31**; for Desert shrub-mixture of grass, weeds, and low growing brush, with brush the minor element, Soil Group **B** (consisting of sandy soils, the predominate soils on-site) and 0-30% Vegetation Cover; Hydrologic Condition "poor"; **CN = 77**.
- Based on the final cover design, input the parameters describing the catchment for the electronic TR-55 Time of Concentration, T_c calculations. Catchments are described by one subarea, and information is located in **Attachment III.3.C pages 5** thru **13**. The calculations are based on Sheet Flow, using a Manning's Roughness of 0.08 for Sparse Vegetation and the accepted maximum flow length of 100'; Shallow Concentrated Flow, using the remaining distance the water must travel to the nearest intentional channel; and Channel Flow, using a Manning's roughness of 0.03 for a vegetated earthen channel and the channel dimensions derived iteratively. TR-55's methodology yields a total Time of Concentration.
- The model then uses the Curve Number, rainfall data, and Time of Concentration to derive the Total Runoff (in depth, inches), Peak Runoff (in flow rate, CFS). From there, the system also calculates the Total Runoff Volume, as shown in the table in **Attachment III.3.C pg 2** and summarized in **Tables III.3.1** and **III.3.2**.

5.0 SURFACE WATER CALCULATIONS – FINAL CONDITION

For the final condition, the subbasins used for the SCS TR-20 Method are updated to represent the final condition surface water runoff flow from the landfill, as well as on-site and contributory run-on. The pre-development observations that the railroad and the highway construction detain water are kept as valid for the final condition calculations, as (for example) the culverts that are 4' above normal grade will not be modified in the scope of this development. **Figure III.3.3** provides landfill runoff drainage areas for the finished landform (i.e. final contours) and the final site drainage control system. The TR-20 Method calculations and hydrologic routing analyses used to determine stormwater runoff flows at Lea Land for the Final Condition are presented in **Attachment III.3.D**. Note that the adjacent MSW facility is intended to be a zero-discharge facility, so contributory waters from it are not considered.



LEGEND

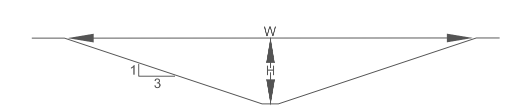
- SECTION BOUNDARY
- PROPERTY BOUNDARY
- EXISTING OCD PERMIT BOUNDARY
- EXISTING HIGHWAY RIGHT-OF-WAY
- EXISTING RAILWAY RIGHT-OF-WAY
- EXISTING TELECOM EASEMENT
- EXISTING EDGE OF PAVED ROADWAY
- EXISTING EDGE OF UNPAVED ROADWAY
- EXISTING CENTERLINE OF RAILWAY
- EXISTING DRAINAGE RUN-ON FLOW PATH
- EXISTING DRAINAGE RUNOFF FLOW PATH
- 3600 PROPOSED FINAL GRADE ELEVATION CONTOUR -INDEX (10')
- PROPOSED FINAL GRADE ELEVATION CONTOUR -INTERMEDIATE (2')
- PROPOSED SURFACE WASTE MANAGEMENT BOUNDARY
- 3600 EXISTING GRADE ELEVATION CONTOUR - INDEX (10')
- EXISTING GRADE ELEVATION CONTOUR - INTERMEDIATE (2')
- PROPOSED EDGE OF UNPAVED ROADWAY
- PROPOSED DRAINAGE RUN-ON FLOW PATH
- PROPOSED DRAINAGE RUN-OFF CHANNEL
- 3600 EXISTING PERMIT COMPLETION GRADE CONTOUR - INDEX (10')
- EXISTING PERMIT COMPLETION GRADE CONTOUR - (2')
- PROPOSED NEW PERMIT LINED LANDFILL BOUNDARY
- PROPOSED NEW PERMIT UNIT BOUNDARY
- PROPOSED LEACHATE COLLECTION PIPE
- PROPOSED DRAINAGE CATCHMENT BOUNDARY
- N 5580000 / E 708000 PROPOSED DRAINAGE CATCHMENT BOUNDARY
- RUN-ON DRAINAGE
- RUN-OFF DRAINAGE

Channel Flow Results and Sizing

Element ID	Length (ft)	(H) Height (ft)	(W) Width (ft)	Peak Flow (cfs)	Peak Flow Depth (ft)	Design Flow Capacity (cfs)
E-Prop-Nhalf	1,499.64	3.00	20.00	107.63	1.53	384.13
E-Prop-Shalf	1,284.15	3.00	20.00	86.72	1.36	384.13
HwyClvtC-Out_NWcollctn	2,194.11	2.00	32.00	33.26	0.43	465.74
HwyClvtE-Out_NWcollctn	3,882.48	2.00	32.00	164.32	1.11	465.74
HwyWtoCen	2,375.24	1.00	14.00	15.13	0.41	69.56
NEcnr	905.56	3.00	20.00	107.61	1.43	432.15
NWcollctn_NWbsn	5,342.82	2.00	34.00	336.84	1.38	620.76
SEcnr	1,816.82	3.00	20.00	86.73	1.36	384.13
S-Prop	1,421.18	3.00	20.00	65.42	1.16	384.13
W-Prop-LDAedge	863.84	1.00	10.00	7.54	0.36	43.90

Subbasin Summary

Subbasin ID	Area (ac)	Peak Runoff (cfs)	Total Runoff Volume (ac-in)
Processing	57.98	54.06	127.21
RoadsideEast	26.63	90.89	58.42
RoadsideW	12.87	21.99	28.23
Runoff-Central	103.88	183.00	227.91
Runoff-East	57.16	82.47	125.42
Runoff-North	158.86	200.79	348.55
Runoff-West	115.39	159.59	253.16
RunOn-South	94.60	65.68	207.56
RunOn-SouthEast	75.01	30.52	164.57
RunOn-SouthWest	43.95	69.02	96.43
RunOn-West	31.10	78.93	68.22



TYPICAL TRAPEZOIDAL CHANNEL
CROSS-SECTION

GORDON ENVIRONMENTAL PSC

CHARLES WALTER FIEDLER
NEW MEXICO
19731
06/26/2019
PROFESSIONAL ENGINEER
P.E.

LEA LAND LLC SURFACE WASTE MANAGEMENT FACILITY

PERMIT APPLICATION

LEA COUNTY, NEW MEXICO

KEY PLAN

N

0 300' 600'

NO	DATE	DESCRIPTION
1	06/03/2019	REVISIONS PER OCD REVIEW
-	04/10/19	OCD PERMIT APPLICATION REVIEW

ISSUING OFFICE: RIO RANCHO PROJECT NO: 0416.18

POST-DEVELOPMENT CONDITION DRAINAGE PLAN

FIGURE III.3.3

FILE NAME: X:\2018\0416.18\02_DSGN\02_DWG\050_CIVIL\02_CONTENT\PERMIT FIGURES\RAI2\POST CONDITION DRAINAGE FIGURE III.3.3.dwg LAYOUT NAME: C-106 PRINTED: Tuesday, July 23, 2019 - 11:13am USER: dmill

Below may be found a general description of the methodology, and a summary of the results of the calculations.

- Obtain the 24-hour rainfall depth directly from the table in **Attachment III.3.B**
  P_{24} = 4.48 inches.
- Calculate the drainage area, A, in acres (**Tables III.3.3** and **III.3.4**):

TABLE III.3.3 – Final Condition Run-On Drainage Summary

RUN-ON DRAINAGE AREAS				
SUB-BASIN ID	**AREA (ACRES)**	**PEAK DISCHARGE(CFS)**	**VOLUME (ACRE-FT)**	**DISCHARGE TO:**
Run-On-South	94.6	65.68	17.30	S_Drn-W-S_Start
Run-On-SouthEast	75.0	30.52	13.71	S_Drn-E-Scnr
Run-On-SouthWest	44.0	69.02	8.04	RRClvt-S
Run-On-West	31.1	78.93	5.69	RRClvt-N

TABLE III.3.4 – Final Condition Runoff Summary

RUNOFF DRAINAGE AREAS				
WATERSHED	**AREA (ACRES)**	**PEAK DISCHARGE(CFS)**	**VOLUME (ACRE-FT)**	**DISCHARGE TO**
Processing	57.98	54.06	10.60	ProcStrg
RoadsideEast	26.63	90.89	4.87	HwyClvt-E
RoadsideW	12.87	21.99	2.35	HwyClvt-Cen
Runoff-Central	103.88	183.00	18.99	Hwy-CenEst
Runoff-East	57.16	82.47	10.45	E_BdryMid
Runoff-North	158.86	200.79	29.05	NWBasinIn
Runoff-West	115.39	159.59	21.10	ContactWtrStrg

- Determine curve number "CN": From Table 3-1 "Runoff Curve Numbers for Arid and Semiarid Rangelands" in **Attachment III.3.A pg. 4-31**; for Desert shrub-mixture of grass, weeds, and low growing brush, with brush the minor element, Soil Group **B** (consisting of sandy soils, the predominate soils on-site) and 0-30% Vegetation Cover; Hydrologic Condition "poor"; **CN = 77**.
- Based on the final cover design, input the parameters describing the catchment for the electronic TR-55 Time of Concentration, $\mathbf{T_c}$ calculations. Catchments are described by one subarea, and information is located in **Attachment III.3.D pages 5** thru **16**. The calculations are based on Sheet Flow, using a Manning's Roughness of 0.08 for Sparse Vegetation and the accepted maximum flow length of 100'; Shallow Concentrated Flow, using the remaining distance the water must travel to the nearest intentional channel; and Channel Flow, using a Manning's roughness of 0.03 for a vegetated earthen channel and the channel dimensions derived iteratively. TR-55's methodology yields a total Time of Concentration.
- The model then uses the Curve Number, rainfall data, and Time of Concentration to derive the Total Runoff (in depth, inches), Peak Runoff (in flow rate, CFS). From there, the system

also calculates the Total Runoff Volume, as shown in the table in **Attachment III.3.C pg 2** and summarized in **Tables III.3.3** and **III.3.4**.

6.0 STORMWATER BASIN DESIGN

Stormwater Runoff from the landform itself is intended to be completely retained in an isolated stormwater retention basin (ContactWtrStrg). Runoff from the processing areas either must be entirely retained for treatment or shall be designed such that the water is retained in the Evaporation Ponds. Further detention ponds are intended to handle the Run-on from the south and west, as well as the Site's non-contact runoff. Due to the low velocities attained in the stormwater channels, and due to the elevated nature of the highway drainage system, the biggest concern set forth in the NMAC pertaining to this site- sediment transport- is not of great risk. Retention Basins are designed to store the design volume of runoff flow, while the detention basin is designed to reduce the off-site flow rate and detain some of the flow. To determine the volume required in the basins, contributory catchments were analyzed based on design stormwater routing, and the catchment runoff volumes from the TR-20 method, accounting for upstream retention as well. *Autodesk® Inc.'s Storm and Sanitary Analysis* package was used to expedite these calculations, and the corresponding data is shown in **Attachment III.3.D pages 21-29** and is summarized in **Table III.3.5.**

TABLE III.3.5 – Stormwater Retention Basin Design Summary

BASIN	CONTRIBUTING DRAINAGE AREAS	RUNOFF VOLUME (ACRE-FT)	BASIN CAPACITY W/ 1FT. FREEBOARD (ACRE-FT)	BASIN MAX. CAPACITY W/O 1FT. FREEBOARD (ACRE-FT)
ContactWtrStrg	Runoff-West	21.10	24	21.2
ProcStrg	Processing	10.6	10.62	10.6

Note that the containment west of the railroad was not analyzed as it remains unchanged, as do the detention/retention basins caused by the highway drainage structures. The data show that the highway drainage structures, previous to design completion, are currently overwhelmed and cause temporary flooding in the area. The permit drainage plan (**Figure III.3.3**) includes adding the 1.5 acre-foot detention basin *E_Bdry-Mid* along the eastern property boundary, and recognizing three closed-basins the MSW facility closure, the processing area, and the permitted landfill. The data also show that these changes in site drainage eliminate the flooding at the highway that occurs during the design storm.

7.0 TYPICAL CHANNEL DESIGN AND CAPCITY

The design frequency peak flow (Qp) from the TR-20 Method was used to size the landfill perimeter drainage channels. Drainage channels are sized to convey the volume of runoff, using the hydrodynamic modelling included in *Autodesk® Inc.'s Storm and Sanitary Analysis* software package. Storm and Sanitary Analysis software uses the runoff information calculated using the TR-20 Method and computes the velocity and depth of flow in the channels based on design values for channel length, slope and cross section dimensions. Channel design parameters, shown in **Attachment III.3.C pages 17-18**, are summarized in **Table III.3.6**, which demonstrates that each of the channels has more than adequate carrying capacity. Note that the orifices represent existing stormwater control structures, i.e., culverts that create stormwater detention areas, whereas the weirs represent design stormwater basin outfalls to provide some retention volume.

TABLE III.3.6 - Channel Design Summary

CHANNEL	Q_{25} (CFS) [1]	SLOPE (%)	VELOCITY (FT/S)	WATER DEPTH (FT)	CHANNEL DEPTH (FT)
E-Prop-Nhalf	107.63	0.1700	6.38	1.53	3
E-Prop-Shalf	86.72	0.3100	5.97	1.36	3
HwyClvtC-Out_NWcollctn	33.26	0.2700	3.64	0.43	2
HwyClvtE-Out_NWcollctn	164.32	0.0400	6.38	1.11	2
HwyWtoCen	15.13	1.5400	3.45	0.41	1
NEcnr	107.61	0.1500	6.92	1.43	3
NWcollctn_NWbsn	336.84	0.3000	7.82	1.38	1
SEcnr	86.73	0.6600	5.97	1.36	3
S-Prop	65.42	1.4100	5.48	1.16	3
W-Prop-LDAedge	7.54	1.1600	3.09	0.36	2

Notes: 1. Q_{25} represents 25-year, 24-hour storm event flow.
2. model does not include effects from the outflow riprap check dam

ATTACHMENT III.3.A

DRAINAGE DESIGN MANUAL, SECTION 400 HYDROLOGY.

NEW MEXICO STATE HIGHWAY AND TRANSPORTATION DEPARTMENT.

STOVALL, PATRICK L.; EASTERLING, CHARLES M.; BARBER, TED L.; MORGENSTERN, STEVEN; TRUJILLO, DAVID; THOMPSON, DAVID B.; JULY 2018

DRAINAGE DESIGN MANUAL

New Mexico
Department of Transportation

NMDOT

July 2018

Drainage Design Manual

New Mexico Department of Transportation

Prepared by:

Smith Engineering Company

Occam Engineers Inc.

Edited by:

New Mexico Department of Transportation

Drainage Design Bureau Staff

and

Thompson Engineering Consultants, Inc.

July 2018

New Mexico Department of Transportation

Drainage Design Manual

Prepared by:

Smith Engineering Company

Patrick L. Stovall, P.E., CFM

Occam Engineers Inc.

Charles M. Easterling, P.E.

Edited by:

New Mexico Department of Transportation
Drainage Design Bureau

Ted L. Barber, P.E.

Steven Morgenstern, P.E.

David Trujillo Jr., P.E.

and

Thompson Engineering Consultants, Inc.

David B. Thompson, P.E.

July 2018

Foreword

The New Mexico Department of Transportation Drainage Design Bureau is pleased to present this updated comprehensive Drainage Design Manual (July 2018). This Manual provides the drainage criteria, standardized drainage analysis methods and many related references to be applied for New Mexico Department of Transportation Projects. This Manual supersedes the previous drainage criteria and drainage manuals listed here.

Drainage Design Criteria, Fourth Revision, June 2007.

New Mexico Department of Transportation.

Drainage Manual Volume 1, Hydrology, 1995.

New Mexico State Highway and Transportation Department.

Drainage Manual Volume II, Hydraulics, Sedimentation and Erosion, November 1998.

New Mexico State Highway and Transportation Department.

Comments regarding the content of this Manual are welcomed and should be addressed to:

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101 Drainage Design Manual Purpose and Use

The New Mexico Department of Transportation (NMDOT) is responsible for the construction and maintenance of a vast network of roads throughout the State of New Mexico. Public safety and prudent investment of public funds in the road network requires that each facility be both reasonably protected from damaging floods and able to safely carry traffic during most rainfall events. Standard methods of analyses and design are continually evolving largely due to the availability of improved technology and greatly expanded digital databases of watershed land use and related data, hydrologic data, topography and aerial photography. The purpose of this manual is to document and standardize, to the greatest practical extent, the state of the practice for hydrologic, hydraulic, and related drainage analyses, as these are the basis for drainage design for New Mexico roadways. This Drainage Design Manual is an update to the previous manuals and documents that are briefly described here.

Previous Manuals and Documents

Volume 1 - Hydrology, (NMSHTD, 1995) and Volume II - Hydraulics, Sedimentation, and Erosion (NMSHTD, 1998) were developed based on the Department's needs and the state of the practice of highway drainage design current in 1995 and 1998. The Drainage Design Criteria document was last updated in 2007 (NMDOT, 2007).

Many of the best practices presented in the previously referenced documents have been retained in this update. The impetus to supplement and update the previous 1995 and 1998 manuals and also update the criteria presented in the 2007 document is due to:

- The Drainage Design Bureau's desire to provide "state of the art" analysis methods appropriate for the NMDOT and New Mexico
- Changes in the type and quantity of data available (particularly digital) such as rainfall, stream gage, soils, aerial photography, topography, etc.
- Advances in desktop computing and geographic information systems (GIS), coupled with computer software

Hotlinks and Cross-References

This Manual contains many hotlinks to referenced source documents. A hotlink (or hyperlink) is a connection or direct link to the referenced source document that is available on another server website, through the internet. In cases when external guidance documents or references are updated after the publication of this Manual, the latest version of those documents will be considered the effective document. References with hotlinks (where available) are provided for the reader to review the source documents.

The hotlinks in this document should be updated regularly since hotlinks can become inactive when the source websites are modified. If a hotlink becomes inactivated, the reader should type in the source document title into an internet browser, and the document should be found. Hotlinks to external documents are shown in blue and underlined. Cross-references to figures, tables, equations, sections, appendices and example problems within this document are shown in **bold text**.

Drainage Design Manual Update

Many of the design procedures and computation methods have been adopted and extracted directly from updated analysis and design guidance documents published by federal agencies. The two most referenced agencies in this Manual are listed here.

Federal Highway Administration (FHWA) for hydraulics, erosion, sediment transport, scour and countermeasure design (for erosion and scour). The FHWA website hotlink listed here provides a full index of all current and archived FHWA publications.
https://www.fhwa.dot.gov/engineering/hydraulics/library_listing.cfm?archived=false

Natural Resources Conservation Service (previously the Soil Conservation Service) Part 630, Hydrology, National Engineering Handbook, Chapters 1-22. Note that various Chapters have different dates. The Natural Resources Conservation Service (NRCS) website hotlink listed here will access this document.
https://www.nrcs.usda.gov/wps/portal/nrcs/detailfull/national/water/manage/hydrology/?cid=stelprdb1043063

Limitations on the use of each analysis method have been included where applicable. This Drainage Design Manual does not include descriptions of the development of, or derivation of analyses methods except by reference.

This manual is not intended to replace the technical manuals referenced or hotlinked, or to be a textbook for hydrology, hydraulics erosion/sediment transport or scour. It is intended to guide engineers new to highway drainage analysis and design, and those more experienced, with the goal of standardizing the analysis and design process given the extremely variable rainfall, elevations, slopes, and soils in New Mexico.

Contact the NMDOT Drainage Design Bureau (DDB) to request spreadsheets developed by the DDB to assist in various calculations.

The Drainage Analysis and Design Process Basics

These questions should be considered before a project begins, and should be addressed and incorporated into every drainage analysis and design:

- How much analysis is warranted for the drainage structure given the size, cost, importance, availability, and quality of data, and consequence of a failure?
- How are failure and non-failure defined?
- What is the probability of failure?
- Are the costs associated with this solution consistent with the benefits?
- Does the solution make sense?
- Will the solution work?
- Can the proposed solution(s) and improvement(s) be practically maintained?

The results should be verified by considering the history and experience as reported by the local patrol foreman, local records, high water marks, historic aerial photography, “rules of thumb”, and other computational methods. Conducting many drainage analyses will provide the experience that leads to developing good judgment, and will assist in exercising prudent engineering practice.

Drainage Infrastructure Past Performance

The methods prescribed in the previous manuals have adequately met the need for a balance between prudent and appropriate design and the capital improvement costs. This statement is based on discussions with the NMDOT Drainage Design Bureau engineers and general observations of highway drainage structures around New Mexico, since the publication of the previous NMDOT drainage manuals and documents.

Summary of Research

During the development of this update, drainage manuals from ten western states excluding New Mexico, were reviewed to determine the current state of the practice of hydrology and hydraulics. The purpose of the review was to discover if other states have developed methods and/or procedures that would be better suited for New Mexico roadways than those in current use. The review and evaluation of those ten drainage manuals revealed that the NMDOT's previous analyses/methods are best suited for New Mexico's needs. However, there are some analyses and design approaches as well as improved methods, that are borrowed from other states and adapted to New Mexico. **APPENDIX 10** contains the Summary of Research that was conducted prior to the preparation of this Drainage Design Manual.

Hydrology

The standard hydrologic analyses methods presented in this Drainage Design Manual should be applied for all NMDOT projects (except in special circumstances as noted). Use of these standard methods will ensure consistency of analysis and design. A brief description of each analysis method is included in this Drainage Design Manual, followed by a step-by-step procedure to apply the method. In many instances, a brief description of the method has been excerpted from its source. In those cases, a hotlink to the source document is provided. Example hydrologic analyses problems are included in **APPENDIX 6.**

This Drainage Design Manual specifies which hydrologic analysis method should be the best choice for use at a particular drainage structure based on drainage area size, location, available data, and physical circumstances. By standardizing the process for choosing hydrologic analysis methods, a consistent and appropriate type and level of analysis is assured for every drainage structure, large and small. However, despite these efforts to standardize both the selection of methods and their reasonable application, proper drainage analysis and design requires experience and competent engineering judgment. Drainage engineers working on NMDOT projects are expected to seek the advice of more experienced engineers when needed and to apply sound engineering judgment throughout the analysis and design process.

Hydraulics

The previous Volume II (1998) manual was developed during a period when there was a nationwide push to convert highway design to metric standards. Since that time, the universal metrification effort has been largely abandoned in most DOTs around the United States including the NMDOT. Many of the updates in this Drainage Design Manual with respect to Volume II, are related to conversion to English standard units from metric units.

This Manual presents more information and references than the 1998 Manual, specifically many more hydraulic equations and analysis methods regarding, sediment transport, scour and erosion countermeasures. Example hydraulic analysis problems are included in **805APPENDIX 7** and example sediment transport and scour analysis problems are included in **APPENDIX 8**.

102 Acronyms

AASHTO – American Association of State Highway and Transportation Officials

ADT – Average Daily Traffic

AMAFCA – Albuquerque Metropolitan Arroyo Flood Control Authority

BFE – Base Flood Elevation (FEMA term for the 100-year water surface elevation illustrated on a Flood Insurance Rate Map)

BLM – Bureau of Land Management

BMP – Best Management Practice

CoCoRAS – Community Collaborative Rainfall, Hail and Snow Network

CFR – Code of Federal Regulations

COA – City of Albuquerque

CWA – Clean Water Act

DACFC – Doña Ana County Flood Commission

DDB – Drainage Design Bureau

DOT – Department of Transportation

EBID – Elephant Butte Irrigation District

EDAC – Earth Data Analysis Center

EPA – Environmental Protection Agency

ESCAFCA – Eastern Sandoval County Arroyo Flood Control Authority

FEMA – Federal Emergency Management Agency

FHWA – Federal Highway Administration

FIRM – Flood Insurance Rate Map

FIS – Flood Insurance Study

GI – Green Infrastructure

GIS – Geographic Information System

LID – Low Impact Development

LIDAR – Light Detection and Ranging

MRCOG – Mid-Region Council of Governments

MRGCD – Middle Rio Grande Conservancy District

MS4s – Municipal Separate Storm Sewer Systems

NEXRAD – Next Generation Radar

NMDGF – New Mexico Department of Game and Fish

NMDOT – New Mexico Department of Transportation
NMED – New Mexico Environment Department
NMIMT – New Mexico Institute of Mining and Technology
NMOSE – New Mexico Office of the State Engineer
NOAA – National Oceanic and Atmospheric Administration
NPDES – National Pollution Discharge Elimination System
NRCS – Natural Resources Conservation Service
NWS – National Weather Service
PDE – Project Development Engineer
RGIS – Resource Geographic Information System (New Mexico) National Weather Service
ROW – Right-of-Way
RSE – Relative Standard Error
SCS – Soil Conservation Service (now the NRCS)
SSCAFCA – Southern Sandoval County Arroyo Flood Control Authority
SWMP – Storm Water Management Plan
TESCP – Temporary Erosion and Sediment Control Plan
TMDL – Total Maximum Daily Load
USACE – U.S. Army Corps of Engineers
USBLM – U.S. Bureau of Land Management
USBR – U.S. Bureau of Reclamation
USDA – U.S. Department of Agriculture
USEPA – U.S. Environmental Protection Agency
USFS – U.S. Forest Service
USFWS – U.S. Fish and Wildlife Service
USGS – U.S. Geological Survey
USWB – U.S. Weather Bureau

103 References

Federal Highway Administration (FHWA), Website. A full index of all current and archived FHWA publications are located at the following website.
https://www.fhwa.dot.gov/engineering/hydraulics/library_listing.cfm?archived=false

NMSHTD, December 1995, "Drainage Manual, Volume 1, Hydrology", Easterling & Associates, Inc.
http://dot.state.nm.us/content/dam/nmdot/Infrastructure/NMHydrologyManual.pdf

NMSHTD, November 1998, "Drainage Manual, Volume II, Hydraulics, Sedimentation and Erosion", Resource Technology, Inc.
http://dot.state.nm.us/content/dam/nmdot/Infrastructure/NMHydraulicManual.pdf

NMDOT, June 2007, “Drainage Design Criteria for New Mexico Department of Transportation Projects, Fourth Revision”, Smith Engineering Company and the NMDOT Drainage Design Bureau Engineers.
http://dot.state.nm.us/content/dam/nmdot/Infrastructure/drainageDesignCriteria.pdf

NRCS, “Part 630 Hydrology, National Engineering Handbook”. Note that various Chapters have different dates.
https://www.nrcs.usda.gov/wps/portal/nrcs/detailfull/national/water/manage/hydrology/?cid=stelprdb1043063

200 DRAINAGE CRITERIA

201 Introduction

This section establishes minimum recommended criteria for drainage structure analyses and design for NMDOT projects. This section also addresses the NMDOT's principles and guidelines related to drainage structure analysis and design criteria. The design criteria were developed based on highway or road classification, Average Daily Traffic (ADT), location (urban or rural), public safety and protection, property protection, public funds availability and economic impacts.

The design criteria must be applied in conjunction with current NMDOT documents and drawings that include the "Standard Specifications for Highway and Bridge Construction" and the "Standard Drawings". These may be obtained from the following hotlinks:

http://dot.state.nm.us/content/dam/nmdot/Plans_Specs_Estimates/2014_Specs_For_Highway_And_Bridge_Construction.pdf

http://dot.state.nm.us/content/nmdot/en/Standards.html

Design variances may be required as a result of budget impacts, right-of-way limitations, environmental and property impacts, or other constraints. Refer to the NMDOT document titled "Design Exception, Design Variance & ADA Design Variance Procedures", November 8, 2016. Refer to the following hotlink to obtain design variance information from that document.

http://dot.state.nm.us/content/dam/nmdot/Plans_Specs_Estimates/Design_Directives/2016/IDD-2016-11_(Design_Exception_Variance_and_ADA_Design_Variance.pdf

Such variances are only allowed when all other options have been considered and found inadequate. If departure from the criteria and design standards for major drainage structures or systems is necessary, a risk assessment may be required. **Section 408** describes the risk assessment procedure. If a jurisdiction or organization has more stringent criteria than the NMDOT drainage criteria, those criteria shall govern the hydrologic analyses, hydraulic analyses and design.

202 Drainage Principles, Guidelines and Definitions

Principles and Guidelines

Drainage system design must consider the following principles and guidelines:

- Preserve, as best possible, the existing drainage path
- Minimize adverse hydraulic affects upstream and downstream of the watercourse crossing
- Minimize the effect on adjacent properties
- Preserve, as best possible, the existing floodplains
- Promote the passage of sediment and debris as much as possible
- Minimize the effects to the environment including impact on fish, wildlife, and wetlands
- Consider safety and welfare of the traveling public

- Protect historic properties and archaeological sites
- Consider and plan for context sensitive design
- Adhere to EPA Permit requirements for Municipal Separate Storm Sewer Systems (MS4s)
- Consider Green Infrastructure (GI) and Low Impact Development (LID) in MS4 areas
- The drainage system design must be in compliance with all environmental regulations and permit requirements
- The design must also plan for maintenance access operations

Definitions

Definitions of terms included in this Drainage Criteria **Section 200** are included in **APPENDIX 1**. Many of these terms are also presented in other Sections of this Manual.

203 Storm Duration and Frequency Criteria

The 24-hour duration storm shall be adopted for all hydrologic analyses.

Minor Arterials, Collectors and Local Roads

Table 203-1 presents the “Storm Frequency Criteria” associated with the Design Flood and Check Flood for various drainage design items with respect to urban and rural locations and ADT ranges for Minor Arterials, Collectors, and Local Roads.

Interstate Highways and Principal Arterials

Table 203-2 presents the “Storm Frequency Criteria” associated with the Design Flood and Check Flood for various drainage design items for Interstate Highways and Principal Arterials. The criteria are applicable to all ADT ranges.

Table 203-1 Storm Frequencies for Minor Arterials, Collectors and Local Roads

	All Urban and Rural >= 400 ADT		Rural < 400 ADT	
	Design Flood	**Check Flood**	**Design Flood**	**Check Flood**
	Storm Frequency in years "y"			
Bridge Freeboard	50 y	100 y	25 y	50 y
Bridge Scour (a)	100 y	500 y	50 y	100 y
Existing Culverts	50 y	100 y	25 y	50 y
New Culverts	50 y	100 y	25 y	50 y
Sidewalk Culverts	50 y	100 y	25 y	50 y
Bridge Deck Drains	50 y	100 y	25 y	50 y
Roadside Ditches and Inlets	50 y	100 y	10 y	25 y
Median Ditches and inlets	50 y	100 y	10 y	25 y
Concrete Channels	50 y	100 y	10 y	25 y
Trunk Lines	50 y	100 y	10 y	25 y
Curb Drop Inlets (b)	50 y	100 y	10 y	25 y
Concrete Wall Barrier (c)	50 y	100 y	10 y	25 y

a - Check other flood frequencies as appropriate for greater scour depths

b - Curb Drop Inlets criteria apply to curbs and similar vertical barriers up to 8" height; also applies to slotted drains

c - Concrete Wall Barrier criteria also apply to Concrete Barrier Railing and vertical barriers greater than 8" height

Table 203-2 Storm Frequencies for Interstate Highways and Principal Arterials

	ADT Range - All	
	Design Flood	**Check Flood**
	Storm Frequency in years "y"	
Bridge Freeboard	50 y	100 y
Bridge Scour (a)	100 y	500 y
Existing Culverts	50 y	100 y
New Culverts	50 y	100 y
Sidewalk Culverts	50 y	100 y
Bridge Deck Drains	50 y	100 y
Roadside Ditches and Inlets	50 y	100 y
Median Ditches and inlets	50 y	100 y
Concrete Channels	50 y	100 y
Trunk Lines	50 y	100 y
Curb Drop Inlets (b)	50 y	100 y
Concrete Wall Barrier (c)	50 y	100 y

a - Check other flood frequencies as appropriate for greater scour depths

b - Curb Drop Inlets criteria apply to curbs and similar vertical barriers up to 8" height, also applies to slotted drains

c - Concrete Wall Barrier criteria also apply to Concrete Barrier Railing and vertical barriers greater than 8" height

204 Hydraulic Criteria for Drainage Structures

Figure 204-1 and **Figure 204-2** present typical roadway sketches to define the basic roadway and drainage related features listed in the criteria tables.

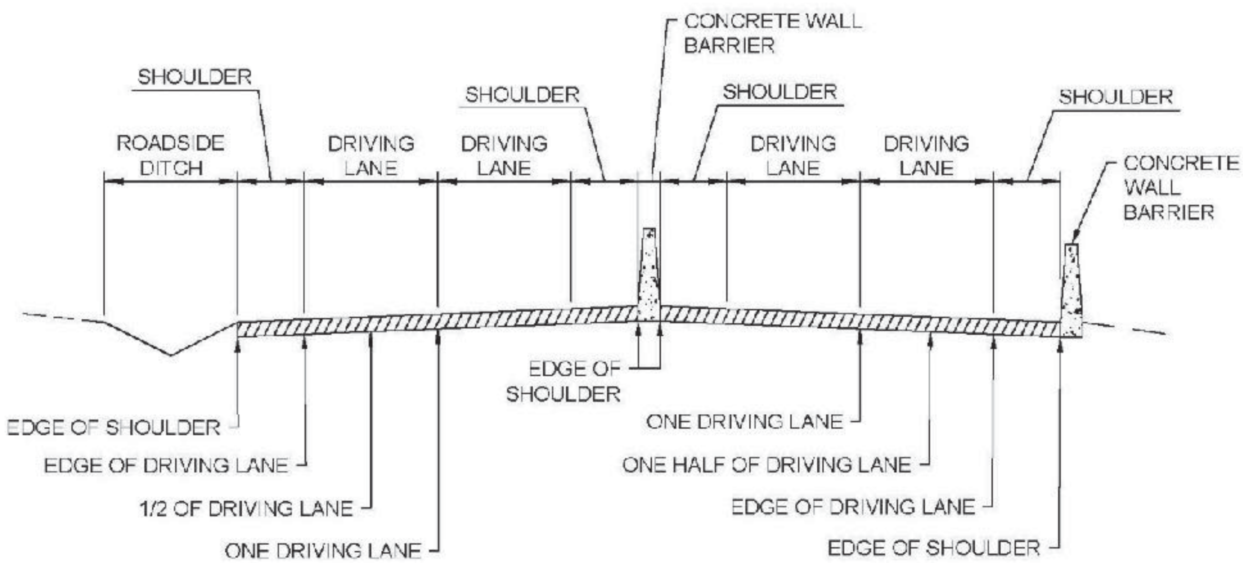


**Figure 204-1 Typical Roadway Schematic:
Section with Roadside Ditch and Concrete Wall Barrier**

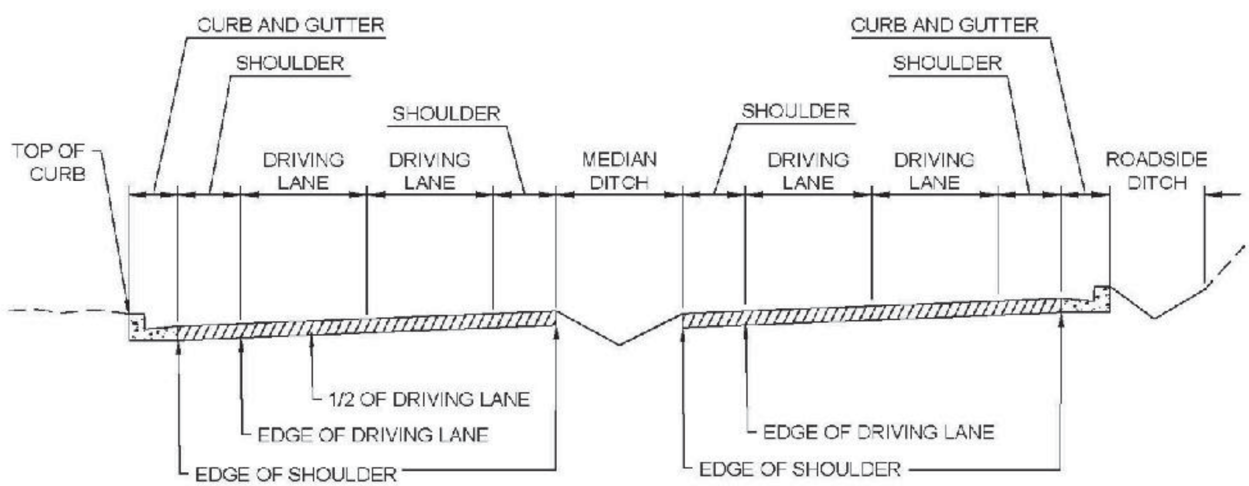


**Figure 204-2 Typical Roadway Schematic:
Section with Median Ditch and Curb and Gutter**

Table 204-1 Design Flood Hydraulic Criteria for Drainage Structures

Design Flood (c)	Two, Four and Six Lane Roads	Interstate
Bridge Freeboard	Minimum of 2 feet	Minimum of 2 feet
Existing Culverts	Limit headwater spread to edge of driving lane	Limit headwater spread to edge of driving lane
New Culverts	Ratio of headwater depth to culvert rise shall not exceed 1.5 and limit headwater to edge of shoulder	Ratio of headwater depth to culvert rise shall not exceed 1.5 and limit headwater to edge of shoulder
Sidewalk Culverts	Limit headwater depth to top of sidewalk	Not applicable
Bridge Deck Drains	Limit water spread to edge of driving lane	Limit water spread to edge of driving lane
Roadside Ditches and Inlets	Limit water spread to edge of shoulder	Limit water spread to edge of shoulder
Median Ditches and Inlets	Limit water spread to edge of shoulder	Limit water spread to edge of shoulder
Concrete Channels	Compute freeboard with equations in Section 204	Compute freeboard with equations Section 204
Trunk Lines	Limit hydraulic grade line to 1 foot below top of grate elevation	Limit hydraulic grade line to 1 foot below top of grate elevation
Curb Drop Inlets (a)	Two Lane - Limit water spread to half of driving lane Four and Six Lane - Limit water spread to 1 driving lane	Limit water spread to edge of driving lane
Concrete Wall Barrier (b)	Two Lane - Limit water spread to half of driving lane Four and Six Lane - Limit water spread to 1 driving lane	Limit water spread to edge of driving lane

a - Curb Drop Inlets criteria apply to curbs and similar vertical barriers up to 8" height
b - Concrete Wall Barrier criteria also apply to Concrete Barrier Railing and vertical barriers greater than 8" height
c - Criteria for both the Design Flood and Check Flood must be achieved

Table 204-2 Check Flood Hydraulic Criteria for Drainage Structures

Check Flood (c)	Two Lane Roads	Four and Six Lane Roads	Interstate
Bridge Freeboard	Below the low chord	Below the low chord	Below the low chord
Existing Culverts	Limit headwater spread to one half of a driving lane	Limit headwater spread to one driving lane	Limit headwater spread to edge of driving lane
New Culverts	Limit headwater spread to one half of a driving lane	Limit headwater spread to one driving lane	Limit headwater spread to edge of driving lane
Sidewalk Culverts	Overtopping the sidewalk is allowed	Overtopping the sidewalk is allowed	Not applicable
Bridge Deck Drains	Limit water spread to one half of a driving lane	Limit water spread to one driving lane	Limit water spread to edge of driving lane
Roadside Ditches and Inlets	Limit water spread to one half of a driving lane	Limit water spread to one driving lane	Limit water spread to edge of driving lane
Median Ditches and Inlets	Limit water spread to edge of driving lane	Limit water spread to edge of driving lane	Limit water spread to edge of driving lane
Concrete Channels	Maximum water surface below top of channel	Maximum water surface below top of channel	Maximum water surface below top of channel
Trunk Lines	Limit hydraulic grade line to the top of grate	Limit hydraulic grade line to the top of grate	Limit hydraulic grade line to the top of grate
Curb Drop Inlets (a)	Limit water depth to top of curb	Limit water depth to top of curb	Limit water spread to edge of driving lane
Concrete Wall Barrier (b)	Limit water spread to one half of a driving lane	Limit water spread to one driving lane	Limit water spread to edge of driving lane

a - Curb Drop Inlets criteria apply to curbs and similar vertical barriers up to 8" height

b - Concrete Wall Barrier criteria also apply to Concrete Barrier Railing and vertical barriers greater than 8" height

c - Criteria for both the Design Flood and Check Flood must be achieved

Peak Discharge Computation at Culverts and Bridges

When roadside ditches or storm drains add flow to the upstream side of a culvert or bridge, peak flow from the ditch/storm drain must be added to the peak flow rate of the arroyo to determine the appropriate flow rate to model through the culvert or bridge. Except in unusual situations and as approved by the NMDOT Drainage Design Bureau, differences in Time of Concentration (Tc) will not be used in this calculation, and the respective peak flows will be simply added together.

Bridge Scour

Calculate the maximum bridge scour depths at piers and abutments. Refer to **Section 607** for scour computation methods. The maximum scour depth may occur during more frequent, less intense storm events than the frequencies for the Design Flood or Check Flood. Evaluate scour for more frequent events if warranted for the circumstance, and then compare to the Design Flood and Check Flood scour results.

Bridge foundations should be designed by an interdisciplinary team of hydraulic, geotechnical, and structural engineers. Bridge foundations shall be designed to withstand the effects of estimated/calculated total scour that is comprised of long-term channel degradation, contraction scour, abutment scour and pier scour (if piers are present).

Concrete Channels

Rectangular channels should be avoided if possible due to additional structural design and construction costs since the walls act as retaining walls. In addition, the vertical walls (depending on channel depth) may be difficult to climb out of during a flood, and therefore present safety issues. Trapezoidal shaped channels are preferred because the problems described for rectangular channels are minimized.

Channel Freeboard

Channel freeboard is the additional wall height applied to a calculated water surface. Concrete channel freeboard shall be computed based on the Design Flood. Freeboard computations are not required for the Check Flood; however, the Check Flood water surface must remain below the top of the channel. The City of Albuquerque Development Process Manual (DPM) (City of Albuquerque, October 2008) criteria and related equations for trapezoidal and rectangular channels are adopted by the NMDOT. The hotlink to the DPM main document is provided below.

http://library.amlegal.com/nxt/gateway.dll/New Mexico/albuqdpm/albuquerquenewmexicodevelopmentprocessma?f=templates$fn=default.htm$3.0$vid=amlegal:albuquerque_nm_mc$anc=JD_DPM

If further DPM information is required from the website, please follow these instructions. After the DPM opens, perform a search for “freeboard,” then select “Chapter 22 Drainage, Flood Control, and Erosion Control”, and the appropriate page will be obtained that contains the trapezoidal and rectangular channel equations and criteria listed below.

Trapezoidal Channels

Adequate channel freeboard above the Design Flood water surface must be provided and shall not be less than determined by the following:

where:

V = velocity, ft/s

d = flow depth, ft

D_c = critical depth, ft

1. For flow rates of less than 100 cfs and average flow V of less than 35 ft/s:
   Freeboard (ft) = 1.0 + 0.025 V $d^{1/3}$
2. For flow rates of 100 cfs or greater and average flow velocity (V) of 35 ft/s or greater:
   Freeboard (ft) = 0.7 (2.0 + 0.025 V $d^{1/3}$)
3. For supercritical flow where the specific energy is equal to or less than 1.2 of the specific energy at Dc, the wall height will be equal to the sequent depth, but not less than the heights required above. This condition should be avoided.

http://library.amlegal.com/nxt/gateway.dll/New Mexico/albuqdpm/albuquerquenewmexicodevelopmentprocessma?f=templates$fn=default.htm$3.0$vid=amlegal:albuquerque_nm_mc$anc=JD_DPM

Rectangular Channels (not used except with NMDOT Drainage Design Bureau approval)

1. For flow depths of 1.0 ft or less and average flow velocities less than 35 ft/s, add 1.0 ft
2. For flow depths of 1.0 ft or less and average flow velocities greater than 35 ft/s, add 1.5 ft
3. For flow depths of greater than 1.0 ft and average flow velocities less than 35 ft/s, add 2.0 ft
4. For flow depths of greater than 1.0 ft and average flow velocities greater than 35 ft/s, add 3.0 ft
5. For supercritical flow where the depth is between critical depth (D_C) and 0.80 D_C, the wall height must be equal to the sequent depth (depth after a hydraulic jump), but not less than the heights required above. This condition should be avoided.

Summary

Freeboard, as determine from the previous equations, will be in addition to any super-elevation of the water surface, standing waves, and/or other water surface disturbances. When the total expected height of disturbances is less than 0.5 ft, disregard their contribution.

Unlined portions of the drainage way may not be considered as freeboard unless specifically approved by the NMDOT Drainage Design Bureau.

205 Additional Criteria for Bridges, Channels, Culverts, Inlets, Concrete Wall Barriers and Other Considerations

Table 205-1 Additional Criteria for Bridges, Channels, Culverts, Inlets, Concrete Wall Barriers and Other Considerations

Bridges - Debris	Estimate pier (if present) debris width and depth and account for conveyance loss in the hydraulic and scour analyses. Estimate based on urban or rural location, watershed and watercourse conditions.
Bridges - Sedimentation	Evaluate the structure and mitigate effects with respect to - significant changes to channel velocity, aggradation or degradation, scour, head cutting, and conveyance.
Culverts - Bulking and Debris Factor	Urban and Rural – For clear water calculations apply a 20% factor. For flows determined by regression equations or a USGS Bulletin 17C analysis of stream gage data, no additional bulking factor should be applied. Refer to **Section 402.11** for bulking factors.
Pipe (storm drain and culvert) - Material and Wall Thickness	Select wall thickness based on Corrosion Resistance Number – **Section 800** (NMDOT Spec. 570.2.3.1) and cover height.
Curb & Median Drop Inlet Grates - Clogging Factor	Inlet Grates on Grade - assume a 25% minimum grate clogging factor.
	Inlet Grates in Sag - assume a 50% clogging factor. Inlet grates in sag will require a minimum of one flanking inlet (an inlet near to and upstream of the sag inlet).
	Median Inlet Grates - assume a 50% grate clogging factor.
Concrete Wall Barrier - Clogging Factor (drainage slots)	Assume a 50% clogging factor due to minimal opening size. Wall barrier in sag will require a minimum of one flanking inlet (an inlet near to and upstream of the sag inlet).
Detour Drainage Structures	Shall be designed to convey the 2-year flood as a minimum. However, some circumstances listed here may require larger flood events. Consult with the Drainage Design Bureau. - A long construction period (longer than 9 months) - Safety concerns due to roadway overtopping - Environmental concerns and potential for environmental damage - Potential for property damage and related economic consequences
Waterstops/turnout humps	All turnouts to NMDOT ROW must be constructed with waterstops (humps), matching the height of the existing curb and gutter or having a minimum height of 4” if curb and gutter is not present. If full-height waterstops are not geometrically feasible, consult with the NMDOT Drainage Engineer for alternative configurations. Turnouts or driveways may discharge runoff to the NMDOT ROW provided that the contributing runoff is included in design calculations for the roadway and storm drain system. If NMDOT will discharge roadway runoff to private property, drop inlets, or other methods to reduce the runoff down the turnout should be installed immediately upstream of the turnout.
Adjacent Properties	Consider and avoid detrimental effects - flooding, sedimentation, or erosion - on adjacent property.
Irrigation Ditches	Ensure that the proposed design does not adversely affect irrigation ditches.
Channel or Stream Deterioration and Modifications	Evaluate the proposed structure and mitigate effects with respect to channel velocity, aggradation or degradation, scour, head cutting, and conveyance. Make allowance in channels for conveyance loss due to debris, vegetation and sedimentation.
Regulatory Requirements	Evaluate proposed structure/project and ensure that any channel or stream modifications meet the requirements of the U.S. Army Corps of Engineers, the NM Environment Department, U.S. Fish & Wildlife Service, U.S. EPA, FEMA, and other agencies.

206 Design Criteria for Storm Drains and Culverts

Table 206-1 Design Criteria for Storm Drains and Culverts

Design Criteria for Storm Drains and Culverts	
Item	**Design Criteria**
STORM DRAINS	
Minimum diameter trunk line	24 inch
Minimum diameter laterals	24 inch
Maximum distance between manholes:	
24 inch storm drain	300 feet
27-36 inch storm drain	400 feet
42-54 inch storm drain	500 feet
60 inch or greater storm drain	600 feet
Minimum cover on pipe	See NMDOT Standard Drawings
Minimum storm drain slope	0.3%
Minimum velocity (trunk and laterals)	2.5 ft/s
Manhole location	Not within an intersection for linear storm drains, may be at an intersection for two trunk lines intersecting at an intersection
CULVERTS	
Minimum diameter turnout culverts	18 inch
Minimum diameter non-turnout culverts	24 inch
Minimum cover on pipe	See NMDOT Standard Drawings
Minimum slope	0.5%
Slope	Match existing slope if steeper than 0.5%
Minimum velocity	3 ft/s
TEMPORARY CULVERTS	
Minimum diameter culverts	12 inch (18 inch is preferable)
Minimum diameter highway culverts	24 inch
Minimum cover on pipe	See NMDOT Standard Drawings and account for load during construction
Minimum slope	0.5%
Slope	Match existing slope if steeper than 0.5%
Minimum velocity	3 ft/s

207 Design Criteria for Detention and Retention Ponds

Jurisdictional Dams and Non-Jurisdictional Dams

Refer to **APPENDIX 1** for definitions as obtained from the following document.

NMOSE Dam Safety Bureau, December 2010, "Rules and Regulations Governing Dam Design, Construction and Dam Safety".

Design of jurisdictional dams shall be avoided for all NMDOT projects.

DETENTION AND RETENTION PONDS

Refer to New Mexico Environment Department (NMED) for Retention Pond definition, stormwater infiltration description, and permitting requirements, if any.

NMDOT Requirement - Infiltration losses, considered in retention pond volume computations, must be documented by infiltration test data or by a qualified reference.

Pond Design Criteria (Detention and Retention Ponds)

- Sediment Bulking
    - Computed/simulated clear water hydrographs shall be increased by a sediment bulking factor to account for sediment volume within the water volume
    - Bulking factors will typically range from about 1.0 for a 100 percent urban impervious watershed including hard lined conveyance systems (no exposed soil or landscape areas), to a maximum factor of about 1.25 for a rural undeveloped or damaged watershed. **Section 402.11** presents more information and items to consider regarding determination of sediment bulking factors. **Figure 402-19** presents a range of bulking factors for various return period floods.
    - Obtain approval from the Drainage Design Bureau regarding sediment bulking factor assumptions and computed or selected values applied for pond analysis and design
    - Sediment bulking factors shall be applied in addition to the dead storage volume requirement (see **Table 207-1**). Dead storage design provides for additional design storage volume due to sediment deposition, and accounts for either lack of maintenance (sediment removal to maintain the design storage volume) and/or storage volume loss from frequent floods/sediment deposition between maintenance activities.
    - A maintenance schedule may be warranted, depending on accumulated sediment loads (volumes) and available storage space.

- Principal Spillways
    - Minimum outfall conduit diameter shall be 24 inches
    - Outfall conduit design maximum pressure and allowable joint pressure capacity shall be documented
    - Detention Ponds - spillways shall provide for floatable debris retention
    - Retention Ponds – do not have principal spillways
    - Outfall design shall include erosion/scour and energy dissipation structures

  - Outfall conduit shall be oriented in the direction of, and outfall to, the natural watercourse
  - Include water quality features as appropriate (e.g., trash racks, perforated riser)
  - Outfall conduit through an embankment shall have piping protection

- Emergency Spillways
  - Detention Ponds - shall have an emergency spillway with sufficient capacity to pass the Check Flood without overtopping the embankment
  - Retention Ponds - shall have an emergency spillway with sufficient capacity to pass the Check Flood without overtopping the embankment
  - Spillways shall be directed to the natural watercourse
  - Spillway approach, crest, chute, and toe design shall include erosion/scour and energy dissipation structures

- Pond Embankments
  - Maximum pond side slopes and embankment slopes shall be 1 vertical to 3 horizontal (1V:3H) if an approved "seeded gravel mulch" is applied. Otherwise maximum slopes of 1V:6H or flatter are required to minimize rill/gulley erosion.
  - Maximum embankment height is defined as the vertical distance from the lowest point on the downstream embankment toe to the lowest point on the embankment crest as defined by the NM Office of the State Engineer Dam Safety Bureau (NMOSE, December 2010). This definition shall also apply to NMDOT pond embankments.
  - Embankment crest width shall be:
    - 12 feet minimum width if a maintenance access road on crest is required by NMDOT Drainage Design Bureau
    - Crest width may be less than 12 feet if a maintenance access road is not required, but not less than 3 feet. Crest widths less than 12 feet must be approved by the NMDOT Drainage Design Bureau
    - Crest width shall be designed in conjunction with embankment design and documented by geotechnical specifications and recommendations
    - Crest width requirements do not apply to retention ponds excavated below ground on all sides

- Maintenance Access Road to Pond Bottom
  - Required – maximum slope allowed shall be 1V:8H (12.5%)
  - Road surface shall be designed to ensure access and may include crushed gravel, base course, or other approved materials and design as required
  - Road should lead to principal spillway structure if possible

- Miscellaneous Pond Requirements
  - An approved permanent sediment stage indication marker (marked in 1 ft increments) shall be installed in all ponds and shall be located near the embankment toe and near the principal spillway
  - Grade detention pond bottoms to drain at minimum 0.5% slope towards the principal spillway. Retention pond bottoms may have 0% slope.

- Fencing shall be installed along the perimeter of all ponds as required. A variance to the fence requirement may be possible based on specific circumstances. For example, a shallow 1 ft maximum depth pond in a gore area

All designs must be approved by the NMDOT.

Refer to **Table 207-1** for additional pond design criteria including:

- Dead storage
- Freeboard
- Allowable peak water surface elevation
- Drain time

Table 207-1 Criteria for Detention and Retention Ponds

Flood		**Design Flood**	**Check Flood**
Storm Frequency		**50-year 24-hour**	**100-year 24-hour**
	Design Item		
DETENTION PONDS (Non-Jurisdictional) (b) (c)	Dead Storage	Rural - Use Check Flood	Rural - provide additional storage volume equal to 20% of inflow hydrograph volume
		Urban - Use Check Flood	Urban - provide additional storage volume equal to 10% of inflow hydrograph volume
	Freeboard	Rural and Urban - 2 ft of freeboard to top of embankment	Rural and Urban - 1 ft of freeboard to top of embankment
	Allowable Peak Water Surface	Rural and Urban - Water surface elevation at or below emergency spillway	Rural and Urban - Emergency spillway may flow with 1 ft of freeboard to top of embankment
	Drain Time	Rural and Urban - must drain in less than 96 hours (a)	Rural and Urban - must drain in less than 96 hours (a)
RETENTION PONDS (Non-Jurisdictional) (b) (c)	Dead Storage	Rural - Use Check Flood	Rural - provide additional storage volume equal to 30% of inflow hydrograph volume
		Urban - Use Check Flood	Urban - provide additional storage volume equal to 20% of inflow hydrograph volume
	Freeboard	Rural and Urban - 2 ft of freeboard to top of embankment	Rural and Urban - 1 ft of freeboard to top of embankment
	Allowable Peak Water Surface	Rural and Urban - Water surface elevation at or below emergency spillway	Rural and Urban - Emergency spillway may flow with 1 ft of freeboard to top of embankment
	Drain Time	Rural and Urban - must infiltrate/evaporate in less than 96 hours (a)	Rural and Urban - must infiltrate/evaporate in less than 96 hours (a)
MS4 Permit Requirements		See **Section 207** text and **Section 700** for more information	
JURISDICTIONAL DAMS		(a)	
a - See **APPENDIX 1** for definitions of non-jurisdictional and jurisdictional dams. Refer to NMOSE Dam Safety Bureau, December 2010, "Rules and Regulations Governing Dam Design, Construction and Dam Safety".			
b - Design all ponds with stormwater quality improvement features. See **Section 506.6.1** for ported principal spillway concepts and **Section 700** for stormwater quality permitting guidance.			
c - See **Section 207** text for further design requirements including sediment bulking factors only for Detention Ponds.			

Stormwater Quality MS4 Requirements

All projects and ponds shall be designed with stormwater quality improvement features. See **Section 700** for permit requirements, additional information regarding stormwater quality design criteria and Green Infrastructure (GI)/Low Impact Development (LID) information.

Municipal Separate Storm Sewer System (MS4) Permit considerations, computations and designs shall be addressed in the Preliminary and Final Drainage Reports. The EPA has a Draft MS4 Permit and a Middle Rio Grande Watershed Based Permit. Note that as the various permittees begin to implement the permit conditions, it is likely that new best management practices suited to New Mexico will be developed, and it is possible that the permit conditions may change. Consult with the Drainage Design Bureau at project inception regarding the latest permit and design requirements.

(Note – Hotlinks for the referenced documents previously located on the EPA website, were not available during the preparation of this Drainage Design Manual.)

Pond Design Criteria

MS4 ponds shall be designed for the clear water runoff volume. Sediment bulking factors are not required unless special circumstances exist. Dead storage volume is not required but is recommended if special circumstances exist. Verify pond design criteria with the Drainage Design Bureau.

Controlling Runoff from New Development and Re-development

One requirement from the Draft MS4 Permit and the existing Middle Rio Grande Watershed Based MS4 Permit, is that Green Infrastructure (GI) and Low Impact Development (LID) practices and control measures shall be implemented under the Post-Construction Stormwater Management, for New Development and Re-development. Permit conditions also include requiring controls that mimic pre-development runoff. For purposes of the MS4 Permit, the pre-development hydrology can be met by retention of the storm volume associated with the 90th percentile storm event for new development sites, and the 80th percentile storm event for re-development sites.

The 90th and 80th percentile storm depths may be computed by following instructions in the Draft Permit and related technical document, or the values in the following table may be adopted by selection of the nearest location given in the table. **Table 207-2** values were obtained from the Draft MS4 Permit.

Table 207-2 80th and 90th Percentile Rainfall Events (inches)

Source: USEPA, March 2015, EPA Publication Number 832-R-15-009, "Estimating Pre-Development Hydrology for Urbanized Areas in New Mexico".

LOCATION NAME	80th Percentile	90th Percentile
Albuquerque International Airport	0.48	0.65*
Farmington Agricultural Science Center	0.40	0.53
Los Alamos	0.53	0.69
Los Lunas 3 SSW	0.48	0.71
Santa Fe 2	0.50	0.68
State University (Las Cruces)	0.55	0.78
El Paso Airport	0.54	0.82

*Use 0.615 inches per the following paragraph.

Notes related to **Table 207-2** and information for the Albuquerque area follow.
The previous predevelopment runoff study (Kosco, et al., 2014) used data from the Albuquerque International Airport for the period 1950-2012. Because rainfall data for the other stations studied in the 2015 report did not extend back to 1950, the 2015 report used the most recent 30-year period of record (1983-2013) for all stations which resulted in a slightly higher 90th percentile event for Albuquerque. For all NMDOT projects within the small MS4 permit areas, use the values in **Table 207-2**.

For the Albuquerque urban area, the following rainfall depth data should be applied from the previous predevelopment runoff study (Kosco, et al., 2014): 0.48 inches = 80th %, 0.615 inches = 90th %. This study is referenced specifically in the Middle Rio Grande Watershed MS4 Permit, and the 0.615 inches shown in this report is the value the EPA has directed to be used.

Alternatively, values may be estimated through site specific pre-development hydrology and associated storm event discharge volume using the methodology specified in the 2015 USEPA Technical Report "Estimating Predevelopment Hydrology for Urbanized Areas in New Mexico".

(Note – Hotlinks for the referenced documents previously located on the EPA website, were not available during preparation of this Drainage Design Manual.)

The pre-development hydrology requirement may be achieved by retaining the increase in runoff that will occur from the added impervious area, computed as follows:

1. New Development –The 90th percentile rainfall depth (inches) multiplied by the new development impervious area, or,
2. Re-development - The 80th percentile rainfall depth (inches) multiplied by the additional re-development impervious area. The retained runoff volume = (post-construction impervious area – pre-construction impervious area) * (80th percentile rainfall depth).

Refer to **Section 700** for more information.

208 References

AASHTO, 2001, “A Policy on Geometric Design of Highways and Streets, Fourth Edition”. http://nacto.org/docs/usdg/geometric_design_highways_and_streets_aashto.pdf

AASHTO, 2011, “A Policy on Geometric Design of Highways and Streets, 6th Edition”.

AASHTO, 2014, “AASHTO Drainage Manual, Chapter 11”.

City of Albuquerque, October 2008, “Development Process Manual, Chapter 22, Drainage, Flood Control and Erosion Control”.
http://library.amlegal.com/nxt/gateway.dll/New Mexico/albuqdpm/albuquerquenewmexicodevelopmentprocessma?f=templates$fn=default.htm$3.0$vid=amlegal:albuquerque_nm_mc$anc=JD_DPM

EPA, Region 6, “Current Internet Download – Region 6 sMS4 General Permit, NMR04000 Stormwater General Permit for Small Municipal Separate Storm Sewer Systems (MS4s)”.
(Note – Hotlinks for the referenced document previously located on the EPA website, were not available during preparation of this Drainage Design Manual.)

EPA, March 2015, Publication Number 832-R-15-009, “Estimating Pre-Development Hydrology for Urbanized Areas in New Mexico”.
(Note – Hotlinks for the referenced document previously located on the EPA website, were not available during preparation of this Drainage Design Manual.)

FHWA, December 1995, “Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation’s Bridges”.
https://www.fhwa.dot.gov/bridge/mtguide.pdf

NMDOT Website, “Standard Specifications for Highway and Bridge Construction”.
http://dot.state.nm.us/content/dam/nmdot/Plans_Specs_Estimates/2014_Specs_For_Highway_And_Bridge_Construction.pdf

http://dot.state.nm.us/content/nmdot/en/Standards.html

NMDOT Website, “Standard Drawings”.
http://dot.state.nm.us/content/nmdot/en/Standards.html

NMDOT, November 8, 2016, “Design Exception, Design Variance & ADA Design Variance Procedures”, Infrastructure Design Directive IDD-2016-11.
http://dot.state.nm.us/content/dam/nmdot/Plans_Specs_Estimates/Design_Directives/2016/IDD-2016-11_(Design_Exception_Variance_and_ADA_Design_Variance.pdf

NMOSE, December 31, 2010, “Rules and Regulations Governing Dam Design, Construction and Dam Safety”, New Mexico Office of the State Engineer Dam Safety Bureau.
http://www.ose.state.nm.us/DS/Regs/19-25-12-NMAC-2010.pdf

300 NMDOT DRAINAGE ANALYSES CHECKLISTS, REPORT AND CONSTRUCTION PLAN REQUIREMENTS

301 Introduction

This Section presents guidance, information, data sources, and lists most topics that should be considered for field work and for inclusion into NMDOT Drainage Report submittals. Adherence to direction provided in this section will promote reports that lead to a holistic evaluation of drainage and design issues and will minimize the review effort by the NMDOT Drainage Design Bureau, and will minimize report re-submittals. The ultimate goal is to promote economic design, constructability, and sustainability of proposed drainage structures.

Questions that should be asked during the drainage analysis and design and be addressed or answered in the drainage report include:

- Is the design buildable?
- Was maintenance access considered and included in the design? Is the design maintainable?
- Was sustainability considered in the planning and design?
- Were location and related issues considered such as:
    - high mountains (snow and ice accumulations, freeze/thaw, perennial streams, fish habitat and environmental issues, brush and tree debris at culverts and bridges, erosion and sedimentation);
    - desert areas (blowing sand, brush debris, erosion and sedimentation);
    - irrigated valleys or low-lying areas (saturated soils)
- Are the subgrade soils and soil profile appropriate for infiltration and recharge?
- Are the subgrade soils expansive or collapsible that requiring special attention to protect the subgrade from water?
- Will the design enhance, be protective of, or adversely impact wetlands or valuable habitat?
- Will the ditches and shoulders likely be vegetated?
- Is there a high probability of large volumes of debris, brush, trash impacting drainage structures?
- Would acquiring more right-of-way make the project easier to maintain and/or construct? (reducing erosion, avoiding retaining walls, and reducing the sizes of headwalls)
- Did the Engineer consider that in urban areas, as Average Daily Traffic (ADT) increases, so does highway generated pollution?
- Where would the water discharge if the structure was overtopped or partially clogged?
- What impact will the project have on existing wetlands, sensitive or critical habitat?
- Are there opportunities to create stormwater mitigation areas or credits within or in association with the project?
- How does the design impact adjacent properties?

- Are there known water quality issues/limitations (303(d) listed receiving waters – Clean Water Act Section 303(d) Impaired Waters and Total Maximum Daily Loads (TMDLs))? https://www.epa.gov/tmdl
- Have stormwater quality improvement features been considered at all locations?

302 Supplemental Data Sources

Supplemental data sources to obtain drainage, flood and water resource information, master drainage and development plans/record drawings (as-built plans), geographic information system (GIS) data, mapping, satellite imagery include but are not limited to the following:

Government Agencies:

- NMDOT maintenance patrol records/verbal information
- NMDOT Maps and Records – record drawings (as-built plans)
- Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA)
- Southern Sandoval County Arroyo Flood Control Authority (SSCAFCA)
- Doña Ana County Flood Commission (DACFC)
- Federal Emergency Management Agency (FEMA), Flood Insurance Study (FIS) Reports and Flood Insurance Rate Maps (FIRMs)
- NOAA Atlas 14 (rainfall data server)
- Next Generation Radar (NEXRAD)
- Community Collaborative Rainfall, Hail and Snow Network (CoCoRAS) (volunteer rainfall data network, managed by the National Weather Service)
- National Weather Service (NWS) (rainfall data)
- Natural Resources Conservation Service (NRCS) (cover type and soils data)
- U.S. Army Corps of Engineers (USACE)
- U.S. Bureau of Reclamation (USBR)
- U.S. Fish and Wildlife Service (USFWS)
- U.S. Geological Survey (USGS) (on-line stream gage data)
- Mid-Region Council of Governments (MRCOG) (current and historic aerial photographs and mapping)
- Cities, towns, and villages
- Local community officials – city and county (public works directors and city engineers)
- New Mexico State Police
- County Sheriffs

Irrigation Districts:

- Elephant Butte Irrigation District (EBID) – operates and maintains many irrigation canals, drains and dams between Percha Dam (below Caballo Dam) and the New Mexico/Texas state line
- Middle Rio Grande Conservancy District (MRGCD) – operates and maintains many irrigation canals and drains between Cochiti Dam and the north boundary of the Bosque Del Apache National Wildlife Refuge

Other Sources:

- Earth Data Analysis Center (EDAC) – maintains a large repository of historical and recent aerial photography and contour mapping

- Google Earth and Bing Maps (current and historical aerial photography and street view)
- Internet search for flood or rainfall reports
- New sources, as methods and technologies develop and supersede others
- Individuals that live near the location
- Newspaper records

303 Field Inspection Checklists

Preparation is required prior to a field visit. During the field visit, various items/tasks must be observed, measured and documented. **APPENDIX 1** contains a Field Trip Preparation Checklist and a Field Trip Observations and Measurements Checklist. Each checklist should be copied, reviewed, and completed as appropriate. The Observations and Measurements Checklist and associated information obtained during the field trip will provide necessary data required for hydrologic and hydraulic analyses. These checklists will guide the engineer to include all items that should be addressed and may help avoid the need for an additional field visit.

304 Drainage Analysis Requirements

Each drainage study will result in one or more required drainage report(s), each report will document all analyses and recommended drainage related improvements. Other tasks that may be required include preparation of drainage and project related permits and coordination with agencies such as:

- U.S. Environmental Protection Agency (EPA) for: sediment/erosion control and stormwater quality issues
- U.S. Army Corps of Engineers (USACE) for: stormwater quality and environmental related issues
- U.S. Fish and Wildlife Service (USFWS) for: biological assessments, stream and riparian area wildlife habitat issues
- Federal Emergency Management Agency (FEMA) for: floodplain related issues
- New Mexico Environment Department (NMED) for: stormwater quality and related environmental issues, infiltration permits
- New Mexico Office of the State Engineer (NMOSE) for: water rights issues and jurisdictional dam determination (for detention ponds)

The engineer may be required to prepare a Temporary Erosion and Sediment Control Plan (TESCP). In addition, coordination with other NMDOT Sections and District offices may be required.

Project Development and Drainage Tasks

NMDOT projects include a standard set of project development tasks and milestones. The standard project tasks and milestones are listed below with drainage related tasks shown in bold text.

Typical Project Development Schedule and Milestones

- Preliminary Scoping Report
- Preliminary Field Review
- **Drainage Field Inspection***
- 30% Plan Review
- 60% Plan Review
- **Preliminary Drainage Report**
- Temporary Erosion and Sediment Control Plan
- **Draft Final Drainage Report**
- 90% Plan Review
- **Revised Final Drainage Report**
- Final Design Review
- Plans, Specifications, and Estimates

*The Drainage Field Inspection is sometimes combined with the 30% Plan Review.

305 Drainage Reports and Submittal Format

Preliminary Drainage Report

The Preliminary Drainage Report should summarize the results of the preliminary drainage analyses. Structure size recommendations will be reviewed by the NMDOT Drainage Design Bureau and will be used for design plans by the NMDOT Highway Design Regions. The Preliminary Drainage Report is prepared concurrently with the 60% plan preparation. Basic elements which should be included in the Preliminary Drainage Report are listed below. A much more detailed Drainage Report Checklist and a Drainage Report Table of Contents Template are included in **APPENDIX 3** and should be used for the actual development of the scope of analyses and report preparation. The following is a brief list of the requirements for preparing Preliminary and Final Drainage Reports:

Items Required on the Cover Include:

- Project Number
- Project Control Number
- Date
- Route Number
- Beginning Milepost Number
- Ending Milepost Number
- Bridge Number(s)
- Document Type: example - Final Drainage Report
- Document Description

Other Items Within the Report Include:

- Professional Engineer - signature, stamp and date
- Drainage design criteria
- Drainage area topographic map with structure locations identified
- Identify soil types, vegetation and land use distribution
- Runoff Curve Number (CN) or Rational Formula Method (C) calculations

- Rainfall tables
- Time of Concentration calculations
- Summarize the drainage field inspection results
- Document the Patrol Foreman interview
- Drainage Structure Field Inspection forms
- Summary Table of existing and recommended drainage structure sizes and types
- Identify data sources and references used in the analysis

The Preliminary Drainage Report typically does not include detailed output from hydrologic or hydraulic analyses, however, data and electronic models generated in the analyses process should be kept on file and submitted with the Preliminary Drainage Report.

Final Drainage Report

The Final Drainage Report is a refinement of the Preliminary Drainage Report. Preparation of the hydrologic and hydraulic calculations and models occurs concurrently with the development of the project design and plan sets. In order to facilitate timely technical review of the drainage assumptions, analysis, and design, a Draft Final Drainage Report should be developed and submitted prior to the 90% Plan Review. This allows time for any necessary changes to the analysis or design. A Revised Final Drainage Report can be submitted after the 90% Plan Review.

The highway design data must include: plan and profile sheets (with grades), typical roadway sections, toe of slope lines, and drainage structure survey data. Modifications to the preliminary hydrologic analyses are completed as required, and final structure sizes are established. A detailed hydraulic analysis (backwater profiles, flow velocities, etc.) is required for bridge structures and for some large culvert locations. Analysis of scour depths at critical locations is required to assist in the design of permanent erosion countermeasure design. At bridge watercourse crossings with unprotected (unlined) beds/overbanks/abutments/piers, a sediment transport and sediment continuity analyses upstream and downstream of the bridge will usually be required.

Drainage Report Checklist

Please refer to **APPENDIX 3** for a Drainage Report Checklist that presents a comprehensive drainage report outline which will serve as a guide during drainage report preparation. This Checklist will assist both the engineer in preparing the scope of the drainage report, and the NMDOT reviewer.

Drainage Reports may not require every item in the Checklist as some items may not be relevant to the analysis or design. The Checklist is provided as a reminder to consider these items during analysis, design, and report development. A Drainage Report Table of Contents Template is also included in **APPENDIX 3**.

Drainage Reports Submittal Format

The NMDOT Drainage Design Bureau will require the following items:

- A digital PDF copy of the stamped and signed drainage report text and appendices
- A digital submission of the hydrologic and hydraulic models
- A digital submission of spreadsheets and other relevant supporting computations and documents

- Quality Assurance and Quality Control (QA/QC) documentation, including written responses to all comments on Plan Sets, Preliminary and Final Drainage Reports

The NMDOT will typically not require a paper submittal, unless specifically requested. Coordinate with the NMDOT Drainage Design Bureau regarding additional or specific information and the format required to assist in the NMDOT review of the preliminary and final drainage analyses, models, recommendations, and reports.

Municipal Separate Storm Sewer Systems (MS4s)

For projects within a USEPA designated MS4, the requirements, applicable data, information and calculations shall be included in the Drainage Report(s). Refer to **Section 700** for permitting requirements.

306 Temporary Erosion and Sediment Control Plans

Design of temporary erosion and sediment control measures or plans are not included in the Preliminary or Final Drainage Reports. The drainage design for erosion and sediment control features and Best Management Practices requires the engineer to refer to the document "National Pollutant Discharge Elimination System Manual (Stormwater Management Guidelines for Construction and Industrial Activities, Revision 2)", NMDOT, August 2012, or current version. The Drainage Design Bureau or the Bureau consultants, prepare Final Stabilization, Erosion and Sediment Control Plans (post construction conditions), while it is the construction contractors' responsibility to prepare Temporary Erosion and Sediment Control Plans for construction phase activities.

NMDOT, August 2012, "National Pollutant Discharge Elimination System Manual - Stormwater Management Guidelines for Construction and Industrial Activities, - Revision 2". http://dot.state.nm.us/content/dam/nmdot/Infrastructure/NPDESM.pdf

307 Construction Plan Drainage Requirements

The following information must be included in the NMDOT construction plans, typically within the 10-Series.

Bridges - Annotate the plans with the following information:

a. DA = drainage area in acres or square miles
b. Qx = design peak flow rate in cfs = Design Flood flow; with "x" representing the Design Flood recurrence interval
c. HWx = headwater in feet; listed as either depth from the upstream bridge invert to water surface at the upstream bridge deck, or the elevation of water surface; with "x" representing the recurrence interval

Through Culverts - Annotate the plans with the following information:

d. DA = drainage area in acres or square miles
e. Qx = design peak flow rate in cfs = Design Flood flow; with “x” representing the Design Flood recurrence interval
f. HWx = headwater in feet; listed as either depth from the culvert invert to water surface, or the elevation of water surface; with “x” representing the recurrence interval

Drop Inlets - Annotate the plans with the following information:

g. DA = drainage area in acres or square miles
h. Qx = design peak flow rate in cfs = Design Flood flow; with “x” representing the Design Flood recurrence interval
i. HGLx = hydraulic grade line shown in profile; with “x” representing the recurrence interval

Storm Drain Network Pipes - Annotate the plans with the following information:

j. Vx = velocity in ft/s for the Design Flood flow; with “x” representing the Design Flood recurrence interval
k. Qx = Design peak flow rate in cfs = Design Flood flow; with “x” representing the Design Flood recurrence interval
l. HGLx = hydraulic grade line shown in profile; with “x” representing the recurrence interval

308 References

NMDOT, August 2012, “National Pollutant Discharge Elimination System Manual - Stormwater Management Guidelines for Construction and Industrial Activities, - Revision 2”.
http://dot.state.nm.us/content/dam/nmdot/Infrastructure/NPDESM.pdf

U.S. Environmental Protection Agency, current internet site, “Clean Water Act Section 303(d): Impaired Waters and Total Maximum Daily Loads (TMDLs)”.
https://www.epa.gov/tmdl

400 HYDROLOGY

The standard methods of hydrologic analyses presented in this Drainage Design Manual should be used for all New Mexico Department of Transportation (NMDOT) structure analyses and design projects. Use of these standard methods will ensure consistency of analysis and design methods to the greatest extent possible. A brief description of each analysis method is included, followed by a step by step procedure to apply the method. **APPENDIX 6** contains example problems to assist the drainage engineer. Note, that for the purposes of water quality protection within a designated Municipal Separate Storm Sewer System (MS4), methods other than the standard methods are prescribed in **Section 700.**

This Drainage Design Manual specifies which hydrologic analysis method should be applied for use at a particular drainage structure based on drainage area size, location, available data, and physical circumstances. By standardizing the process for choosing hydrologic analysis methods, the intent is that a consistent, appropriate type, and level of analysis is assured for every drainage structure, large and small. Despite the efforts to standardize both the selection of methods and their reasonable application, proper drainage analysis and design is not complete without the inclusion of competent engineering judgement. Drainage engineers working on NMDOT projects are expected to apply sound engineering judgement and/or to seek the counsel of more experienced engineers when questions or uncertainty exists throughout the analysis and design development process.

Questions such as these should be considered in every drainage analysis:

- How much analysis effort is warranted for this structure given the size, cost, importance, and consequences of a failure?
- How are failure and non-failure defined?
- What is the probability of failure?
- What are the consequences of a failure?
- Do the analyses results make sense?
- Are the costs associated with the proposed structure(s) consistent with the benefits?
- Will the proposed structure(s) be functional?
- Can the proposed improvement(s) be practically maintained?

Checking the analyses results against experience reported by the local patrol foreman, local records, high watermarks, historic aerial photography, “rules of thumb”, and other computational methods are all part of gaining experience that leads to developing good judgment, and the exercising of prudent engineering practice.

401 NMDOT Approach to Hydrologic Analyses

The NMDOT is tasked with providing transportation facilities that are reasonably safe for the public within the realities of budget and widely varying soils, topography and climate conditions. A safe roadway environment includes proper roadway drainage, and properly designed drainage structures. The NMDOT’s goal is to design and construct roadways and drainage structures that meet minimum design standards and do so within the realities of budgetary constraints. **Section 200** of this Manual presents the current minimum drainage criteria that shall be applied for NMDOT projects.

The NMDOT also recognizes that the effort associated with the design and analysis of drainage structures and roadways must be commensurate with the importance of the transportation facility. Small culverts on low volume roads in remote areas normally do not require exhaustive analyses. For this reason, the NMDOT has established a hierarchy of drainage analysis methods to ensure that appropriate design methods are available and applied.

The goal of the NMDOT Drainage Design Bureau is to standardize the hydrologic analysis methods applied on NMDOT projects, which have a demonstrated performance record in New Mexico. Many hydrologic analysis methods have been used in New Mexico with widely varying results. Some of these methods do not work well in this state, or perhaps are valid only for a particular region of New Mexico. Furthermore, within each hydrologic analysis method, there is some range of judgement or interpretation needed and allowed.

By standardizing hydrologic analysis methods, drainage analysis confusion and debate will be minimized. This Manual provides guidelines for the use of NMDOT approved hydrologic analysis methods, along with visual aides to promote consistency in the selection of parameters which describe physical characteristics such as Runoff Curve Numbers.

The hydrologic methods presented in this manual (with exception of the Rational Formula Method) are based almost entirely on the three publications by the Natural Resources Conservation Service (NRCS), formerly the Soil Conservation Service (SCS). These three document titles and hotlinks as available are listed here.

NRCS, "Part 630 Hydrology, National Engineering Handbook". Note that various Chapters have different dates.
https://www.nrcs.usda.gov/wps/portal/nrcs/detailfull/national/water/manage/hydrology/?cid=stelprdb1043063

NRCS, June 1986, "TR-55, Urban Hydrology for Small Watersheds".
https://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/stelprdb1044171.pdf

SCS, February 1985, "Peak Rates of Discharge for Small Watersheds, Chapter 2, Engineering Field Manual for Conservation Practices".
(Not available on the NRCS website or the internet)

The most pertinent sections from these references have been excerpted directly for ease of use. If further explanation or background information is required, the engineer is directed to the NRCS website where the complete National Engineering Handbook and TR-55 may be found. **APPENDIX 5** contains a copy of the February 1985 document as it is not available on the NRCS website or the internet.

Organization of the Hydrology Section of this Manual

Section 402 provides material that is foundational to the understanding and use of the hydrologic methods which follow in **Section 403** through **Section 408.** However, to facilitate the use of this Manual, sufficient information is provided within each of the method specific sections for the experienced practitioner to be able to perform analyses without having to reference material outside that section. As a result, there is necessarily some repetition of material from

Section 402 in the sections that follow. If, when needing a refresher or clarification of foundational principles, the material and references are provided in **Section 402**.

401.1 Purposes Served by Hydrologic Analyses

Hydrologic analyses are required in both the evaluation of the hydraulic and scour design adequacy of existing drainage structures and to appropriately size and protect proposed new structures. These analyses also serve to determine the drainage impacts that existing and proposed facilities will have on upstream and downstream properties and facilities.

Hydrologic analysis considers the physical processes in a watershed that convert precipitation to runoff. The hydraulic analysis and drainage structure design is dependent on the hydrologic analysis results.

The analyses and design of drainage facilities requires the engineer to:

- Select the appropriate design storms and level of protection desired, specified in terms of the probability of the facility's capacity being exceeded
- Determine the flow rate and/or volume
- Compute in many cases, the corresponding water surface elevation, sediment transport, and scour for that particular stream reach and structure

Peak runoff or discharge in cubic feet per second (cfs) is generally all that is needed in the design of facilities such as storm drain systems, culverts, and sometimes bridges. Hydrographs (flow rate as a function of time) are required for systems that are designed to detain or retain a specified runoff volume, such as detention storage facilities, pump stations, flood routing through culverts/bridges, or when sediment transport analyses are required. Thus, depending on the needs of a particular project, the hydrology study may provide:

- A flow rate for which a return period is specified
- A volume of runoff expected with a specified storm duration, for which the storm return period is specified
- A hydrograph (flow rate as a function of time) for a specified return period. The addition of time allows for determining the effects of storage and/or hydrologic routing from one analysis point to another, and is required for sediment transport analyses

Several methods are provided for use in hydrologic analyses in New Mexico, which are discussed in more detail in **Section 401.2**. A summary of these methods is provided below.

- <u>Rational Formula Method</u> – This Method is appropriate for simple watersheds of 160 acres or less and where only a peak runoff rate is needed, however is not to be used for runoff volume computations. **Section 403** describes the use of the Rational Formula Method.

- <u>NRCS Simplified Peak Discharge Method</u> – This Method is based on the SCS, February 1985 document titled, "Peak Rates of Discharge for Small Watersheds, Chapter 2, Engineering Field Manual for Conservation Practices", and in watersheds with areas up to 10 square miles. Refer to **Section 404.2** for limitations that must be observed with this Method. **Section 404** describes the NRCS Simplified Peak Discharge Method.

- NRCS (SCS) Unit Hydrograph Method within U.S. Army Corps of Engineers "HEC-HMS (Hydrologic Modeling System)" – The HEC-HMS program is a very robust modeling tool and is applicable, but perhaps not most appropriate for all applications. **Section 405** describes the use of the NRCS Unit Hydrograph Method within HEC-HMS.
- USGS Regional Regression Equations – The U.S. Geological Survey, in cooperation with the NMDOT, updated estimates of peak-discharge magnitude for individual gaging stations in the region and updated regional equations for estimation of peak discharge and frequency at ungaged sites. Equations were developed for estimating the magnitude of peak discharges for recurrence intervals of 2-, 5-, 10-, 25-, 50-, 100-, and 500-years at ungaged sites by use of data collected through 2004 for 293 gaging stations on unregulated streams that have 10 or more years of record. **Section 406** describes the use of the USGS Regional Regression Equations. StreamStats is a web-based tool that provides stream flow statistics, drainage basin statistics and other useful information for USGS stream gaging stations and for user selected ungaged steam site locations.
- Watersheds with Stream Gage Data – Performing hydrologic analyses on watersheds with stream gage data is described in **Section 406**.
- Statistical Methods in Watersheds without Stream Gage Data – This topic is described in **Section 407**.
- Risk and Uncertainty in Hydrologic Analyses and Design – This topic is described in **Section 408**.
- Hydrologic Information Required for Water Quality Protection – This topic is described in **Section 700**.

401.2 Selection of Hydrologic Method

The NMDOT Drainage Design Bureau has established specific hydrologic analysis methods to be used on NMDOT projects. The appropriate method is initially selected based on study requirements and the level of effort required as defined by the Drainage Design Bureau. Then the method selected is based on drainage area size and whether the highway facility is located in an urban or rural area. In general, NMDOT personnel and consultants to the NMDOT are required to use the hydrologic methods specified below. The NMDOT Drainage Design Bureau may allow or require other hydrologic analysis methods to be used, depending on project specific circumstances. Contact the Drainage Design Bureau and obtain approval if there appears to be a conflict between methods required by this Manual and local methods before using a method other than those specified below.

Figure 401-1 and **Figure 401-2** are used to select the appropriate hydrologic method for rural watersheds or urban conditions for a particular drainage structure. In areas where a local government agency has a drainage policy which mandates a specific hydrologic analysis method, consult with the NMDOT Drainage Design Bureau to determine the appropriate analysis method. For example, the City of Las Cruces specifies the use of the NRCS Simplified Peak Discharge Method for all projects except those requiring a hydrograph (ponds). Also, when a drainage basin size is on the border (plus or minus 10%) between two size categories, the more detailed analysis method shall generally be used. At the discretion of the engineer and approval of the NMDOT Drainage Design Bureau, the Unit Hydrograph Method may be substituted for the Simplified Peak Discharge Method and the Simplified Peak Discharge Method may be substituted for the Rational Formula Method.

Given the wide range of Standard Error of Estimates of peak discharges found in the USGS Regional Regression Equations, the use of this approach as the sole source of estimates of peak discharge is only allowed with the approval of the NMDOT Drainage Design Bureau. With the availability of public Geographic Information System (GIS) based aerial photography, soils data, and the ease by which this data can be collected and incorporated into both the NRCS Simplified Peak Discharge Method and the NRCS Unit Hydrograph Method in HEC-HMS, these methods should be used to develop the primary hydrology on basins exceeding the 160 acre Rational Formula Method limit. The USGS Regression Equations should generally be limited to confirming order of magnitude validations of deterministic methods and only for very preliminary estimating.

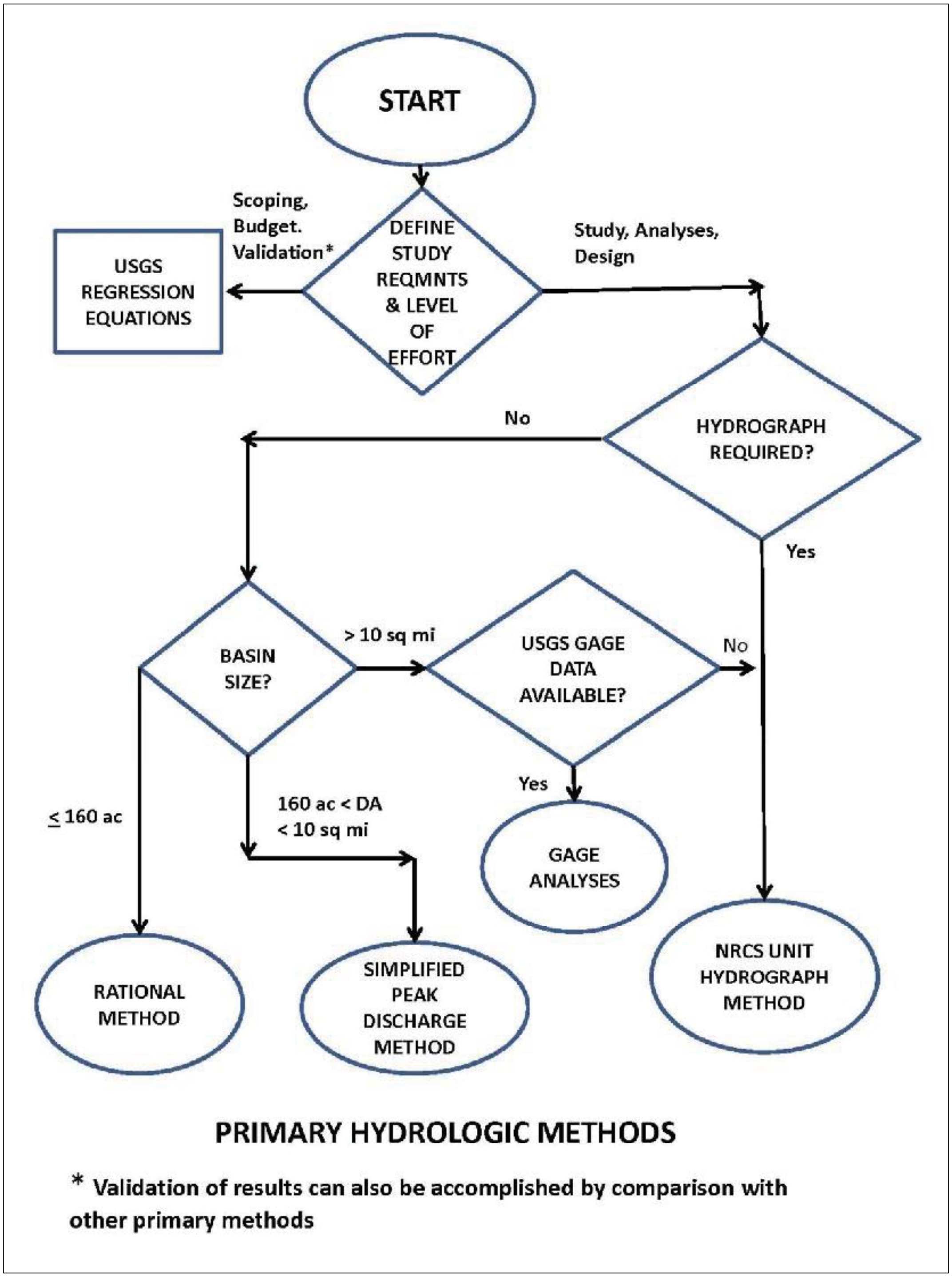
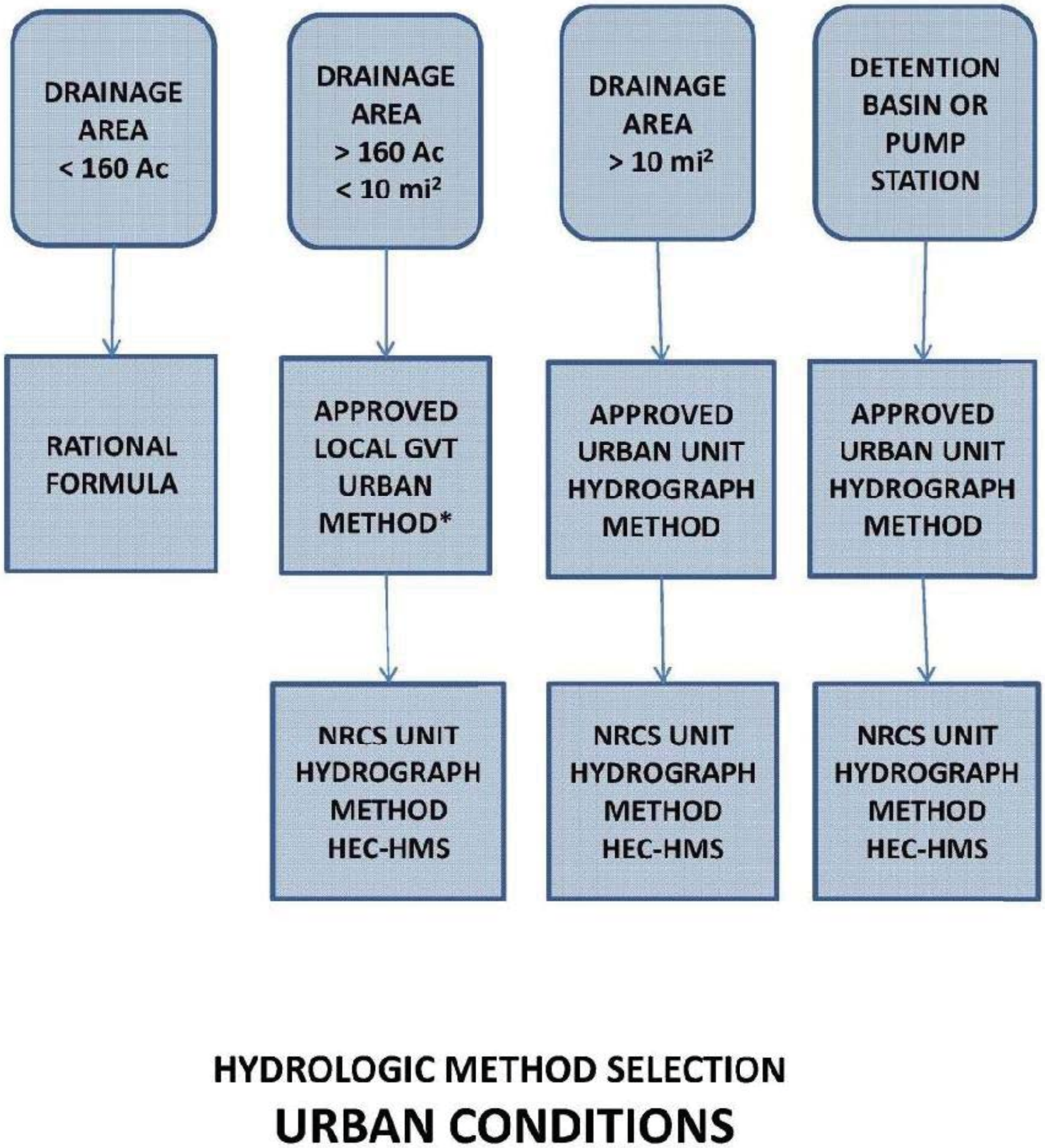


Figure 401-1 Hydrologic Method Selection – Rural Watersheds



* "Approved local government urban methods" are those which are specified by local governments and the use of which NMDOT has approved for a specific project application

Figure 401-2 Hydrologic Method Selection – Urban Conditions

401.3 Basic Requirements for Drainage Studies

This Section describes the basic requirements of a drainage study and schedule for a NMDOT project. NMDOT projects that require drainage studies and drainage reports must identify the drainage criteria applied, and the hydrologic and hydraulic methods/analyses applied to develop the drainage structure design requirements. Most projects require two or more drainage reports that summarize the required drainage improvements for the project. The drainage engineer's responsibility typically does not end with the drainage report.

The NMDOT Drainage Design Bureau staff engineers prepare drainage reports and provide support to the NMDOT Environmental Bureau for obtaining permits (EPA, USACE, FEMA). NMDOT Drainage Design Bureau engineers also develop Sediment and Erosion Control Plans, and coordinate with other NMDOT sections. Similar responsibilities may be required of NMDOT consultants. No matter how limited or broad the project scope of services, a drainage study and associated drainage report(s) will be required.

Most NMDOT projects include a standard set of project development milestones within the NMDOT project development schedule. These standard milestones including drainage elements are shown in bold below.

<u>Typical Project Development Schedule and Milestones</u>

- Preliminary Scoping Report
- Preliminary Field Review
- **Drainage Field Inspection***
- 30% Plan Review
- 60% Plan Review
- **Preliminary Drainage Report**
- Temporary Erosion and Sediment Control Plan
- **Draft Final Drainage Report**
- 90% Plan Review
- **Revised Final Drainage Report**
- Final Design Review
- Plans, Specifications and Estimates

*The drainage field inspection is sometimes combined with the 30% Plan Review.

401.4 Drainage Field Inspection and Drainage Reports

Drainage Field Inspection

Field inspection of the project from a drainage perspective is a critical element of the drainage study process. A thorough inspection will often reveal design considerations which cannot be deduced from aerial photography and available topographic mapping. The drainage field inspection should be performed in the preliminary drainage report phase of the project, after basic data collection and after the preliminary hydrologic analysis has been performed. In this sequence, the field inspection can be used to verify design assumptions, locate and size existing structures, and evaluate the potential impacts of proposed drainage improvements. This is an opportunity to field verify preliminary design assumptions. A list of questions/items should be developed during the preliminary hydrologic analysis which need field verification.

A Field Observation and Measurements Checklist is located in **APPENDIX 3.** A checklist may be used as a reminder of features to observe and quantify in the field. The checklist forms should be completed in the field for all existing drainage structures. Be sure to allow adequate time for the drainage field inspection, particularly if field surveys of structure inlet/outlet conveyances are planned.

Preliminary and Final Drainage Reports

Refer to **Section 305** for more information regarding drainage reports and report submittal requirements.

401.5 References

NRCS, June 1986, "TR-55, Urban Hydrology for Small Watersheds".
https://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/stelprdb1044171.pdf

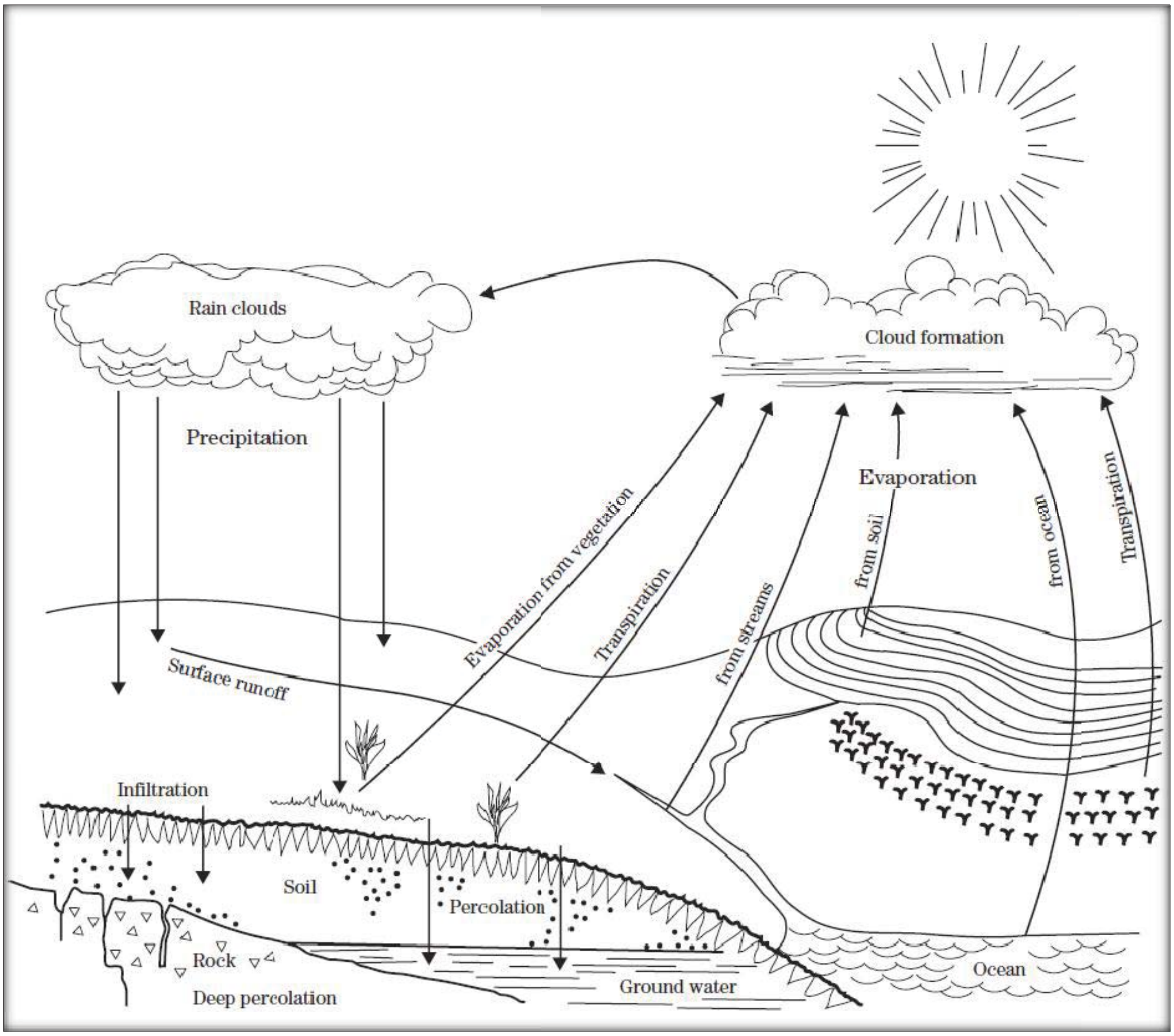
NRCS, "Part 630 Hydrology, National Engineering Handbook". Note that various Chapters have different dates.
https://www.nrcs.usda.gov/wps/portal/nrcs/detailfull/national/water/manage/hydrology/?cid=stelprdb1043063

Soil Conservation Service (NRCS), 1973, Rev. ed. February 1985, Rev. ed. 2014, "Peak Rates of Discharge for Small Watersheds, Chapter 2, Engineering Field Manual for Conservation Practices".
(Not available on the NRCS website or the internet, **APPENDIX 5** contains a copy)

U.S. Army Corps of Engineers Hydrologic Modeling System HEC-HMS, 2015.
http://www.hec.usace.army.mil/software/hec-hms/

402 General Data Requirements for Hydrologic Analyses

To properly prepare hydrologic analyses, it is fundamental to have a solid grasp of the major physical processes, especially, between precipitation and the earth upon which it falls. **Figure 402-1** depicts the hydrologic cycle in schematic form illustrating the processes and interactions.



Source: NRCS, 1997, "Part 630 Hydrology, National Engineering Handbook, Chapter 1 Introduction", Cover Page.
http://www.wcc.nrcs.usda.gov/ftpref/wntsc/H&H/NEHhydrology/ch1.pdf

Figure 402-1 Hydrologic Cycle

Hydrologic analyses are generally data intensive. Unlike structural and pavement design with known loads, the design discharges are unknown, and must be determined for each design project for each component within a project. No two drainage structures share exactly the same circumstances (drainage area, shape, slope, soils, land use, rainfall, and design criteria), the specifics drive the design analysis.

The basic assumptions which are the foundation of each of the hydrologic analysis methods described in this Manual are:

- Rainfall is distributed uniformly over the basin (or subbasin in very large models)
- The rainfall/runoff derivation (Runoff Curve Number (CN), Rational Formula Method Runoff Coefficient (C)) is representative of the average runoff conditions in the basin or subbasin
- The basin Time of Concentration (Tc) represents the time it takes for runoff to reach the analysis point from the most *hydraulically remote* location in the basin or subbasin
- The basin or subbasin slope is relatively uniform throughout the basin or subbasin

When these assumptions are not met, the results are less likely to be accurate or reproducible. Most often, the solution is to subdivide the basin further (within reason).

402.1 Record Drawings and Planned Improvements Information

The hydrologic analysis method selection process begins with the specific project and structure requirements which are determined by the current and/or planned importance of the highway facility it supports. If the project involves existing drainage structures, it is critical to obtain the record drawings (as-built drawings) and ideally, the drainage report which supported the original design. If the project involves new construction, schematic design plans should be available for use in locating and sizing structures. See **Section 200** for more discussion on drainage design criteria related to roadway classification and other parameters.

402.2 Basin and Subbasin Delineation

Regardless of the hydrologic analysis selected, the drainage basin area is always required. Basic to all hydrologic methods is the assumption that the basin or subbasin can be reasonably characterized by one set of hydrologic parameters (soils, slope, rainfall, vegetative cover, and land use). The further from this assumption and the parameters within a basin and subbasin vary, the less accurate and reproducible the results of the analyses will be.

Good "rules of thumb" to follow regarding basin and subbasin sizing are that the length of a basin or subbasin should not exceed 4 times its width and that no subbasin should be more than 10 times larger than the smallest subbasin (NRCS, 2007, "Part 630 Hydrology, National Engineering Handbook, Chapter 16 Hydrographs").
http://www.wcc.nrcs.usda.gov/ftpref/wntsc/H&H/NEHhydrology/ch16.pdf

Basins should be delineated so that soils, cover, land use, slope, and size allow each subbasin to be relatively homogeneous within itself rather than being driven or limited strictly by the location and/or number of analyses points (points of interest) within the basin. These limitations will generally lead to the creation of smaller subbasins that is sometimes dictated by the number and/or location of analysis points. Subbasin size delineation (small, medium, large) within a basin, is based on judgment and experience, and these can be gained by regularly analyzing several different subbasin sizes and configurations, and comparing the results. This sensitivity analysis should be developed early in the hydrologic analysis in order to select the appropriate size subbasins. Experience will lead to confidence in knowing how to delineate and size subbasins correctly. **Figure 402-2** is an example of the subbasin delineation process.

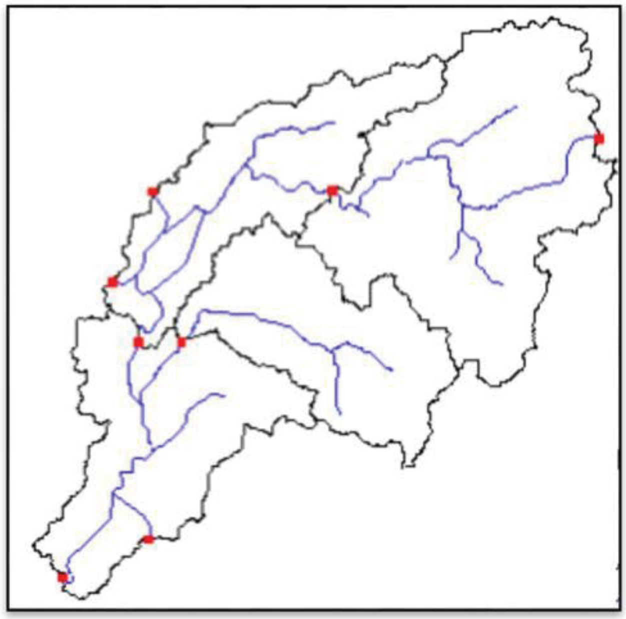


Figure 402-2 Basin Delineation

Drainage basins and subbasins are typically defined graphically using the best available topographic mapping, supplemented with aerial photography and when possible, field verification. USGS topographic maps at 1:24,000 scale provide adequate detail for most rural NMDOT projects and are available for all areas of New Mexico digitally from New Mexico Resource Geographic Information System (RGIS) at: http://rgis.unm.edu/getdata/#. In addition, LIDAR topography is available for many parts of the state in digital form, and the LIDAR coverage area is ever increasing.

Drainage structures crossing roadways are typically located at low spots in the terrain and are always provided where a watercourse crosses or impacts the roadway. Drainage basin boundaries are drawn from the drainage structure location(s), on topographic maps, proceeding uphill such that the boundary encompasses all land which can drain to the crossing structure location. A simple test is to imagine a drop of rain falling on the ground and to follow the path it takes as it flows downhill. Drainage basin boundary lines are drawn perpendicular to the topographic contour lines, following the ridgetops.

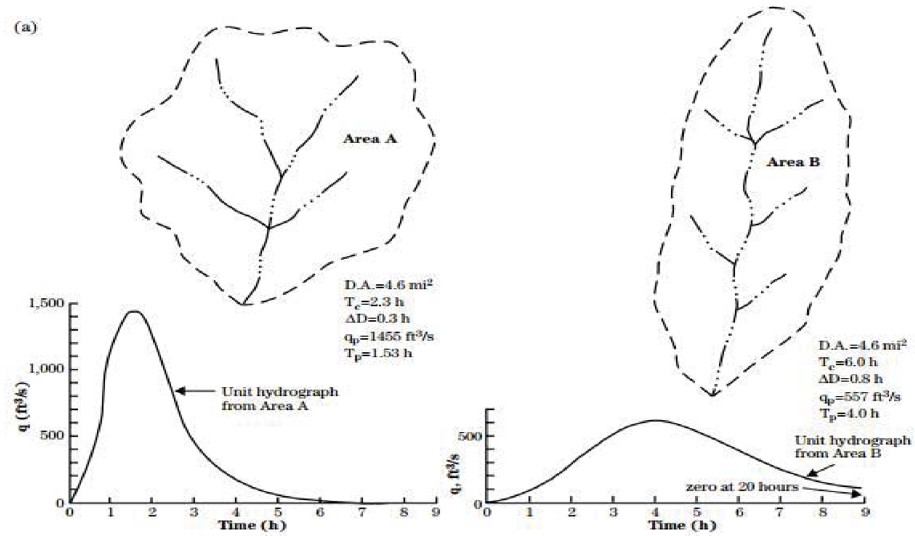
The total basin drainage area can be measured after the drainage basin has been defined. USGS maps are now available in digital format so that this measurement can be made with a GIS tool. A simple guideline should be employed to crosscheck the total drainage area by multiplying the average watershed length by the average watershed width.

Each drainage basin should be qualitatively assessed by the following:

- What hydrologic analysis method is required based on drainage basin size? This may be an iterative process since some methods have size limitations. (e.g. Rational Formula Method ≤ 160 acres, NRCS Simplified Peak Discharge Method ≤ 10 square miles).
- Is the overall drainage basin shape somewhat consistent with implicit assumptions built into the analytical design methods? (i.e., length/width ratio, size relative to other subbasins in the watershed model).
- Subbasins should be sized as uniformly as possible (don't mix 0.5 square mile subbasins with 20 square mile subbasins). The guideline is that no subbasin should be more than 10 times larger than the smallest one in the basin.
- Subbasins should have fairly homogeneous soils, land use, topographic characteristics, and drainage network patterns within themselves. For example, significant areas of mountains, foothills, alluvial plains, and valleys should be in separate subbasins where possible.
- Subbasins should be delineated for each significant tributary at the confluence with the major waterscourse where possible.
- Check to see if roads, diversions, ponds, or other features within the subbasin(s) prevent it from behaving as a uniform, homogeneous watershed. Determine if these features alter flow paths or velocities, create significant storage, or contribute to directly connected imperviousness determinations.
- In flat terrain, are there roads, railroad fill, irrigation facilities or other development features which act as drainage divides or diversions?
- Are there effects of storm drainage networks within urban areas?

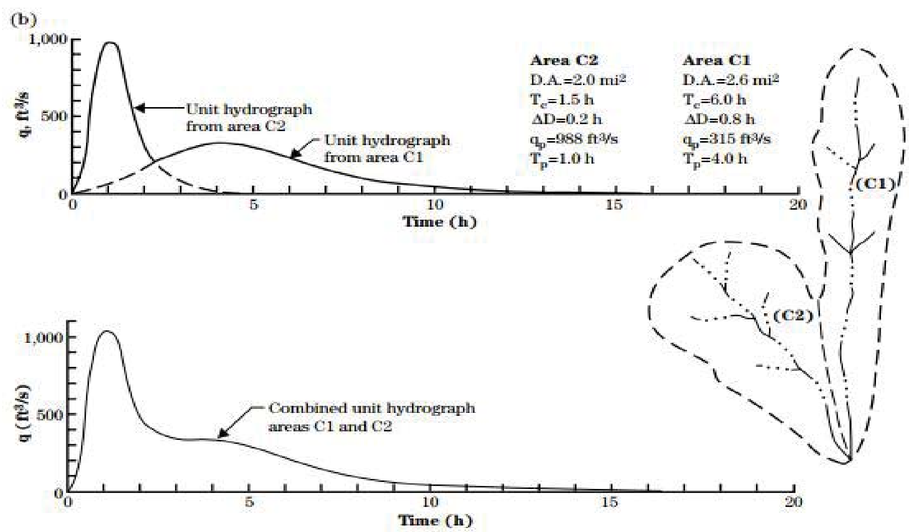
When these factors are accounted for, parameters such as Time of Concentration (Tc), Runoff Curve Number (CN) and Rational Formula Method (C), will more accurately portray the basin runoff response.

An additional consideration when delineating basins is the recognition of the effect that the basin shape can have on the shape (and peak rate) of the resulting hydrograph. **Figure 402-3** and **Figure 402-4** show the effects on the shape of the resultant hydrograph from different shaped drainage basins. Avoid delineating drainage subbasins which are particularly elongated or short and wide. Consider redelineating the subbasins to generally follow the "rules of thumb" (**Section 402.2**).



Source: NRCS, 2007, "Part 630 Hydrology National Engineering Handbook, Chapter 16 Hydrographs", Figure 16-2(a), p. 16-5.
http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=17755.wba

Figure 402-3 Basin Shape Effects on Hydrograph Shape



Source: NRCS, 2007, "Part 630 Hydrology National Engineering Handbook, Chapter 16 Hydrographs", Figure 16-2(b), p. 16-6.
http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=17755.wba

Figure 402-4 Combined Basin Effects on Shape of Hydrograph

402.3 Rainfall Volume and Temporal Distribution Data

Rainfall data is a necessary input parameter for all peak rate computations performed on NMDOT projects (except statistical). The total rainfall volume and the time distribution of the rainfall will both affect the resulting runoff volume and peak runoff rate.

The return frequency of the Design Flood and Check Flood to be used for a particular project or drainage structure must be determined. Design frequency floods are listed in **Section 200.** Note that design criteria and standards are subject to change. Verify that the latest drainage design criteria are applied, and that these criteria are appropriate for the specific roadway classification and design circumstances before proceeding with analysis and design.

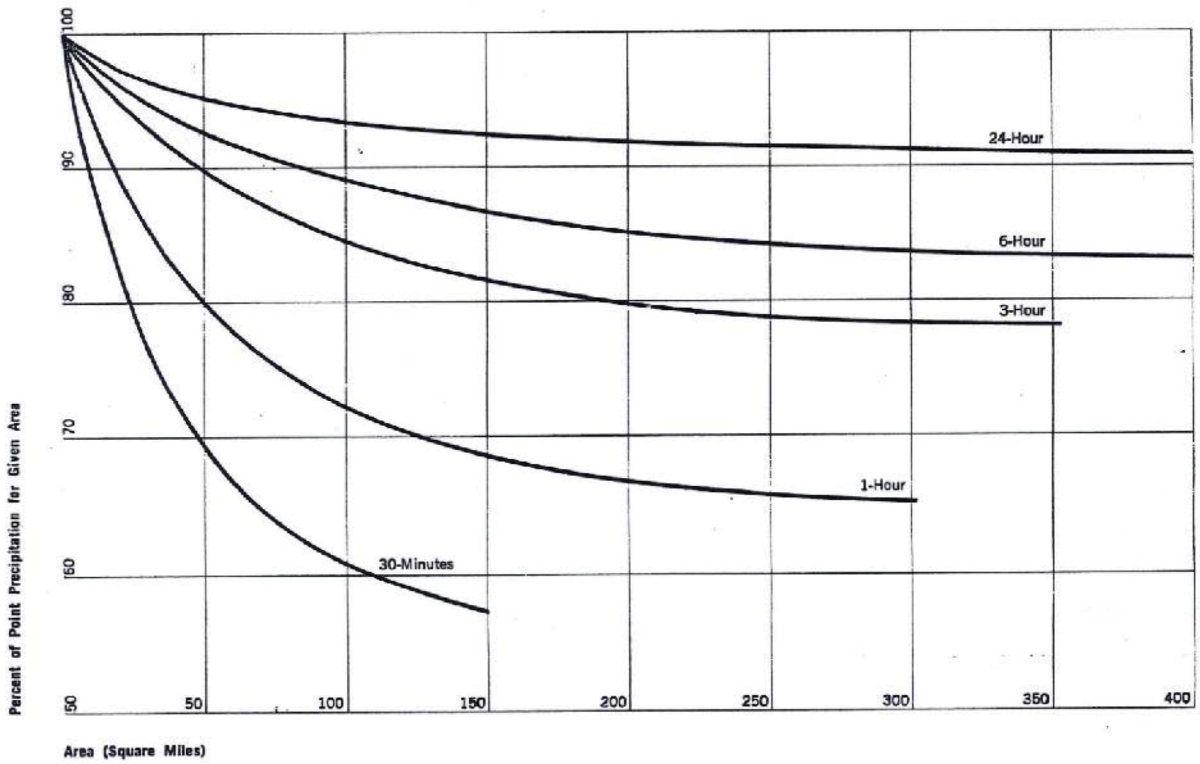
For NMDOT projects, the assumption is made that rainfall frequencies produce equivalent flood frequencies, i.e., the 50-year rainfall event will produce the 50-year runoff event. This assumption is generally valid when all other factors remain reasonably constant (antecedent moisture, etc.), particularly for ephemeral stream systems. There are some situations where this assumption may not be correct. In regions of New Mexico where the seasonal snowpack is significant or that have been affected by severe wildfire, contact the NMDOT Drainage Design Bureau for guidance prior to commencing work.

With the advent of digital rainfall data from NOAA Atlas 14 (2011), rainfall data acquisition is both simpler and more accurate than in the past when only large-scale paper copies of rainfall atlases were available (NOAA Atlas 2, 1973). The NOAA Atlas 14 rainfall data sets are more extensive and more accurate than what was available with NOAA Atlas 2. The NOAA Atlas 14 data has its limitations that should be recognized. Refer to the NOAA Atlas 14 text for a complete discussion of the limitations. It is strongly recommended that the NOAA Atlas text be reviewed and occasionally revisited. New Mexico is covered by *NOAA Atlas 14 Precipitation-Frequency Atlas of the United States Volume 1, Version 5.0 (Rev. ed. 2011)* which is available at:
http://www.nws.noaa.gov/oh/hdsc/PF_documents/Atlas14_Volume1.pdf.

Rainfall data is also available in digital form for any point in New Mexico from the NOAA Hydrometeorological Design Studies Center Precipitation Frequency Data Server (PFDS) at: http://hdsc.nws.noaa.gov/hdsc/pfds/

On all but the largest watersheds (those greater than 10 square miles) and some basins with significant mountain face contributing areas, the rainfall amounts given at the centroid of the basin are appropriate for hydrologic analyses. When performing hydrologic modeling on large watersheds (greater than 10 square miles) and mountain face areas, the rainfall amounts may vary significantly from the furthest downstream point to the most upstream point and, therefore, may be significantly different between subbasins within the model. Subbasin rainfall variations may be simulated within the model.

NOAA Atlas 14 has not yet developed rainfall areal reduction factors (at the time of this Drainage Design Manual preparation). For large basins, NOAA Atlas 14 refers users to NOAA Atlas 2 (1973) that provides guidance on rainfall areal reduction factors. See **Figure 402-5** for NOAA Atlas 2 (1973) area reduction factors for New Mexico. HEC-HMS will accept separate rainfall point amounts for subbasins.



Source: NOAA, 1973, Atlas 2 (not available in digital format)

Figure 402-5 Area Reduction Factors for New Mexico

The NOAA Precipitation Frequency Data Server now provides all the data needed to produce a Precipitation-Intensity Curve for use in the Rational Formula Method. This process is described in **Section 403.2.**

A temporal (time) distribution of rainfall, in addition to the volume, is required for NMDOT designs and Drainage Reports that require a unit hydrograph based modeling effort. The NRCS recommends that a Type II-a design storm distribution be used in New Mexico. The NRCS previously had developed (with the aid of the National Weather Service) a family of temporal distributions that further subdivided the Type II-a storm family for specific parts of New Mexico (i.e.-Type II 60-75). Since the publication of NOAA Atlas 14, tools are available to develop a site-specific distribution that generally follows the NRCS Type II-a distribution and is, therefore, compatible with the NRCS Unit Hydrograph Method. These tools are found in the NOAA Precipitation Frequency Data Server (PFDS) and HEC-HMS. Point rainfalls for various storm durations and frequencies from the PFDS are input into HEC-HMS with a temporal distribution specified to create the design storm distribution for use in developing hydrographs. A more detailed description is included in **Section 405.3**.

Before using rainfall data, read the text provided in NOAA Atlas 14 to gain a better understanding of the source of the data methods used in producing the precipitation frequency information, and the limitations inherent in its use.

402.4 Soils Data

This Section presents detailed soil descriptions and information as background to the Hydrologic Soil Groups (HSGs) as defined by the Natural Resources Conservation Service (NRCS). Note that with GIS tools, the detail presented here is generally not required when completing soils data collection and preparing the related hydrologic data based on the HSGs.

The texture, composition and density of soils have a direct impact on the amount and rate at which rainfall becomes runoff. Therefore, the determination of the soil type(s) is a critical in the development of rainfall/runoff calculations. In general, soils are classified as sandy, silty, loamy or clayey. There can be an infinite number of combinations of these characteristics. The NRCS has divided the extremely wide range of soil textures by their hydrologic (runoff producing) characteristics into four Hydrologic Soils Groups (HSG): Type A, B, C, and D. Type A being generally sandy soils and low runoff producers, and Type D being clayey soils and high runoff producers for a given rainfall volume. Type B and Type C soils have runoff characteristics that are subdivisions within the range of Type A to Type D soils as described below.

Group A

Soils in this group have low runoff potential when thoroughly wet. Water is transmitted freely through the soil. Group A soils typically have less than 10 percent clay and more than 90 percent sand or gravel and have gravel or sand textures. Some soils having loamy sand, sandy loam, loam or silt loam textures may be placed in this group if they are well aggregated, of low bulk density, or contain greater than 35 percent rock fragments.

The limits on the diagnostic physical characteristics of Group A are as follows. The saturated hydraulic conductivity of all soil layers exceeds 40.0 micrometers per second (5.67 inches per hour). The depth to any water impermeable layer is greater than 50 centimeters (20 inches). The depth to the water table is greater than 60 centimeters (24 inches). Soils that are deeper than 100 centimeters (40 inches) to a water impermeable layer are in Group A if the saturated hydraulic conductivity of all soil layers within 100 centimeters (40 inches) of the surface exceeds 10 micrometers per second (1.42 inches per hour).

Group B

Soils in this group have moderately low runoff potential when thoroughly wet. Water transmission through the soil is unimpeded. Group B soils typically have between 10 percent and 20 percent clay and 50 percent to 90 percent sand and have loamy sand or sandy loam textures. Some soils having loam, silt loam, silt, or sandy clay loam textures may be placed in this group if they are well aggregated, of low bulk density, or contain greater than 35 percent rock fragments.

The limits on the diagnostic physical characteristics of Group B are as follows. The saturated hydraulic conductivity in the least transmissive layer between the surface and 50 centimeters (20 inches) ranges from 10.0 micrometers per second (1.42 inches per hour) to 40.0 micrometers per second (5.67 inches per hour). The depth to any water impermeable layer is greater than 50 centimeters (20 inches). The depth to the water table is greater than 60 centimeters (24 inches). Soils that are deeper than 100 centimeters (40 inches) to a water impermeable layer or water table are in Group B if the saturated hydraulic conductivity of all soil layers within 100 centimeters (40 inches) of the surface exceeds 4.0 micrometers per second (0.57 inches per hour) but is less than 10.0 micrometers per second (1.42 inches per hour).

Group C

Soils in this group have moderately high runoff potential when thoroughly wet. Water transmission through the soil is somewhat restricted. Group C soils typically have between 20 percent and 40 percent clay and less than 50 percent sand and have loam, silt loam, sandy clay loam, clay loam, and silty clay loam textures. Some soils having clay, silty clay, or sandy clay textures may be placed in this group if they are well aggregated, of low bulk density, or contain greater than 35 percent rock fragments.

The limits on the diagnostic physical characteristics of Group C are as follows. The saturated hydraulic conductivity in the least transmissive layer between the surface and 50 centimeters (20 inches) is between 1.0 micrometers per second (0.14 inches per hour) and 10.0 micrometers per second (1.42 inches per hour). The depth to any water impermeable layer is greater than 50 centimeters (20 inches). The depth to the water table is greater than 60 centimeters (24 inches). Soils that are deeper than 100 centimeters (40 inches) to a restriction or water table are in Group C if the saturated hydraulic conductivity of all soil layers within 100 centimeters (40 inches) of the surface exceeds 0.40 micrometers per second (0.06 inches per hour) but is less than 4.0 micrometers per second (0.57 inches per hour).

Group D

Soils in this group have high runoff potential when thoroughly wet. Water movement through the soil is restricted or very restricted. Group D soils typically have greater than 40 percent clay, less than 50 percent sand, and have clayey textures. In some areas, they also have high shrink-swell potential. All soils with a depth to a water impermeable layer less than 50 centimeters (20 inches), and all soils with a water table within 60 centimeters (24 inches) of the surface are in this group. Although some may have a dual classification, as described in the next section, if they can be adequately drained.

The limits on the physical diagnostic characteristics of Group D are as follows. For soils with a water impermeable layer at a depth between 50 centimeters and 100 centimeters (20 and 40 inches), the saturated hydraulic conductivity in the least transmissive soil layer is less than or equal to 1.0 micrometers per second (0.14 inches per hour). For soils that are deeper than 100 centimeters (40 inches) to a restriction or water table, the saturated hydraulic conductivity of all soil layers within 100 centimeters (40 inches) of the surface is less than or equal to 0.40 micrometers per second (0.06 inches per hour).

Site-specific information regarding the hydrologic characteristics of the soils needed for analyses in a watershed has been surveyed by NRCS and other agencies for almost the entire country and state of New Mexico. This information is generally available from the NRCS by consulting the Natural Resources Conservation Service's (NRCS) Field Office Technical Guide or the Web Soil Survey Website:
https://websoilsurvey.sc.egov.usda.gov/App/HomePage.htm

Occasionally, when dealing with public lands (U.S. Forest Service, BLM, military bases), the soils information will not be shown in the NRCS database but may be available from the local office of the land management agency responsible for those lands.

It is important to recognize that the NRCS has classified thousands of soils with infinitely varying combinations of textures, thicknesses, and settings into just four Hydrologic Soils Groups (HSGs). Further, it needs to be recognized that within each family of soils there are soils with characteristics that justified them being classified as sub-sets within that family (all of which may not be in the HSG as the parent soil). The engineer may find that some soils do not exhibit the general characteristics of the HSG to which its family has been assigned. When this is observed, it may be helpful to investigate the text of the soil survey report information more thoroughly. An example of a real situation where this condition was found to exist and how it was resolved is provided in a technical paper titled "Hatch Site 6 Runoff Methods Revisited" (Easterrling, Charles, M., May 2004), this is located in Appendix 6 as **Example Problem 6-7.**

For more information on Hydrologic Soil Groups, refer to the following source.

NRCS, 2009, "Part 630 Hydrology, National Engineering Handbook, Chapter 7 Hydrologic Soils Groups".
http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=22526.wba

402.5 Hydrologic Soil-Cover Complexes

A combination of a HSG (soil), land use, and treatment class (cover) is a hydrologic soil-cover complex. A range of Runoff Curve Numbers (CN) has been developed by the NRCS from empirical data and is published by the NRCS in their National Engineering Handbook, Chapter 9 as well as in multiple other locations. The CN represents the runoff potential of a particular soil/cover complex during periods when the soil is not frozen. A higher CN indicates a higher runoff potential, and logically, a lower CN indicates a lower runoff potential. Engineers are strongly encouraged to review and become familiar with the discussion provided in Chapter 9 (Soil-Cover Complexes) of NRCS Part 630 Hydrology, National Engineering Handbook and the academic papers referenced at the end of this Section.
http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=17758.wba

The CN is an input to both the Simplified Peak Discharge Method and the NRCS Unit Hydrograph Method analyses. **APPENDIX 4** contains a series of photographs provided as an aid in the selection of hydrologic conditions as a supplement to the descriptions, figures, and table provided herein. Subbasin runoff volume is governed by the hydrologic soil-cover (vegetation) complexes and impervious surfaces.

402.5.1 Vegetation Effects

Vegetation affects runoff as described here:

- The foliage and its litter maintain the soil's infiltration potential by preventing the sealing of the soil surface from raindrop impact
- Foliage and litter retain some of the raindrops, increasing their chance of being evaporated and/or infiltrated
- Some of the moisture is intercepted on the plant and withheld from the initial period of runoff

- Vegetation and litter transpire soil moisture leaving a greater void in the soil to be filled
- Vegetation, including its ground litter, forms numerous barriers along the path of the water flowing over the surface of the land (these can lengthen the travel time and increase opportunity for infiltration)

Table 402-1 contains information that can be used as a guide in determining the vegetative cover conditions for range sites. Grass cover is evaluated on plant basal area while trees and shrubs are evaluated using canopy cover.

Table 402-1 Vegetative Cover Classes – Grassland

Source: NRCS, 2002, Part 630 Hydrology, National Engineering Handbook, Chapter 8 Land Use and Treatment Classes, Table 8-1, p. 8-3
https://directives.sc.egov.usda.gov/viewerFS.aspx?hid=21422

Vegetative Condition	Hydrologic Condition
Heavily grazed—No mulch or has plant cover on < 0.5 of the area	Poor
Not heavily grazed—Plant cover on 0.5 to 0.75 of the area	Fair
Lightly grazed – Plant cover on > 0.75 of the area	Good

See **Figure 402-6** and **Figure 402-7** on the following pages for further explanation of the relationship between cover condition and Runoff Curve Number.

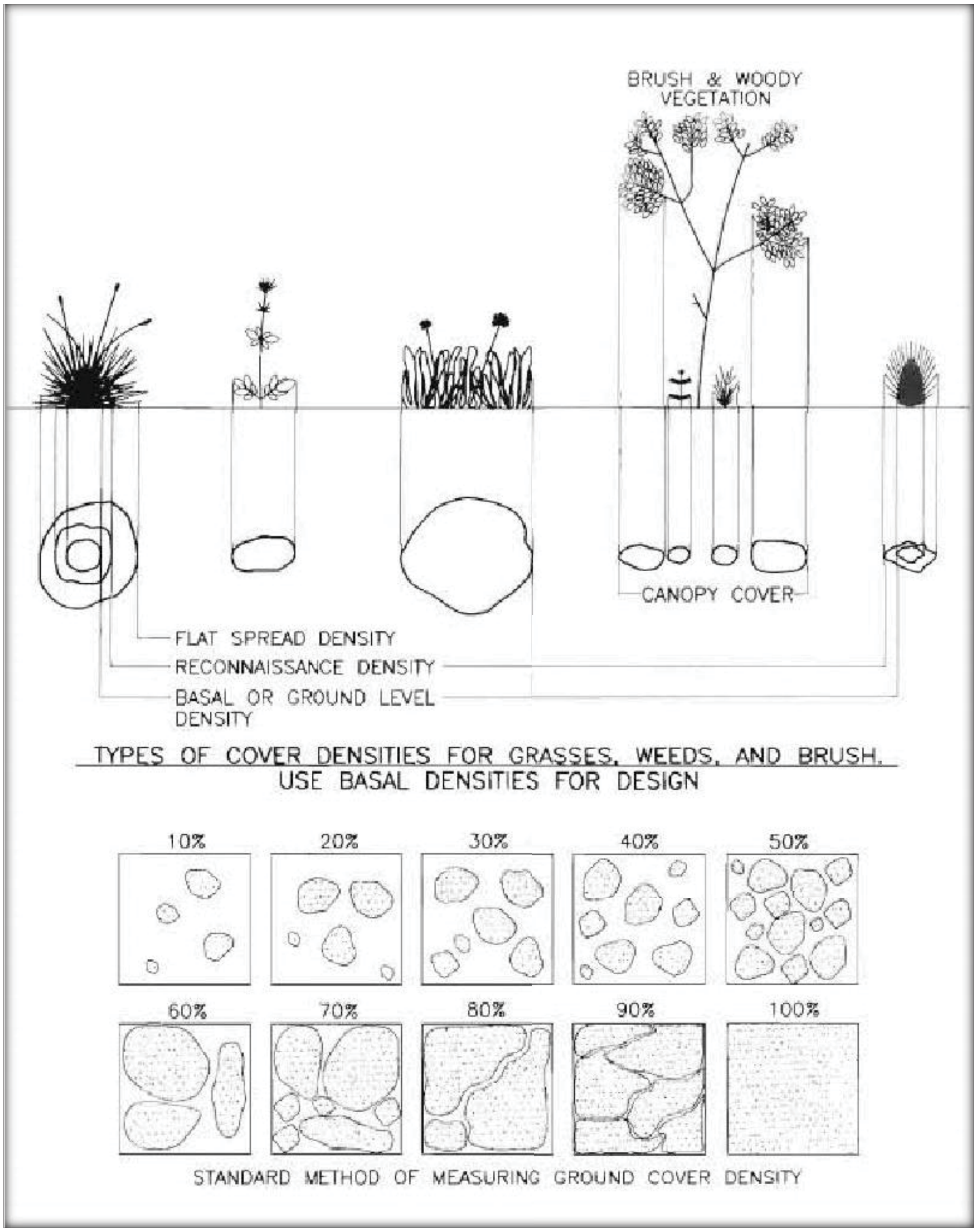
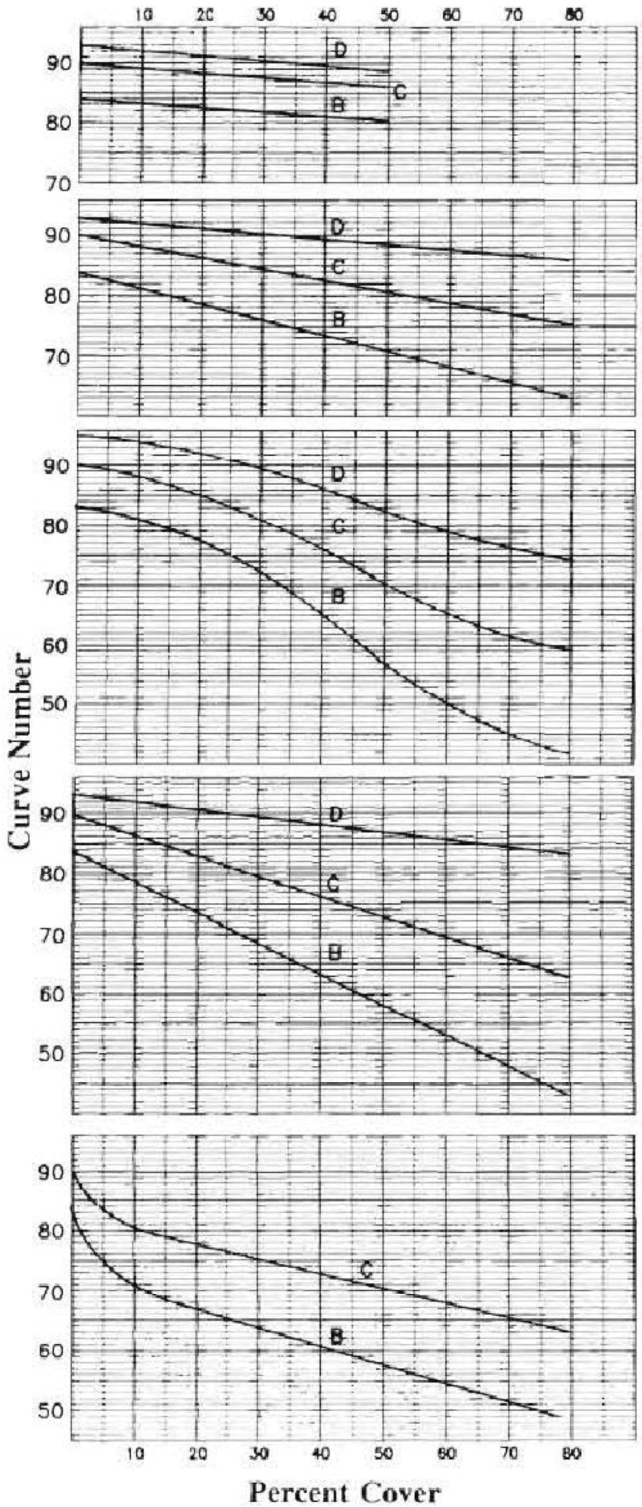


Figure 402-6 Determining Soil-Cover Complex – Vegetative Density



Desert Brush: Brush–weed and grass mixtures with brush the predominant element. Some typical plants are – Mesquite, Creosote, Yuccas, Sagebrush, Saltbush, etc. This area is typical of lower elevations of desert and semi–desert areas.

Herbaceous: Grass–weed–brush mixtures with brush the minor element. Some typical plants are – Grama, Tobosa, Broom Snakeweed, Sagebrush, Saltbush, Mesquite, Yucca, etc. This area is typical of lower elevations of desert and semi–desert areas.

Mountain Brush: Mountain brush mixtures of Oak, Mountain Mohagany, Apache Plume, Rabbit Brush, Skunk Brush, Sumac, Cliff Rose, Snowberry, etc. Mountain Brush is typical of intermediate elevations and generally higher annual rainfall than Desert Brush and herbaceous areas.

Juniper – Grass: These areas are mixed with varying amounts of juniper, piñon, grass, and cholla cover, or may be predominantly of one species. Grass cover is generally heavier than desert grasses due to higher annual precipitation. Juniper – Grass is typical of mountain slopes and plateaus of intermediate elevations.

Ponderosa Pine: These are forest lands typical of higher elevations where the principal cover is timber.

Hydrologic Soil – Cover Complexes and Associated Curve Numbers

Adapted from SCS, Chapter 2 for NM, 1985

Source: SCS, February 1985, Chapter 2 for NM.

Figure 402-7 Hydrologic Soil-Cover Complexes and Associated Curve Numbers

Figure 402-6 and **Figure 402-7** provide good guidance for determining the percentage of vegetative coverage and describe the five principle range and forest soil-cover complex conditions found in New Mexico. For a more complete guide to determining the percentage of vegetative cover, see “Sampling Vegetation Attributes”, Interagency Technical Reference 1996 (Rev. ed. 1997 and 1999) at:
http://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/stelprdb1044175.pdf

Land use has a direct bearing on the amount and types of impervious surfaces that overlay the soils. The type and density of land use also affects the amount of initial abstraction losses that occur in the rainfall/runoff relationship. Most urban areas are only partially covered by impervious surfaces; therefore, the soil remains an important factor in runoff estimates. Urbanization has a greater effect on runoff in watersheds with soils having high infiltration rates (sands and gravels) than in watersheds predominantly of silts and clays, which generally have low infiltration rates. Whether or not impervious areas are directly connected to the stream can make a significant difference in transmission losses, particularly in the case of smaller, more frequent storm events.

Note that the Rational Formula Method Runoff Coefficient (C) is in itself a somewhat simplified term describing the relationship between rainfall and the impacts of soils and cover. Further discussion on this topic is found in **Section 403.3**.

402.6 Runoff Curve Number

The NRCS Runoff Curve Number (CN) (also called Curve Number) is a lumped watershed parameter. It often serves as a proxy for all losses from the beginning of precipitation until runoff reaches the point of interest in a hydrologic analysis. As such, it should not be interpreted as a point infiltration value but rather as representing all losses (initial abstraction, infiltration, transmission, evaporation, etc.) unless separate calculations are developed for ponding and transmission losses.

Methods for selecting a Runoff Curve Number and for making areal adjustments are described below. When carefully followed, these methods will yield a Curve Number which represents the runoff response of the basin or subbasin for the assumed watershed conditions. Seasonal changes in vegetation and ground cover density will occur in the watershed during the year that may cause CN value variations, and should be considered. However, in practice, normally only the largest CN value is adopted. The condition of the watershed may vary dramatically from the date of field reconnaissance to the annual season of largest historic runoff.

Note that NMDOT policies do not allow the analyses to be based on anticipated changes in development unless they are imminent. Check with the Drainage Design Bureau before proceeding regarding proposed development.

Variation in the CN is most evident in cultivated agricultural areas and heavily grazed rangeland where:

1. The land is planted in row crops that are short or tall depending on plant type and growing season, or
2. The crop has been harvested and the ground is plowed or fallow, or the crop type may be changed from year to year, or
3. The plant cover is severely impacted in times of drought.

Note that the rainfall/runoff relationship found in the Curve Number Method is not linear for the many CNs when coupled with design rainfall amounts in New Mexico. The effect is that a small change in CN can dramatically increase or decrease the amount of runoff that results under certain combinations of CN and rainfall as presented in **Figure 402-8.**

Therefore, engineering judgement must be exercised to determine the appropriate CN for a particular drainage basin or subbasin.

The following excerpts from Chapter 2 of "TR-55, Urban Hydrology for Small Watersheds", (NRCS, June 1986) provide a relatively complete and clear explanation of the Curve Number, its determination, and its use in hydrologic analyses. A hotlink to the document is provided below.
https://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/stelprdb1044171.pdf

Figure 402-8 describes the relationship of rainfall and runoff for the range of possible Runoff Curve Numbers based on the following equation:

$$Q = \frac{(P - 0.2\,S)^2}{P + 0.8\,S} \qquad \textbf{402-1}$$

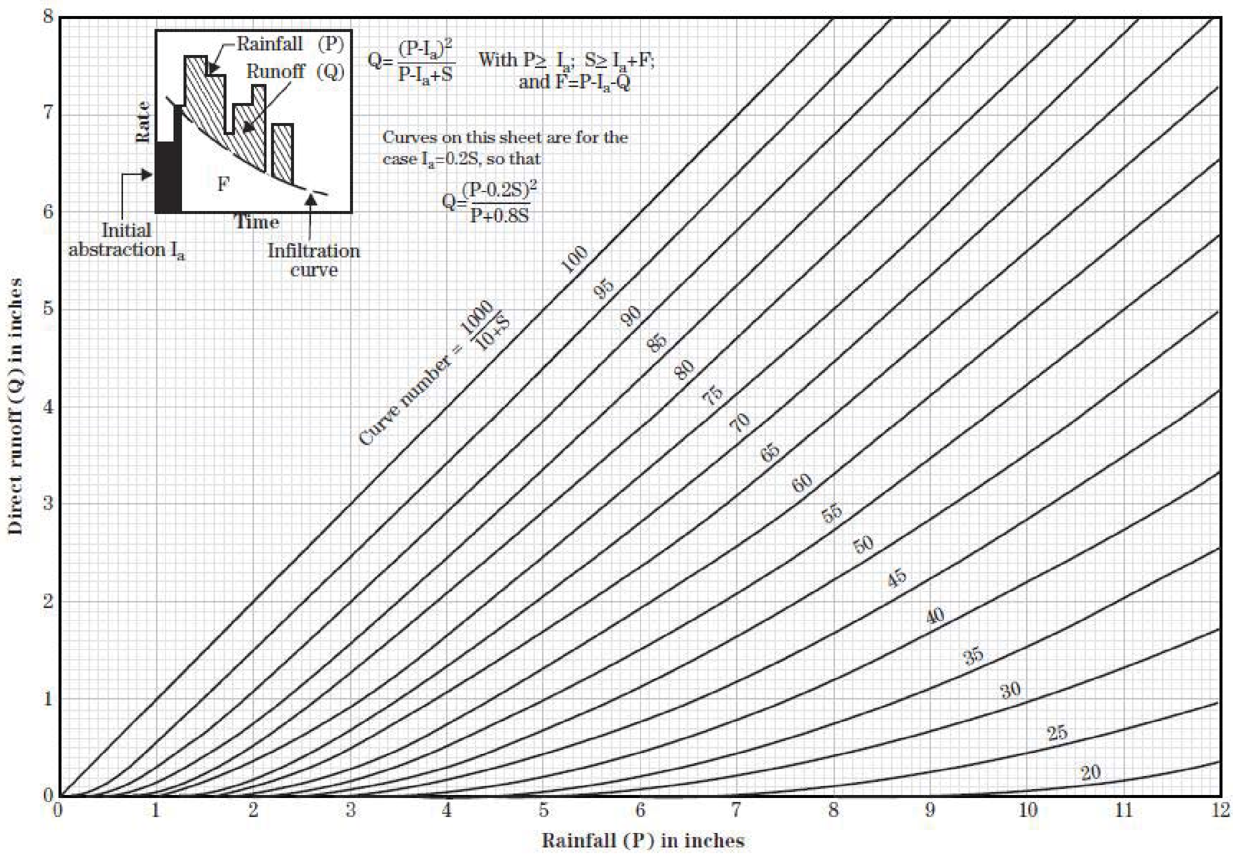
(NRCS, June 1986, "TR-55, Urban Hydrology for Small Watersheds", Eq. 2-3, p. 2-1)
https://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/stelprdb1044171.pdf

where:

Q	=	runoff, inches
P	=	rainfall, inches
S	=	potential maximum soil moisture retention after runoff begins
CN	=	Runoff Curve Number

$$S = \left(\frac{1000}{CN}\right) - 10 \qquad \textbf{402-2}$$

(NRCS, June 1986, "TR-55, Urban Hydrology for Small Watersheds", Eq. 2-4, p 2.1)
https://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/stelprdb1044171.pdf



Source: NRCS, 2004, "Part 630 Hydrology, National Engineering Handbook, Chapter 10 Estimation of Direct Runoff from Storm Rainfall", Figure 10-2, p. 10-4
https://directives.sc.egov.usda.gov/17752.wba

Figure 402-8 Solution of Runoff Equation

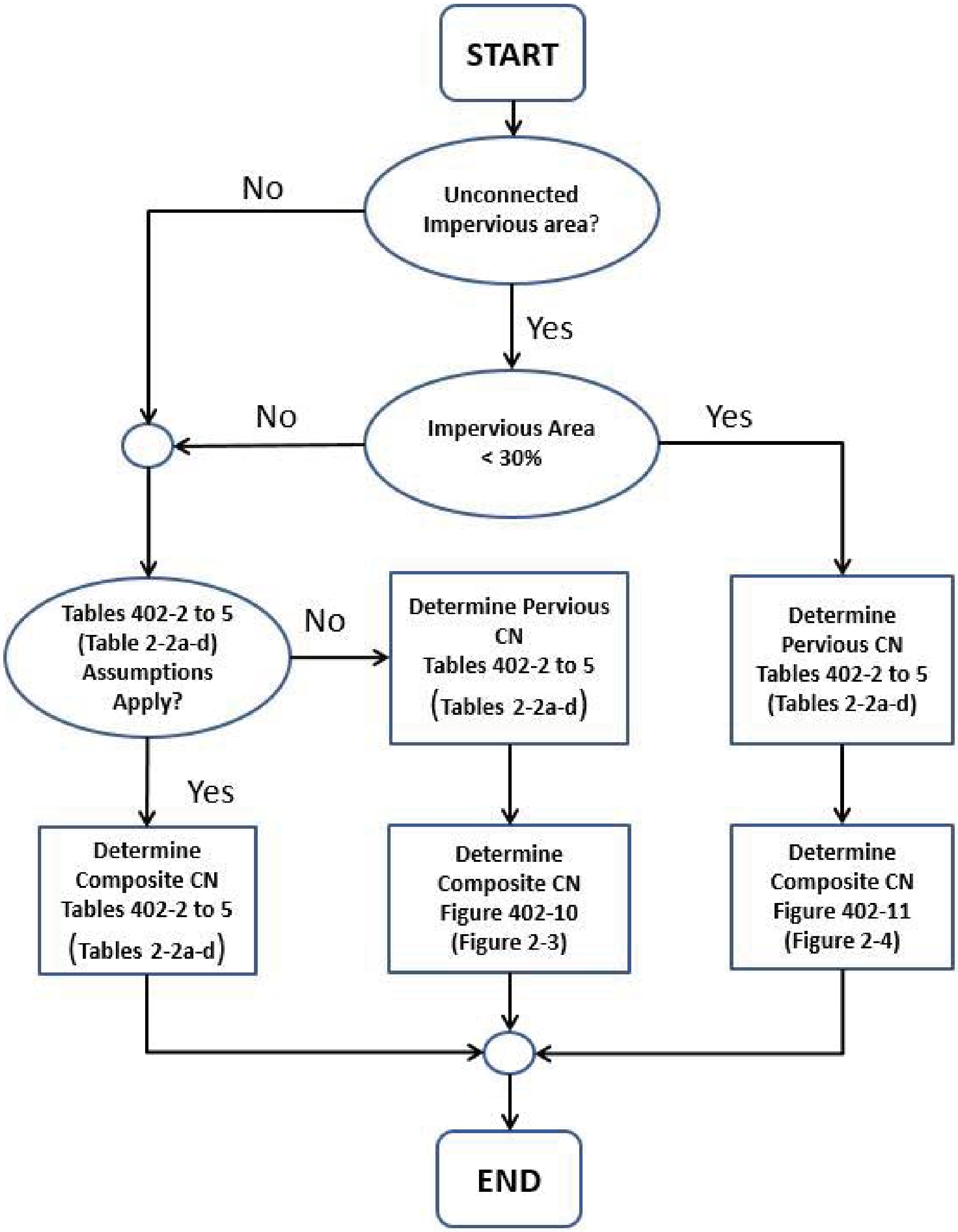
Storm Duration and Storm Recurrence Interval

TR-55 (NRCS, June 1986) states that "Normally a rainfall duration equal to or greater than the Time of Concentration (T_c) is used. Therefore, the rainfall distributions were designed to contain the intensity of any duration of rainfall for the frequency of the event chosen".

TR-55 (NRCS, June 1986) was developed based on the 24-hour rainfall depth (P_{24}) from various rainfall distributions. The Runoff (Q) Equation (**Equation 402-1**) presented in TR-55 was originally developed by the Soil Conservation Service (SCS, now the NRCS) prior to development of TR-55. The initial SCS runoff equation (**Equation 402-1**) was developed for various rainfall depths, without storm duration or recurrence interval limits.

Therefore, the TR-55 Direct Runoff Method (Q), may be applied to the 100-year recurrence interval storm and more frequent recurrence interval storms, and for storms of 24-hour duration and less. However, the 24-hour duration storm is required for NMDOT drainage analyses.

The decision process for determination of a Runoff Curve Number is presented in **Figure 402-9**.



Source: NRCS, June 1986, "TR-55, Urban Hydrology for Small Watersheds", Figure 2-2, p. 2-4.
https://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/stelprdb1044171.pdf

Figure 402-9 Flow Chart for Selecting the Appropriate Figure or Table for Determining Runoff Curve Numbers

Table 402-2 through **Table 402-5** (NRCS Tables 2-2 a-d) describe the effects of various cover and land use conditions for each of the four Hydrologic Soil Groups. Note that the CNs listed are for average runoff conditions. The index of runoff potential before a storm event is the Antecedent Runoff Condition (ARC), refer to **Section 404.5** for more information.

ARC is an attempt to account for the variation in CN at a site from storm to storm. CN for the average ARC at a site is the median value as taken from sample rainfall and runoff data. The amount of precipitation occurring in the five days preceding the storm in question is an indication of the ARC of the soil. Each ARC condition is defined here.

ARC I indicates dry watershed conditions that correlate with low runoff potential

ARC II indicates average watershed conditions that correlate with average runoff potential

ARC III indicates wet watershed conditions that correlate with high runoff potential

The CNs in **Table 402-2** to **Table 402-5** are for an average ARC II. New Mexico most often meets an ARC I or ARC II condition. Use ARC II for NMDOT Projects.

See “Part 630 Hydrology, National Engineering Handbook” (NRCS, 2004) for more detailed discussion of storm-to-storm variation and a demonstration of upper and lower enveloping curves.

Table 402-2 Runoff Curve Numbers for Urban Areas

Source: NRCS, June 1986, "TR-55, Urban Hydrology for Small Watersheds", Table 2-2a, p. 2-5.
https://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/stelprdb1044171.pdf

Table 2-2a Runoff curve numbers for urban areas [1]

Cover description: Cover type and hydrologic condition	Cover description: Average percent impervious area [2]	Curve numbers for hydrologic soil group: A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.) [3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only) [4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation) [5]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

1 Average runoff condition, and $I_a = 0.2S$.
2 The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions may be computed using figure 2-3 or 2-4.
3 CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.
4 Composite CN's for natural desert landscaping should be computed using figures 2-3 or 2-4 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.
5 Composite CN's to use for the design of temporary measures during grading and construction should be computed using figure 2-3 or 2-4 based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.

Table 402-3 Runoff Curve Numbers for Cultivated Agricultural Lands

Source: NRCS, June 1986, "TR-55, Urban Hydrology for Small Watersheds", Table 2-2b, p. 2-6.
https://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/stelprdb1044171.pdf

Table 2-2b Runoff curve numbers for cultivated agricultural lands [1]

Cover description			Curve numbers for hydrologic soil group			
Cover type	Treatment [2]	Hydrologic condition [3]	A	B	C	D
Fallow	Bare soil	—	77	86	91	94
	Crop residue cover (CR)	Poor	76	85	90	93
		Good	74	83	88	90
Row crops	Straight row (SR)	Poor	72	81	88	91
		Good	67	78	85	89
	SR + CR	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured (C)	Poor	70	79	84	88
		Good	65	75	82	86
	C + CR	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured & terraced (C&T)	Poor	66	74	80	82
		Good	62	71	78	81
	C&T+ CR	Poor	65	73	79	81
		Good	61	70	77	80
Small grain	SR	Poor	65	76	84	88
		Good	63	75	83	87
	SR + CR	Poor	64	75	83	86
		Good	60	72	80	84
	C	Poor	63	74	82	85
		Good	61	73	81	84
	C + CR	Poor	62	73	81	84
		Good	60	72	80	83
	C&T	Poor	61	72	79	82
		Good	59	70	78	81
	C&T+ CR	Poor	60	71	78	81
		Good	58	69	77	80
Close-seeded or broadcast legumes or rotation meadow	SR	Poor	66	77	85	89
		Good	58	72	81	85
	C	Poor	64	75	83	85
		Good	55	69	78	83
	C&T	Poor	63	73	80	83
		Good	51	67	76	80

1 Average runoff condition, and $I_a = 0.2S$
2 Crop residue cover applies only if residue is on at least 5% of the surface throughout the year.
3 Hydraulic condition is based on combination factors that affect infiltration and runoff, including (a) density and canopy of vegetative areas, (b) amount of year-round cover, (c) amount of grass or close-seeded legumes, (d) percent of residue cover on the land surface (good ≥ 20%), and (e) degree of surface roughness.

Poor: Factors impair infiltration and tend to increase runoff.

Good: Factors encourage average and better than average infiltration and tend to decrease runoff.

2–6 (210-VI-TR-55, Second Ed., June 1986)

Table 402-4 Runoff Curve Numbers for Other Agricultural Lands

Source: NRCS, June 1986, "TR-55, Urban Hydrology for Small Watersheds", Table 2-2c, p. 2-7.
https://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/stelprdb1044171.pdf

Table 2-2c Runoff curve numbers for other agricultural lands [1]

Cover description		Curve numbers for hydrologic soil group			
Cover type	Hydrologic condition	A	B	C	D
Pasture, grassland, or range—continuous forage for grazing. [2]	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow—continuous grass, protected from grazing and generally mowed for hay.	—	30	58	71	78
Brush—brush-weed-grass mixture with brush the major element. [3]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	30 [4]	48	65	73
Woods—grass combination (orchard or tree farm). [5]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods. [6]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30 [4]	55	70	77
Farmsteads—buildings, lanes, driveways, and surrounding lots.	—	59	74	82	86

1 Average runoff condition, and $I_a = 0.2S$.
2 *Poor:* <50%) ground cover or heavily grazed with no mulch.
Fair: 50 to 75% ground cover and not heavily grazed.
Good: > 75% ground cover and lightly or only occasionally grazed.
3 *Poor:* <50% ground cover.
Fair: 50 to 75% ground cover.
Good: >75% ground cover.
4 Actual curve number is less than 30; use CN = 30 for runoff computations.
5 CN's shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from the CN's for woods and pasture.
6 *Poor:* Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.
Fair: Woods are grazed but not burned, and some forest litter covers the soil.
Good: Woods are protected from grazing, and litter and brush adequately cover the soil.

Table 402-5 Runoff Curve Numbers for Arid and Semiarid Rangelands

Source: NRCS, June 1986, "TR-55, Urban Hydrology for Small Watersheds TR-55", Table 2-2d, p. 2-8.
https://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/stelprdb1044171.pdf

Table 2-2d Runoff curve numbers for arid and semiarid rangelands 1/

Cover description		Curve numbers for hydrologic soil group			
Cover type	Hydrologic condition 2/	A 3/	B	C	D
Herbaceous—mixture of grass, weeds, and low-growing brush, with brush the minor element.	Poor		80	87	93
	Fair		71	81	89
	Good		62	74	85
Oak-aspen—mountain brush mixture of oak brush, aspen, mountain mahogany, bitter brush, maple, and other brush.	Poor		66	74	79
	Fair		48	57	63
	Good		30	41	48
Pinyon-juniper—pinyon, juniper, or both; grass understory.	Poor		75	85	89
	Fair		58	73	80
	Good		41	61	71
Sagebrush with grass understory.	Poor		67	80	85
	Fair		51	63	70
	Good		35	47	55
Desert shrub—major plants include saltbush, greasewood, creosotebush, blackbrush, bursage, palo verde, mesquite, and cactus.	Poor	63	77	85	88
	Fair	55	72	81	86
	Good	49	68	79	84

1 Average runoff condition, and $I_a = 0.2S$. For range in humid regions, use table 2-2c.
2 Poor: <30% ground cover (litter, grass, and brush overstory).
Fair: 30 to 70% ground cover.
Good: > 70% ground cover.
3 Curve numbers for group A have been developed only for desert shrub.

The effects of urbanization, including the amount and connectedness of the impervious areas, has been studied by the NRCS, and a method for assessing the degree to which runoff is affected has been developed and is described below.

Connected Impervious Areas

An impervious area is considered connected if runoff from it flows directly into the drainage system. It is also considered connected if runoff occurs as shallow concentrated flow that runs over a pervious area and then flows into the drainage system, with the logic being that the losses within the pervious reach would be minimal in that circumstance.

Urban CNs related to **Table 402-2** (NRCS Table 2-2a) were developed for typical land use relationships based on specific assumed percentages of impervious area. These CN values were developed on the assumptions that (a) pervious urban areas are equivalent to pasture in good hydrologic condition and (b) impervious areas have a CN of 98 and are directly connected to the drainage system. Some assumed percentages of impervious area are shown in **Table 402-2.**

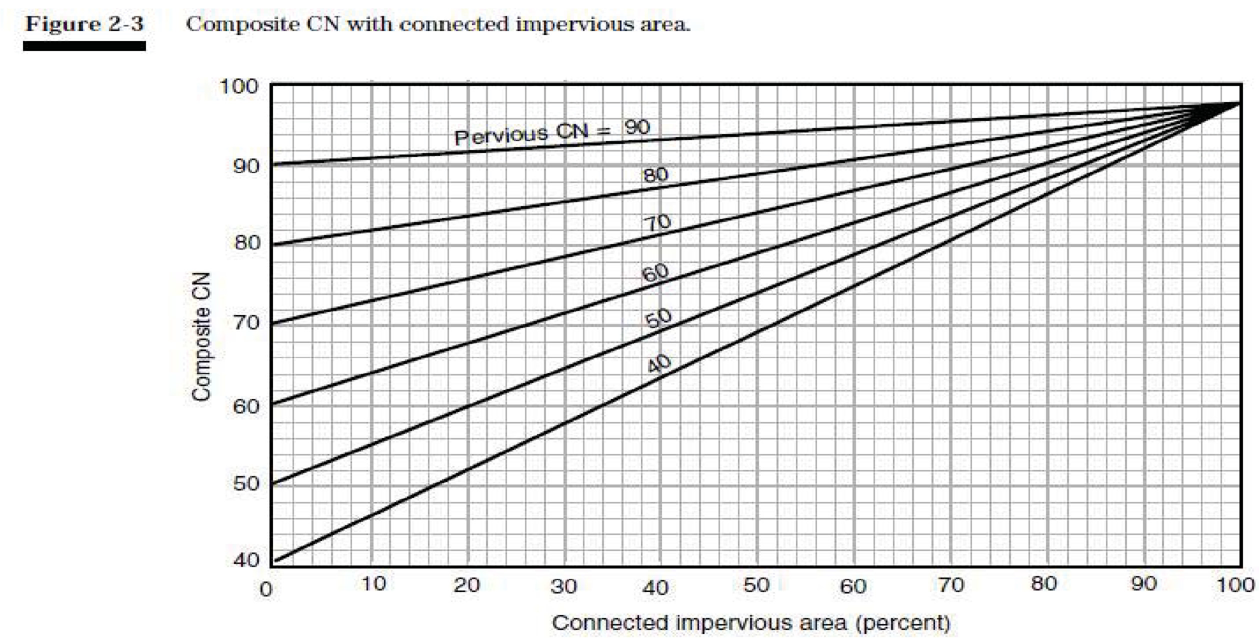
If not all of the impervious area is directly connected to the drainage system, and the impervious area percentages or the pervious land use assumptions in **Table 402-2** are not applicable, use **Figure 402-10** to compute a composite CN.

For example, a ½-acre lot in HSG B, with an assumed impervious area of 25 percent has a CN of 70. Assume that 20% of the impervious area is directly connected and assume the pervious area CN=61. Apply those values in **Figure 402-10** and a composite CN of 68 is determined. The difference between CN= 70 and 68 is because less runoff will be generated from the 80% impervious area that must pass through a pervious area (or not directly connected area), and therefore additional runoff will be infiltrated within the pervious area.

Unconnected Impervious Areas

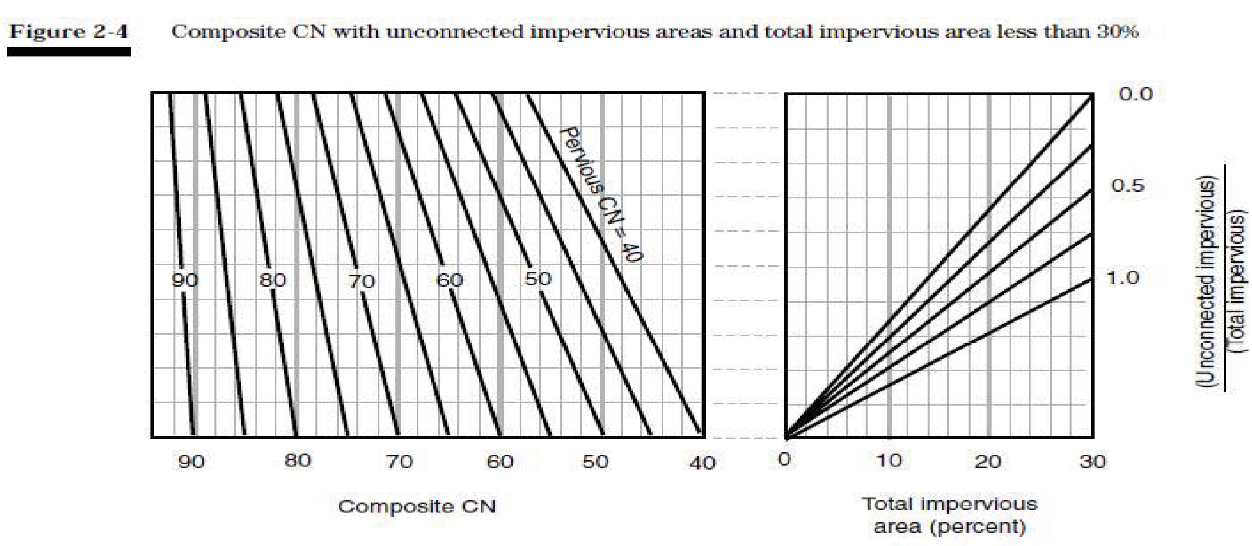
Runoff from unconnected (disconnected) impervious areas is that which spreads over a pervious area as sheet flow. To determine CN when all or part of the impervious area is not directly connected to the drainage system,

1. Use **Figure 402-10** if the total impervious area is greater than or equal to 30 percent, because the absorptive capacity of the remaining pervious areas will not significantly affect runoff.
2. Use **Figure 402-11** if the total impervious area is less than 30 percent.



Source: NRCS, June 1986, "TR-55, Urban Hydrology for Small Watersheds", Figure 2-3, p. 2-10.
https://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/stelprdb1044171.pdf

Figure 402-10 Composite CN with Connected Impervious Areas



Source: NRCS, June 1986, "TR-55, Urban Hydrology for Small Watersheds", Figure 2-4, p. 2-10.
https://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/stelprdb1044171.pdf

Figure 402-11 Composite CN with Unconnected Impervious Areas and Total Impervious Areas Less Than 30%

When impervious area is less than 30 percent, obtain the composite CN by entering the right side of **Figure 402-11** with the percentage of total impervious area and the ratio of total unconnected impervious area to total impervious area. Then move left to the appropriate pervious CN and read down to find the composite CN. For example, for a 1/2-acre lot with 20 percent total impervious area (75 percent of which is unconnected) and pervious CN of 61, the composite CN from **Figure 402-11** is 66. If all of the impervious area is connected, the resulting CN (from **Figure 402-10**) would be 68.

Limitations of the Runoff Curve Number Method

- Use the Runoff Curve Number Method with caution when re-creating specific features of an actual storm. The foundational rainfall/runoff equation does not contain an expression for time and, therefore, does not account for rainfall duration or intensity.
- Runoff from snowmelt or rain on frozen ground cannot be estimated using these procedures.
- The NRCS runoff procedures apply only to direct surface runoff; do not overlook large sources of subsurface flow or high ground water levels that contribute to streamflow. These conditions are often related to HSG A soils and forest areas that have been assigned relatively low CNs in **Table 402-4**. Good judgement and experience based on stream gage records are needed to adjust CNs as conditions warrant. *Note that this condition rarely impacts design decisions in New Mexico.*
- When the weighted CN is less than 40, use 40.

402.6.1 Curve Number Weighting

Examination of **Figure 402-8** reveals that the rainfall/runoff relationship described by the NRCS Curve Number (CN) Method is not linear for small rainfall amounts. This effect is most dramatic for lower CNs, therefore, when hydrologic conditions are reasonably consistent throughout the watershed, the use of a single CN is appropriate. For watersheds where CNs vary by 10 or less, an Area Weighted Curve Number is appropriate. When CNs vary by more than 10 within the basin or subasin, either subdivide the watershed into smaller drainage subbasins to obtain similar CNs, or use a Runoff Weighted Curve Number. Examples of each CN weighting procedure are shown below.

Area Weighted Curve Number

Assume a design rainfall event of 2.0 inches.

40% of the drainage basin is characterized by CN=65

60% of the drainage basin is characterized by CN=88

the area weighted CN = $\frac{(0.40)\times(65)+(0.60)\times(88)}{100}$ = 78.8 **use CN=79**

The runoff resulting from 2.0 inches of rainfall and a CN of 79 = 0.52 inches

Runoff Weighted Curve Number

40% of the drainage basin is characterized by CN=65

60% of the drainage basin is characterized by CN= 88

Use **Figure 402-8** or **Equation 402-1** to estimate 0.14 inches of direct runoff from the CN=65 land and 0.97 inches of direct runoff from the CN=88. **Equation 402-1** will provide more accurate results.

The weighted runoff is calculated by:

$$Q = (0.40) \times (0.14) + (0.60) \times (0.97) = 0.64 \text{ inches}$$

Use **Figure 402-8** to find a runoff weighted CN that will produce 0.64 inches of runoff from a 2.0 inch rainfall event, **CN=82**.

Comparison of Methods

Recall that by the Area Weighted Method, a CN = 79 was obtained. The Runoff Weighted Method determined that CN=82. The runoff difference between these CNs in this example is approximately 0.12 inches of direct runoff (a 23% increase in runoff volume).

Summary

Use the criteria described above to select the correct CN weighting method. Using the Runoff Weighted Curve Number Method requires more effort but will always produce the correct results. The Area Weighted Runoff Method is easier, gives reasonable results, and may be used when CN values vary by less than 10.

402.7 Other Land Use Effects

Recognize that both the Rational Formula Method Runoff Coefficient (C) and the Runoff Curve Number (CN) are lumped runoff parameters. This means that in most cases runoff volumes and sometimes peak rates incorporate all the losses to rainfall from the time it hits the ground until it reaches the analysis point, including canopy wetting, filling of minor depression storage, infiltration, evaporation, and transmission losses. In the case of the Rational Formula Method Coefficient (C), it includes any hydrologic routing effects as well.

Therefore, land use patterns, in addition to the relationship between rainfall and runoff volumes governed by the Soil-Cover Complex and the Rational Formula Method Runoff Coefficient (C) and the Runoff Curve Number (CN), affect the timing of runoff, how subbasins interact with the main stem of the stream system, and ultimately the shape and magnitude of the runoff hydrograph. Note that these effects are not linear. Doubling the rainfall may result in much higher than doubled peak runoff rates and volumes while doubling the drainage area may not have the same relative effect. The types of land use can also have a significant impact on water quality, even between two subbasins with identical soils and percentage imperviousness. Another often overlooked effect of land use is the relative location of the various land uses within a watershed. Further description of land use impacts is found in **Section 405**.

402.8 Travel Time, Lag, and Time of Concentration

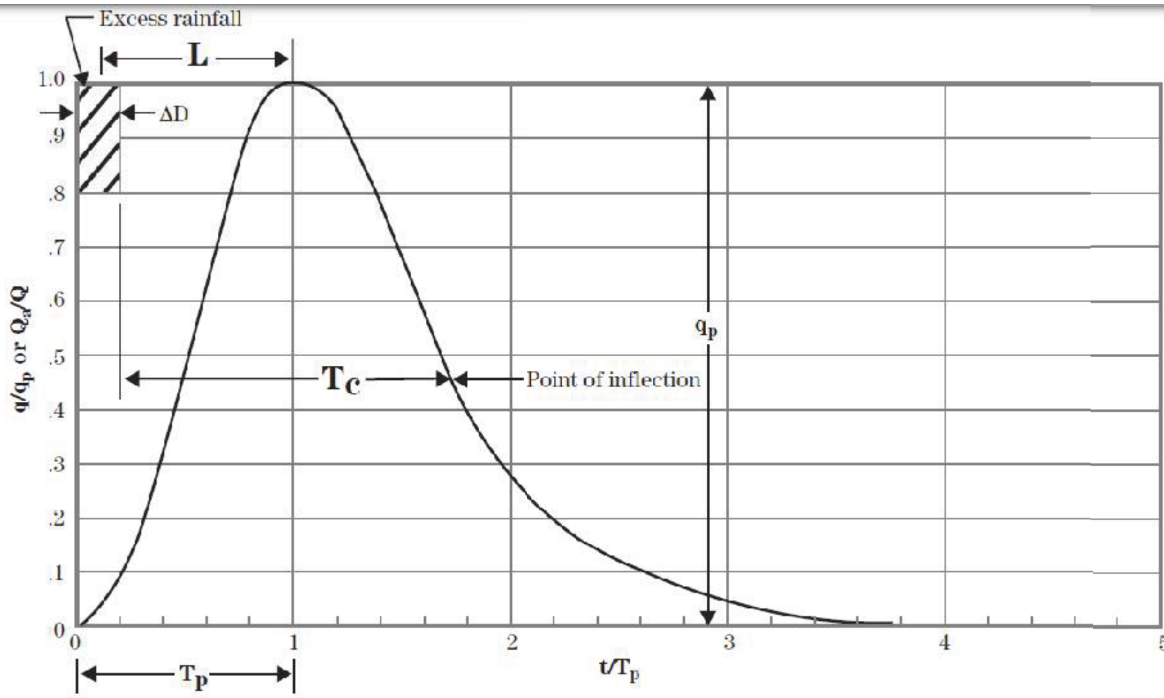
Travel Time (Tt) is the time it takes water to travel from one location to another.

Lag (L) is the delay between the *centroid of excess rainfall* from a rainfall event over a watershed until runoff reaches its maximum flow rate. Conceptually, lag may be thought of as a weighted Time of Concentration (Tc) where, if for a given storm, the watershed is divided into subbasins, the time required for each subbasin runoff to arrive at the outfall is related to the watershed peak by the relative contribution of each subbasin runoff in its individual lag time. In general, hydrologic modeling practice using the NRCS Unit Hydrograph Method, lag is a function of Tc.

Time of Concentration (Tc) is defined as the time required for excess precipitation (runoff) to travel from the hydraulically most remote part of the watershed to the point of interest. Peak rate calculations are very sensitive to Tc; therefore, it is one of the most important drainage basin characteristic needed to calculate the peak rate of runoff. Tc is a simplified proxy for the hydrologic response to precipitation by a watershed, capturing the effects of size, shape, length and slope of the basin or subbasin. The Tc for a watershed or subbasin has the most dramatic effect on the shape of the runoff hydrograph of any parameter. Therefore, accurate estimation of a watershed's Tc is crucial to every type of hydrologic modeling.

The method used to calculate Tc must be appropriate to the hydrologic analysis method selected for design. Engineers working on NMDOT projects must use the Time of Concentration methods specified in this section for each hydrologic method.

Figure 402-12 for a graphical explanation of L and Tc, and their relationship to one another.



where:

L = Lag, h
T_c = time of concentration, h
T_p = time to peak, h
ΔD = duration of excess rainfall, h
t/T_p = dimensionless ratio of any time to time to peak
q = discharge rate at time t, ft^3/s
q_p = peak discharge rate at time T_p, ft^3/s
Q_a = runoff volume up to t, in
Q = total runoff volume, in

Source: NRCS, 2010, "Part 630 Hydrology, National Engineering Handbook, Chapter 15 Time of Concentration", Figure 15-3, p. 15-4.
http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=27002.wba

Figure 402-12 Graphical Representation of Relationships Between Lag, Tp and Tc

Table 402-6 defines the appropriate Time of Concentration method to be used for each hydrologic method.

Table 402-6 Selecting a Time of Concentration Calculation Method

Hydrologic Method	**Watershed Condition**	**Time of Concentration Method**
Rational Formula Method (**Section 403**)	Un-gullied Watershed*	Upland Method
	Gullied Watershed*	Kirpich Equation (Kerby-Kirpich Method for Valley Areas)
Simplified Peak Discharge Method (**Section 404**)	Un-gullied Watershed*	Upland Method
	Gullied Watershed*	Kirpich Formula (Kerby-Kirpich Method for Valley Areas)
	Watershed Partially Gullied	Upland Method for the Un-Gullied Portion, then Kirpich Equation for the Gullied Portion
USGS Regression Equations	varies	Not Required
Unit Hydrograph Method (**Section 405**)	No Defined Stream Channel	Upland Method
	Defined Stream Channel	Iterative Method within the Stream Hydraulic Method
Approved Urban Method	All Conditions	Use Tc Method Specified for the Approved Urban Method

*A watershed is considered un-gullied if 10% or less of the primary watercourse exhibits gullying.

Within each watershed, the engineer begins by locating the flow path to the most hydraulically remote point in the watershed. This is the flow path that extends from the bottom of the watershed, or drainage structure, to the most hydraulically distant (in time) point in the watershed. Generally, this process is begun at the bottom of the watershed and is continued upstream until the longest (in time) flow path has been found. At the top of the watershed, a defined watercourse may not exist. In these areas, overland flow will be the dominant flow type. As the runoff proceeds downstream, overland flows will naturally begin to coalesce, gradually concentrating together. Shallow concentrated flow often has enough force to shape small gullies in erosive soils. Gullies eventually combine until a well-defined stream channel is formed. The watercourse is, often at this point, large enough to be identified on a USGS quadrangle topographic map, or clearly visible in aerial photography depending on its quality.

Reaches along the primary watercourse should be divided into those which are hydraulically similar. In larger watersheds, the reaches may be sufficiently distinct to justify separate estimates of Tc for each reach of the watercourse. Tc in any given watershed is simply the sum of travel times within hydraulically similar reaches along the most remote (in time) flow path. Tc is determined from measured reach lengths and estimated average reach velocities.

The basic equation for Time of Concentration is:

$$Tc = \frac{\left[\frac{L_1}{V_1} + \frac{L_2}{V_2} + \frac{L_3}{V_3} + \frac{L_n}{V_n}\right]}{60} \qquad \textbf{402-3}$$

for minutes (or divide by 360 rather than 60 if Tc in hours is required)

where:

Tc	=	Time of Concentration, minutes (or hours depending on method)
V_1	=	average flow velocity in the uppermost reach of the watercourse, ft/s
L_1	=	length of the uppermost reach of the watercourse, ft
V_2, $V_3 \ldots V_n$	=	average flow velocities in subsequent reaches progressing downstream, ft/s
L_2, $L_3 \ldots L_n$	=	lengths of subsequent reaches progressing downstream, ft

Tc is the time required for runoff to travel from the hydraulically most distant point in the watershed to the outlet. The hydraulically most distant point is the point with the longest travel time to the watershed outlet, and not necessarily the point with the longest flow distance to the outlet, see **Figure 402-13**.

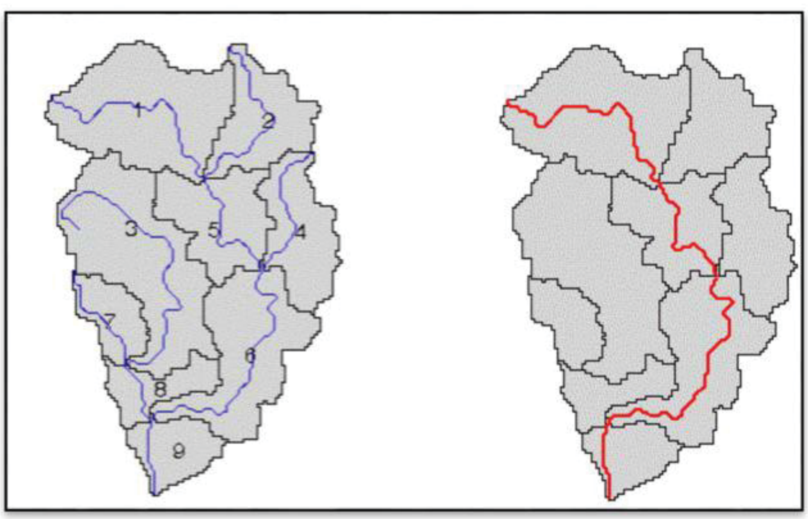


Figure 402-13 Longest Travel Time Illustration in Basin

Time of Concentration (Tc) is generally applied only to surface runoff and may be computed using many different methods. Tc will vary depending upon slope and character of the watershed and the flow path. In hydrograph analysis, Tc is the time from the end of excess rainfall to the point on the falling limb of the dimensionless unit hydrograph (point of inflection) where the recession curve begins, see **Figure 402-12**.

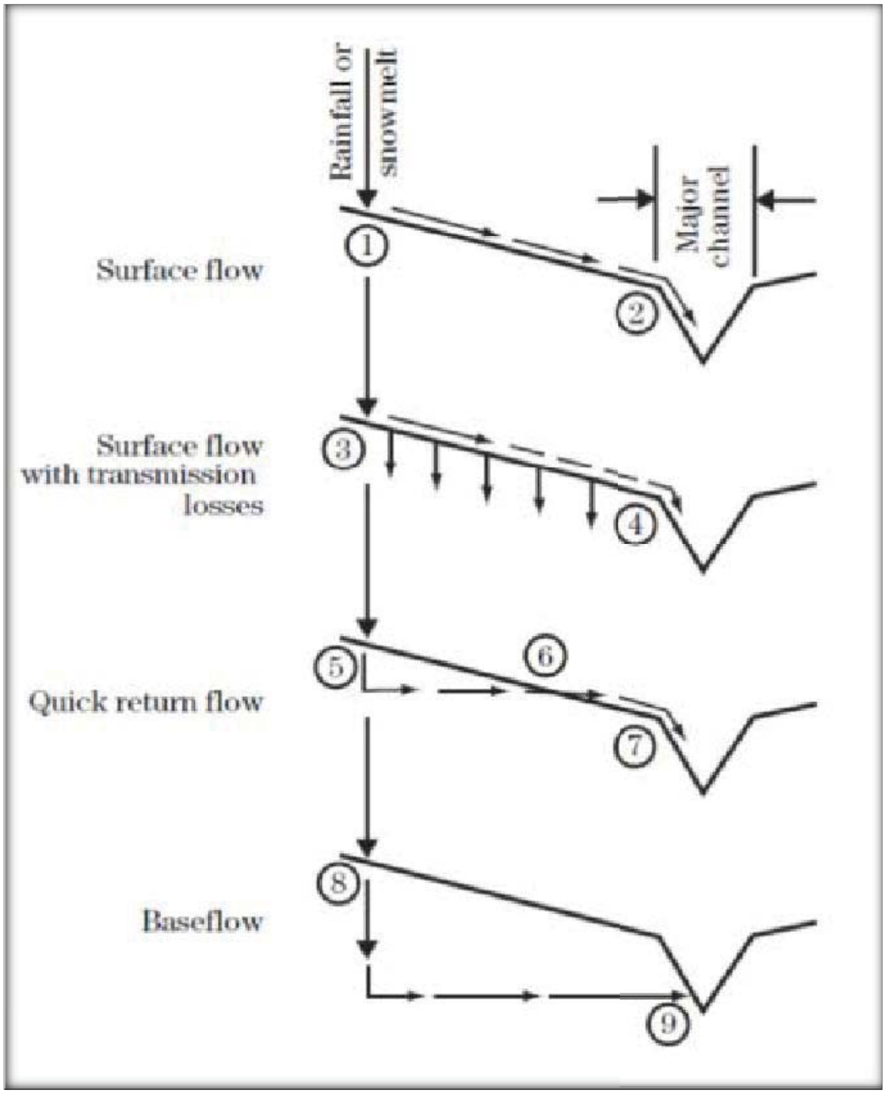
Tc can be estimated using one of the methods listed in **Table 402-6**, depending on the application and circumstances. In cases where only a peak discharge and/or hydrograph are desired at the watershed outlet and watershed characteristics are fairly homogenous, the watershed may be treated as a single basin. However, if land use, Hydrologic Soil Group, slope, or other watershed characteristics are not homogeneous throughout the watershed, or the basin is large enough that the assumption of one rainfall amount is not appropriate, then divide the watershed into smaller subbasins, which requires a Tc estimation for each subbasin. Hydrographs are then developed for each subbasin and routed appropriately to a point of reference using the methods described in **Section 405.11**.

Note: Peak rates of runoff are extremely sensitive to small changes in Tc. For this reason, it is very important that the physical processes and hydraulic principles involved are very well understood and that procedures used to estimate the Tc are valid and uniformly applied.

Rainfall over a watershed (that reaches the ground) will generally follow one of four potential paths:

- Some rain will be intercepted by vegetation and evaporate into the atmosphere
- Some rain will fall onto the ground surface and evaporate
- Some rain will infiltrate into the soil
- Some rain will run directly off from the ground surface

Depending on total storm rainfall and a variety of other factors, a portion of the stormwater runoff will drain to the stream system. There are four types of flow that may occur singly or in combination throughout the watershed as presented in **Figure 402-14**.



Source: NRCS, 2010, "Part 630 Hydrology, National Engineering Handbook, Chapter 15 Time of Concentration", Figure 15-1, p. 15-2.
http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=27002.wba

Figure 402-14 Types of Flow

Relation between Lag, Time to Peak, and Time of Concentration

Lag Time (L), Time to Peak (Tp), and Time of Concentration (Tc) are often misunderstood. When these terms are encountered in the documents referenced in this manual, it is important to understand each of them and their relationships to one another. The following is offered to assist in that understanding.

Researchers (Mockus 1961; Simas 1996) found that **Figure 402-12** graphically portrays the relationship between average natural watershed conditions and an approximately uniform distribution of runoff.

$$L = 0.6 \times Tc \qquad \textbf{402-4}$$

(NRCS, 2010, “Part 630 Hydrology, National Engineering Handbook, Chapter 15 Time of Concentration”, Eq. 15-3, p. 15-3)
http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=27002.wba

where:

L	=	Lag, hr
Tc	=	Time of Concentration, hr

When runoff is not uniformly distributed due to significant differences in slope, drainage patterns, soils cover, and land use in a watershed, the watershed should be subdivided into subbasins with nearly uniform runoff characteristics so that **Equation 402-4** can be applied to each subbasin.

Four methods to calculate Tc presented in this manual are:

- The Upland Method
- The Kirpich Equation
- Kerby Equation
- The Kerby-Kirpich Method
- The Iterative Method within the Stream Hydraulic Method

402.9 Time of Concentration

402.9.1 The Upland Method

The Upland Method (also known as the Velocity Method) is used to estimate travel times for overland flow and shallow concentrated flow conditions. The Upland Method is used for the ungullied portion of the primary watercourse when the overland flow length is 300 feet or less.

The Upland Method was originally developed by the Soil Conservation Service (SCS), which is now the Natural Resource Conservation Service (NRCS). The Upland Method is described in Chapter 15 Time of Concentration of “Part 630 Hydrology, National Engineering Handbook” (NRCS, 2010). Note that in the current (2010) version of Chapter 15, the NRCS has renamed the “Upland Method” to the “Velocity Method.” However, many documents still refer to it as the “Upland Method” and, therefore, the name “Upland Method” is used in this Drainage Design Manual.

The Upland Method is limited to use in watersheds that are less than 2,000 acres in size, or to the upper reaches of larger watersheds. For NMDOT projects the Upland Method may be used for computing the Time of Concentration when using the Rational Formula Method or the Simplified Peak Discharge Method on a largely un-gullied watershed. A watershed is considered un-gullied when 10% or less of the most hydraulically remote flow path exhibits gullying.

Water moves through a watershed as sheet flow, shallow concentrated flow, open channel flow, or some combination of these. The type of flow that occurs is a function of the conveyance system and is best determined by field inspection.

Travel time (Tt) is the ratio of flow length to flow velocity:

$$Tt = \frac{L}{3600 \times V} \qquad \textbf{402-5}$$

(NRCS, 2010, “Part 630 Hydrology, National Engineering Handbook, Chapter 15 Time of Concentration”, Eq. 15-1, p. 15-2)
http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=27002.wba

where:

Tt	=	travel time, hr
L	=	flow length, ft
V	=	average velocity, ft/s
3600	=	conversion factor from seconds to hours

Time of Concentration (Tc), is the sum of Travel Time (Tt) values for the various consecutive flow segments:

$$Tc = T_t + T_2 + T_3 \ldots T_n \qquad \textbf{402-6}$$

(NRCS, 2010, “Part 630 Hydrology, National Engineering Handbook, Chapter 15 Time of Concentration”, Eq. 15-7, p. 15-6)
http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=27002.wba

where:

Tc	=	Time of Concentration, hr
T_n	=	number of flow segments

Sheet Flow

At the top to the watershed, sheet flow is generally the predominant flow regime. Sheet flow is defined as flow over plane surfaces. Sheet flow usually occurs in the headwaters of a stream near the ridgeline that defines the watershed boundary. Typically, sheet flow occurs for no more than 100 to 300 feet before transitioning to shallow concentrated flow (Merkel, 2001).

A simplified version of the Manning’s Kinematic Equation may be used to compute travel time for sheet flow. This simplified form of the Kinematic Equation presented here was developed by (Welle and Woodward,1986) after studying the impact of various parameters on the estimates.

$$Tt = \frac{0.007(\, n\, l\,)^{0.8}}{(\, P_2\,)^{0.5}\; S^{0.4}} \qquad \textbf{402-7}$$

(NRCS, 2010, “Part 630 Hydrology, National Engineering Handbook, Chapter 15 Time of Concentration”, Eq. 15-8, p. 15-6)
http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=27002.wba

where:

Tt	=	travel time, hr
n	=	Manning's roughness coefficient (**Table 402-7**)
l	=	sheet flow length, ft
P_2	=	2-year, 24-hour rainfall, in.
S	=	slope of land surface, ft/ft

This simplification is based on the following assumptions:

- Shallow steady uniform flow
- Constant rainfall excess intensity (that part of a rain available for runoff) both temporally and spatially
- 2-year, 24-hour rainfall assuming standard NRCS rainfall intensity-duration relations apply (Types I, II, and III)
- Minor effect of infiltration on travel time

For sheet flow, the roughness coefficient includes the effects of roughness and the effects of raindrop impact including drag over the surface; obstacles such as litter, crop row ridges, and rocks; and erosion and sediment transport. These "n" values are only applicable for flow depths of approximately 0.1 foot or less, where sheet flow occurs. **Table 402-7** gives roughness coefficient values for sheet flow for various surface conditions.

Table 402-7 Roughness Coefficients (Manning's "n") for Sheet Flow

Source: NRCS, 2010, "Part 630 Hydrology, National Engineering Handbook, Chapter 15 Time of Concentration", Table 15-1, p. 15-6.
http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=27002.wba

Surface description	"n" [1/]
Smooth surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils:0.	
Residue cover ≤20%	0.06
Residue cover >20%	0.17
Grass:	
Short grass prairie	0.15
Dense grasses [2/]	0.24
Bermuda grass	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.

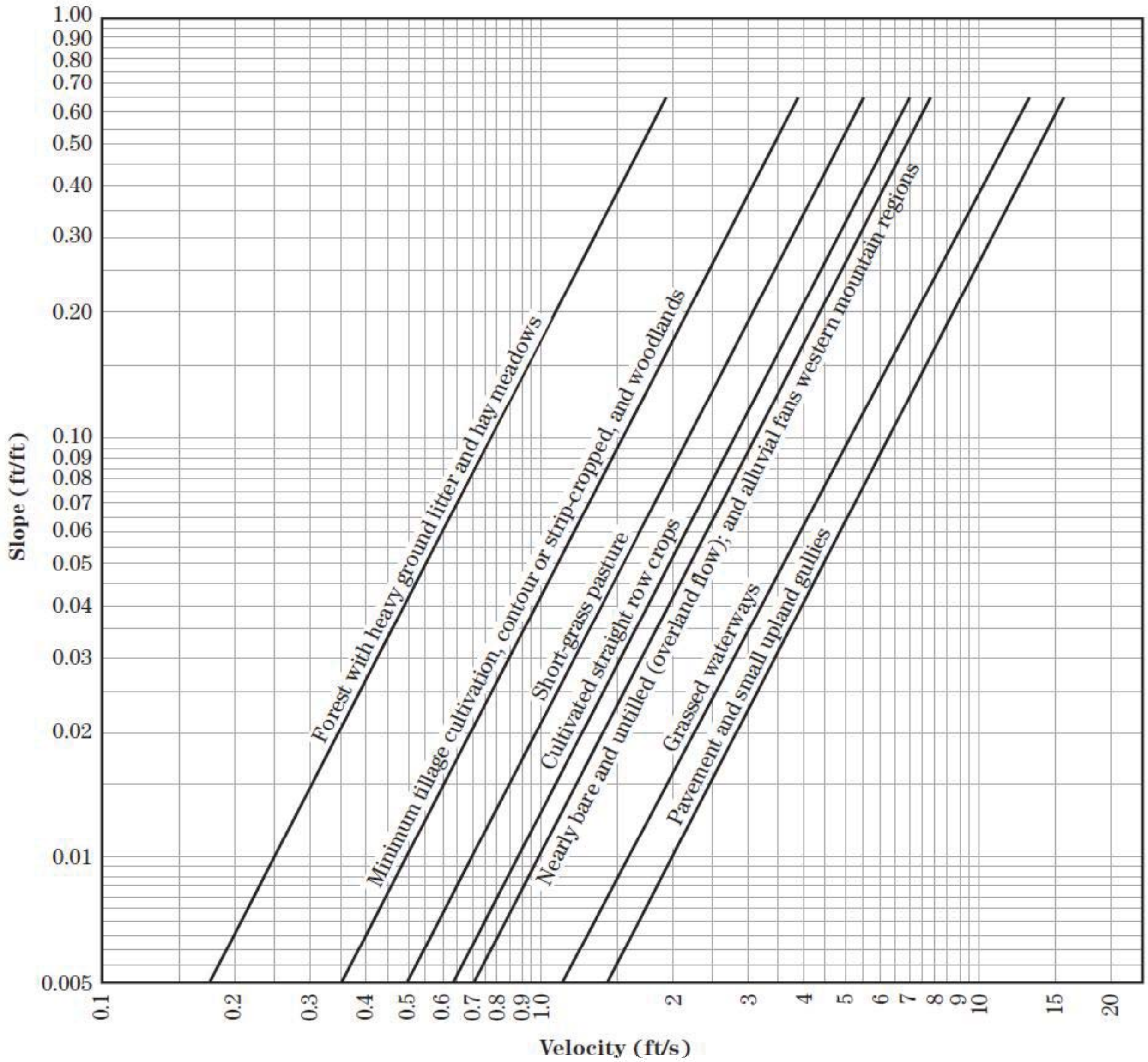
It is important to note that there are many locations in New Mexico where there is little or no runoff resulting from a 2-year storm and that due to the combination of high desert climate and soils in the upper portions of many watersheds, there is no evidence of gully formation for distances far exceeding 100 to 300 feet. However, the maximum sheet flow length used for NMDOT hydrologic analyses should not exceed 300 feet, except when a greater length can be justified by onsite inspection of the upper watershed or through inspection of high resolution aerial photography.

Overland flow continues until the volume of water is sufficient to create a shallow concentrated flow regime. In erosive soil formations with limited ground cover, the length of overland flow may be so short that it is negligible. Given the slope of the land and some knowledge of the ground cover conditions, once the most hydraulically remote flow path is determined, the overland flow length can be determined.

For NMDOT projects, shallow concentrated flow is assumed to occur from the end of overland flow to the bottom of a watershed where there is little or no gullying (10% or less). Where gullying is evident in the majority of the watershed (by field inspection, aerial photography or by a blue line shown on the USGS quadrangle topographic map), the Time of Concentration should be computed by the Kirpich Equation for the entire watershed. When the Simplified Peak Discharge Method is being used for NMDOT projects, the Upland Method may be used for the un-gullied portion of the watercourse, in combination with the Kirpich Equation for the gullied sections of the watercourse. For watersheds with more than 30% of the uplands or with little or no gullying (valley areas), the Kerby-Kirpich Method should be used. The NMDOT Drainage Design Bureau can be contacted to obtain a copy of a spreadsheet to determine Tc using these methods. Note that the Engineer/Consultant is responsible for understanding the use of, and the accuracy of the results from this spreadsheet.

Shallow Concentrated Flow

After approximately 100 to 300 feet, sheet flow usually becomes shallow concentrated flow collecting in swales, small rills, and gullies. Shallow concentrated flow is assumed not to have a well-defined channel and has flow depths of 0.1 to 0.5 feet. It is assumed that shallow concentrated flow can be represented by one of seven flow types. **Figure 402-15** presents curves as Velocity versus Slope for Shallow Concentrated Flow and these curves were used to develop the information in **Table 402-8**. To estimate shallow concentrated flow travel time, velocities are developed using **Figure 402-15**, in which average velocity is a function of watercourse slope and type of channel (Kent, 1973). For slopes less than 0.005 feet per foot, the equations in **Table 402-8** may be used. After estimating average velocity using **Figure 402-15**, use **Equation 402-5** to estimate travel time for the shallow concentrated flow segment.



Source: NRCS, 2010, "Part 630 Hydrology, National Engineering Handbook, Chapter 15 Time of Concentration", Figure 15-4, p. 15-8.
http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=27002.wba

Figure 402-15 Velocity Versus Slope for Shallow Concentrated Flow

Table 402-8 Equations and Assumptions Developed from Figure 402-15

Source: NRCS, 2010, "Part 630 Hydrology, National Engineering Handbook, Chapter 15 Time of Concentration", Table 15-3, p.15-8.
http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=27002.wba

Flow type	Depth (ft)	Manning's *n*	Velocity equation (ft/s)
Pavement and small upland gullies	0.2	0.025	$V = 20.328(s)^{0.5}$
Grassed waterways	0.4	0.050	$V=16.135(s)^{0.5}$
Nearly bare and untilled (overland flow); and alluvial fans in western mountain regions	0.2	0.051	$V=9.965(s)^{0.5}$
Cultivated straight row crops	0.2	0.058	$V=8.762(s)^{0.5}$
Short-grass pasture	0.2	0.073	$V=6.962(s)^{0.5}$
Minimum tillage cultivation, contour or strip-cropped, and woodlands	0.2	0.101	$V=5.032(s)^{0.5}$
Forest with heavy ground litter and hay meadows	0.2	0.202	$V=2.516(s)^{0.5}$

For that portion of the flow path that is channel flow, use Manning's Equation (**Equation 402-10**) to calculate the velocity. The approach outlined in **Section 402.9.5** should be followed to determine the average velocity for the channel reaches.

Once the reach lengths and flow velocities for each defined reach along the flow path have been calculated as described above, the Tc for each of the segments are added together to find the total Tc.

402.9.2 Time of Concentration by the Kirpich Equation

The Kirpich Equation should be used in watersheds when gullying (including manmade conveyances in fully urbanized watersheds such as curb and gutter, storm drains and channels) is evident in more than 10% of the primary watercourse**.** Gullying can be assumed if a blue line appears on the watercourse shown on the USGS quadrangle topographic map or is apparent from field investigation or from inspection of aerial photography. The Kirpich Equation is given as:

$$Tc = 0.0078 \times L^{0.77} \times S^{-0.385} \qquad \textbf{402-8}$$

(TxDOT, July 2016, "Hydraulic Design Manual", Eq. 4-15, p. 4-39)
http://onlinemanuals.txdot.gov/txdotmanuals/hyd/index.htm

where:

Tc	=	Time of Concentration, minutes
L	=	maximum length of water travel, ft
S	=	surface slope, given by H/L, ft/ft
H	=	difference in elevation between the most hydraulically remote point in the drainage basin and the outlet, ft

In small watersheds where the slope is flat, and the flow path of the hydraulically longest flow path is dominated by overland flow greater than 300 feet, the Kerby Equation should be considered for the overland flow portion and Kirpich Equation for the channelized portion.

In gullied (and in fully urbanized) basins, the Kirpich Equation should generally be used for the entire drainage basin. The exception to this rule occurs when the Simplified Peak Discharge Method is being used on NMDOT projects or when the watercourse has a mixture of gullied and un-gullied sections. In these situations, mixing of Time of Concentration methods is allowed and is called the Kerby-Kirpich Method as described in **Section 402.9.4.**

402.9.3 Time of Concentration by the Kerby Equation

For small watersheds where overland flow and overland flow length are an important component of overall travel time, the Kerby Equation can be used. The Kerby Equation is:

$$T_{OV} = K\ (L \times N)^{0.467} \times S^{-0.235} \qquad \textbf{402-9}$$

(TxDOT, July 2016, "Hydraulic Design Manual", Eq. 4-14, p. 4-37)
http://onlinemanuals.txdot.gov/txdotmanuals/hyd/index.htm

where:

T_{OV}	=	overland flow Time of Concentration, minutes
K	=	a unit conversion coefficient, in which K = 0.828
L	=	the overland-flow length, feet
N	=	a dimensionless retardance coefficient
S	=	the dimensionless slope of terrain conveying the overland flow

In the development of the Kerby Equation, the length of overland flow was as much as 1,200 feet. This length is considered an upper limit, and in practice, shorter values generally are expected. The dimensionless retardance coefficient used is similar in concept to the well-known Manning's roughness coefficient; however, for a given type of surface, the retardance coefficient for overland flow will be considerably larger than for open-channel flow. Typical values for the retardance coefficient are listed in **Table 402-9**. Roussel et al., 2005, recommends that the user should not interpolate the retardance coefficients in **Table 402-9.** If it is determined that a low slope condition or a transitional slope condition exists, the user should consider using an adjusted slope in calculating the Time of Concentration.

Table 402-9 Kerby Equation Retardance Coefficient Values

Source: TxDOT, July 2016, “Hydraulic Design Manual”, Table 4-5, p. 4-38.
http://onlinemanuals.txdot.gov/txdotmanuals/hyd/index.htm

Generalized Terrain Description	Dimensionless Retardance Coefficient (N)
Pavement	0.02
Smooth, bare, packed soil	0.10
Poor grass, cultivated row crops, or moderately rough packed surfaces	0.20
Pasture, average grass	0.40
Deciduous forest	0.60
Dense grass, coniferous forest, or deciduous forest with deep litter	0.80

402.9.4 The Kerby-Kirpich Method

The Upland Method is used for the ungullied portion of the primary watercourse when the overland flow length is 300 feet or less. The Kerby Equation should be used for the ungullied portions when the overland flow length is greater than 300 feet. The Kirpich Equation is used for the gullied portion of the watercourse, including those drained by manmade conveyances such as curb and gutter, storm drains and channels. The Tc result from each equation are added to obtain the watershed total Tc, thus the name “Kerby-Kirpich” Method.

402.9.5 The Iterative Method Within the Stream Hydraulic Method

The Iterative Method within the Stream Hydraulic Method is used when calculating peak discharges by the Unit Hydrograph Method in a watercourse where a defined stream channel is evident in the field or aerial photography (or a blue line, solid or broken, on a quadrangle topo map) and is the dominant runoff conveyance in the watershed. The Iterative Method within the Stream Hydraulic Method is applicable principally on larger basins where the longest flow path is dominated by channel flow, but that are small enough not to warrant subdividing the basin, or in basins where gullying is evident all the way to the top of the basin.

The engineer must measure or estimate the hydraulic properties of the stream channel. The total watercourse must be divided into channel reaches which are hydraulically similar within themselves. Often, hydraulically similar reaches will have similar slopes. Dramatic slope changes should be apparent from both topography and channel shape. Field reconnaissance measurements of the stream channel are suggested; however, sometimes direct measurements are not possible. The engineer must determine the slope, channel cross section, and an appropriate hydraulic roughness coefficient for each channel reach using the best information available within the limits of access, time, and budgets (topographic maps, aerial photography, etc.). Average slope is often determined from the topographic mapping. Channel cross sections should be measured in the field whenever possible, but scalable aerial photography may provide sufficient information to assess channel cross section characteristics.

Roughness coefficients of the waterway should be based on actual observations of the watercourse or of accessible nearby watercourses which are believed to be similar. If the reach is inaccessible, and if there is good quality aerial photography available it may provide adequate information for this purpose.

Time of Concentration (Tc) by Iterative Method within the Stream Hydraulic Method is simply the travel time (Tt) in the stream channel. Channel flow velocities can be estimated from normal depth calculations for the watercourse. In addition to the average flow velocity, engineers should compute the Froude number (Fr) of the flow. If the Fr number of the flow exceeds a value of 1.3, the engineer should verify that supercritical flow conditions can be sustained. For most earth lined channels, the velocity calculation should be recomputed using a larger effective Manning's roughness coefficient "n" until the Froude number has a value less than 1.3. Note that most upland arroyos flow very close to critical depth (Fr=1) and in most cases, normal depth and critical depth are very close to the same depth and velocity.

Velocity (V) is determined from Manning's Equation:

$$V = \frac{1.486}{n} R^{0.667} S^{0.5} \qquad \textbf{402-10}$$

where:

V	=	velocity, ft/s
n	=	Manning's roughness coefficient
R	=	hydraulic radius (area/wetted perimeter), ft
S	=	slope of the energy grade line (assumed to be the same as the channel slope) ft/ft

Froude number (Fr) is calculated by the following equation:

$$Fr = \frac{V}{(g\ d)^{0.5}} \qquad \textbf{402-11}$$

where:

Fr	=	Froude number
V	=	velocity, ft/s
g	=	gravitational acceleration, 32.2 ft/s^2
d	=	hydraulic depth (flow cross sectional area/top width of flow), ft

In order to solve Manning's Equation for velocity (V), calculate or estimate the hydraulic radius (R). If the flow depth or flow rate is known, then R may be found directly. However, the usual situation is that neither flow depth nor flow rate are known without first computing the Tc and an initial discharge. Three procedures are provided below for solving this problem.

Simplified Flow Estimating Procedure

Wide Shallow Channels

Use this method for channels where the flow depth is relatively shallow compared to the flow width. When this is true, the hydraulic radius (R) converges toward depth (d). The use of R=d is acceptable for NMDOT projects where the stream channel is relatively wide, and the flow is shallow. Larger arroyo systems in alluvial terrain often satisfy this criterion.

Moderate and Narrow Width Channels

Use this method for all other channels. Estimate the flow depth from high water mark evidence or other available data. For most ephemeral stream channels, the 25-year to 100-year storm flow depths may be in the range of 1 to 3 ft. Where a channel has obvious channel banks in the 1 to 3 ft height range, use the "bank full" depth. For most ephemeral streams use the bank full depth of the low flow channel. If the evidence suggests a flow depth greater than the height of an incised channel bank, use the physical evidence depth but compute the flow velocity based on water in the channel only (no overbank flow considered). Use the flow depth and channel cross section geometry to estimate R. For estimated flow depths deeper than 3 to 5 ft, the engineer should consider using the iterative procedure described below.

Iterative Procedure

For some channel flow conditions, the simplified procedures described above may not be adequate. In these cases, the iterative procedure described here must be followed. First, the peak rate of runoff from the watershed is estimated. A beginning estimate may be obtained using experience and judgment or by using the USGS regional regression equations for New Mexico (see **Section 407** of this Manual.) The flow rate for the velocity calculation is assumed to be two-thirds of the peak rate. Average channel velocity is calculated from **Equation 402-10** using the other hydraulic parameters of the channel. The average channel velocity for each reach is then used to determine the total Tc for the watershed. After the peak discharge from the watershed is computed, reassess the flow rate used to compute an average channel velocity. If the assumed peak discharge is within 10% of the calculated peak discharge, the computed average channel velocity and resulting Tc should be reasonably accurate. Often a second iteration is required using two-thirds of the computed peak flow to compute a new average channel velocity. This iterative procedure should be continued until the assumed peak discharge rate is within 10% of the computed peak discharge rate. Appendix 6 contains **Example Problem 6-5** that demonstrates this Method. Note: use of a computer program to calculate normal depth will greatly expedite this iterative procedure.

402.10 Channel and Floodplain Characteristics

Stream channels, floodplains, and reservoirs can have a significant impact on the delivery of water to any location along a stream network. Flood routing impacts the magnitude of the peak discharge, the time of the peak discharge, depth and extent of flooding, and environmental factors such as stream bank erosion, floodplain scour, sediment transport, and deposition.

The size, shape, and configuration of the channel and floodplain of a stream system are a reflection of the hydrologic processes within the watershed that created the stream system. A channel/floodplain system that is part of a high runoff producing watershed will look dramatically different than one that regularly produces little runoff. The process of both developing the hydrologic parameters needed to perform hydrologic analyses and the qualitative review of the results should include an assessment of the resulting channel/floodplain system.

The Time of Concentration (Tc) calculation is one of the most critical input parameters to any deterministic (as opposed to probabilistic) hydrologic analysis. Tc in a large watershed is determined largely on the hydraulics of the channel and floodplain system while in smaller watersheds, sheet flow and shallow concentrated flow may dominate.

Hydraulic parameters and qualities such as slope, cross section, bed form, Manning's roughness coefficient "n", rating curves, sediment size, sediment volumes, vegetation type and densities, are all related to the watershed's response to rainfall and the climate in which the watershed is located. Experience and judgment are required to assess the relative importance and impacts of each of these parameters. This experience is gained by always beginning with a qualitative assessment of the channel/floodplain system. Then developing hydrologic and hydraulic data, assumptions and calculations, and then checking the analysis results to verify that they are reasonable given the characteristics of the channel/floodplain system.

402.11 Sediment Bulking

Flood flows from high-intensity rainfall events on bare or mostly bare soils and flows within ephemeral sand bottom arroyos often contain significant amounts of sediment. When using one of the deterministic modeling approaches (but not Regional Regression Equations or streams with gage records) in this manual, it should be recognized that the resulting peak discharge and runoff volume are clean or clear water values, and therefore do not include the flow bulking that results from sediment.

Conveyance Structures

If the water conveyance structure (culvert, concrete box culvert, or bridge) has 120% or more of the required design capacity above the clear water discharge to meet NMDOT hydraulic criteria, then no further bulking factor analyses is required. However, if the conveyance structure does not meet the 120% criterion, see **Table 205-1**, then a more rigorous bulking factor analysis must be performed, or upsize the conveyance structure.

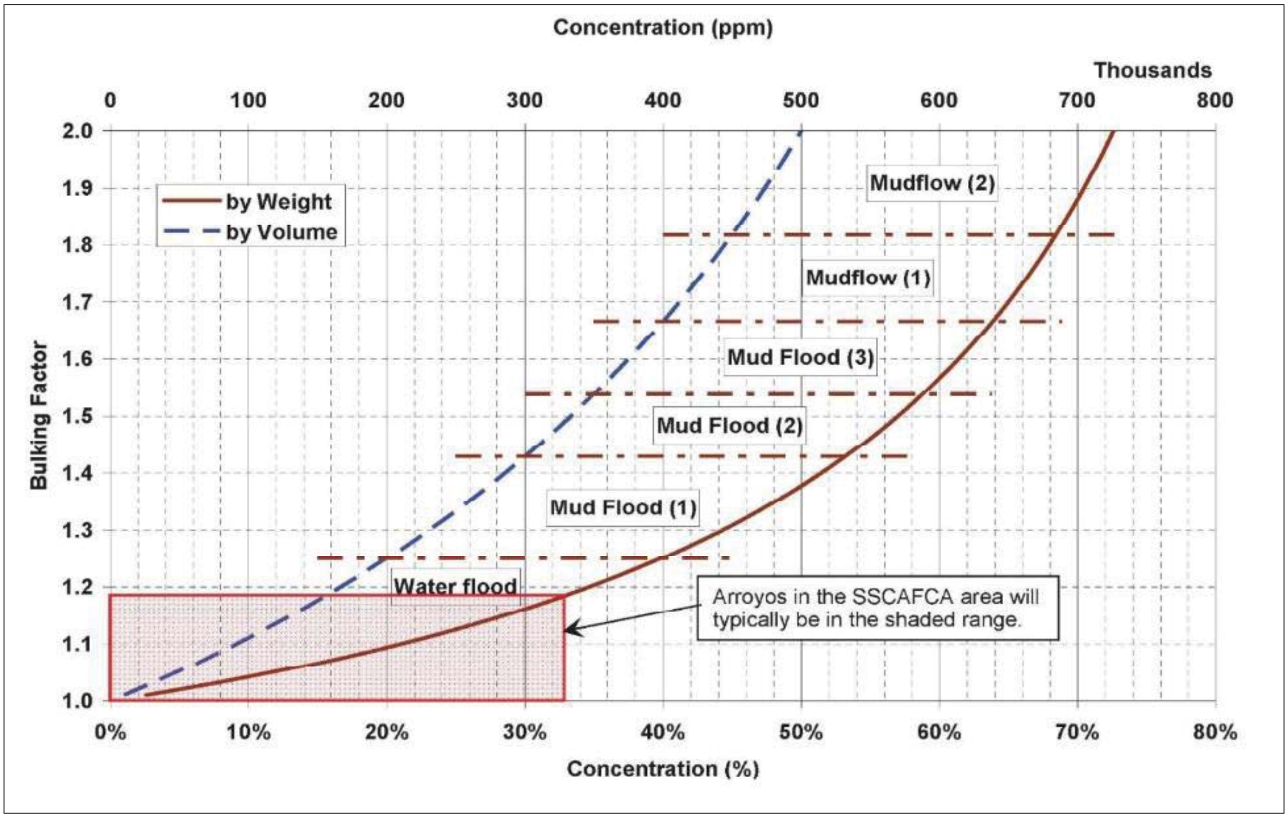
Detention and Retention Ponds

For the hydrologic analyses required for pond design, clear water storm runoff hydrographs must account for sediment by application of sediment bulking factors. The information presented in this Section combined with the pond design requirements presented in **Section 207** must be addressed during pond design.

402.11.1 SSCAFCA Sediment and Erosion Guide

The information in this Section was excerpted from a document titled "Sediment and Erosion Design Guide", November 2008, developed for the Southern Sandoval County Arroyo Flood Control Authority (SSCAFCA), prepared by Mussetter Engineering, Inc. http://sscafca.org/sediment-and-erosion-design-guide/

Figure 402-16 provides a guide to a range of possible sediment bulking factors in relation to sediment concentration for sand arroyos in the Sandoval County area. These figures and the supporting text of the Sediment and Erosion Guide will assist in estimating sediment bulking factors in arroyos outside the Sandoval County area (qualitatively at least).



Source: SSCAFCA, November 2008, "Sediment and Erosion Design Guide", Figure 3.8, p. 3.24. http://sscafca.org/development/documents/sediment_design_guide/Sediment%20Design%20Guide%2012-30-08.pdf

Figure 402-16 Relationship between Total Sediment Concentration and Bulking Factor

Bulking Factors for the SSCAFCA Area

Discharges estimated using standard rainfall-runoff procedures typically do not account for the presence of sediment in the flow. At high sediment loads, the total volume of the water/sediment mixture, and thus, the peak design discharges, can be substantially higher than the corresponding clear-water values. The following relation provides a means of computing a bulking factor (B_f) which is a factor applied to adjust (increase) the clear-water discharges for the presence of the transported sediment, if the sediment load is known:

$$B_f = \frac{Q + Q_{S_{total}}}{Q} = \frac{1}{1 - \dfrac{C_S / 10^6}{S_g - (C_S / 10^6)(S_g - 1)}} \qquad \textbf{402-12}$$

(SSCAFCA, November 2008, “Sediment and Erosion Guide”, Eq. 3.25, p. 3.23) http://sscafca.org/development/documents/sediment_design_guide/Sediment%20Design%20Guide%2012-30-08.pdf

where:

B_f	=	bulking factor
Q	=	clear-water discharge, cfs
$Q_{s\ total}$	=	total sediment load (i.e., combination of bed material and wash load), cfs
C_s	=	total sediment concentration by weight, ppm and
S_g	=	specific gravity of the sediment

This relationship indicates that the bulked discharge for a water/sediment mixture at the upper limit of concentrations for water floods (200,000 ppm by volume or 410,000 ppm by weight) would be about 25 percent greater than the clear water discharge (i.e., a bulking factor of 1.25) (**Figure 402-16**).

Because specific knowledge of the sediment load is often not available, conservative estimates of the bulking factor that can be applied to a range of potential design discharges were made by applying the MPM-Woo procedure for a typical rectangular cross section with width-depth ratio (F_D) at the dominant discharge (Q_D) of 40, assuming critical flow conditions and a range of median (D_{50}) particle sizes. Dominant discharge is defined in **Figure 402-17**, and a method for estimating its magnitude is provided in the text box that follows. Note that the figure enclosed within the text box is difficult to read as is the original document (SSCAFCA, 2008).

Chapter 3 of this guide provides guidance in relating bulking factors to median (D_{50}) bed material size for the following recurrence interval floods: 2-, 5-, 10-, 25-, 50- and 100-year, based on a range of dominant discharge values. D_{50} is defined as the sediment size for which 50% of the sample is finer by weight.

Annual Sediment Yield and Dominant Discharge

The **dominant** (or effective) discharge is defined as the increment of discharge that carries the most sediment over a long period of time (Wolman and Miller, 1960; Andrews, 1980; Biedenharn et al., 2000). In perennial, self-adjusted streams, the dominant discharge is often assumed to be same as the bankfull discharge because this represents the long-term condition to which the channel has adjusted, and it is also often assumed to be equivalent to about the mean annual flood peak. Care must be taken in making these assumptions, however, because the dominant, bankfull and mean annual flood peak discharges can be quite different, even in perennial, self-adjusted stream. For ephemeral streams, the dominant discharge tends to be associated with larger, less frequent flood peaks than in perennial streams, due to the absence of sustained flows and the flashy nature of the storm hydrographs. For design purposes, the dominant discharge for lightly developed watersheds in the SSCAFCA jurisdictional area will typically be in the range of the 5- to 10-year peak discharge. In more highly developed watersheds, the frequency of the dominant discharge is typically less because runoff (and sediment transport) associated with the more frequent storms tends to increase dramatically. As a result, the frequency of the dominant discharge is typically in the range of the 3- to 5-year flood peak.

A quantitative method for estimating Q_D in arroyos

If bed-material transport rating curves and storm hydrographs are available, the dominant discharge can be estimated as the peak of the storm event that will produce a bed-material sediment yield equal to the mean annual bed-material sediment yield. The mean annual sediment yield can be estimated by integrating the sediment yield frequency curve (Chang, 1988):

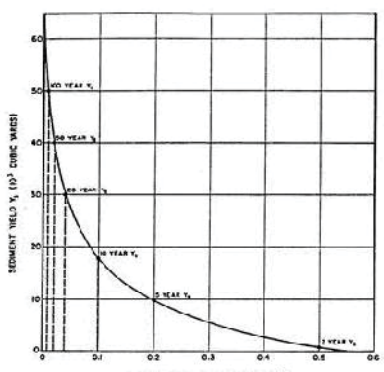
$$Y_{sm} = \int_0^1 Y_s dP_F \tag{3.26}$$

where Y_s is the individual storm sediment yield and P_F is the probability of occurrence of that flood in one year. The product $Y_s \cdot P_F$ represents the contribution of a particular flood to the long-term mean annual yield. For practical purposes, the integration can be accomplished for a series of discrete storm events using the trapezoidal rule. Using the 2-, 5-, 10-, 25-, 50-, and 100-year events, for example, the mean annual sediment yield is approximated by the following relationship:

$$Y_{sm} = 0.015\,Y_{s100} + 0.015\,Y_{s50} + 0.04\,Y_{s25} + 0.08\,Y_{s10} + 0.2\,Y_{s5} + 0.4\,Y_{s2} \tag{3.27}$$

If only the 2-, 10- and 100-year events are used, the following relationship is obtained:

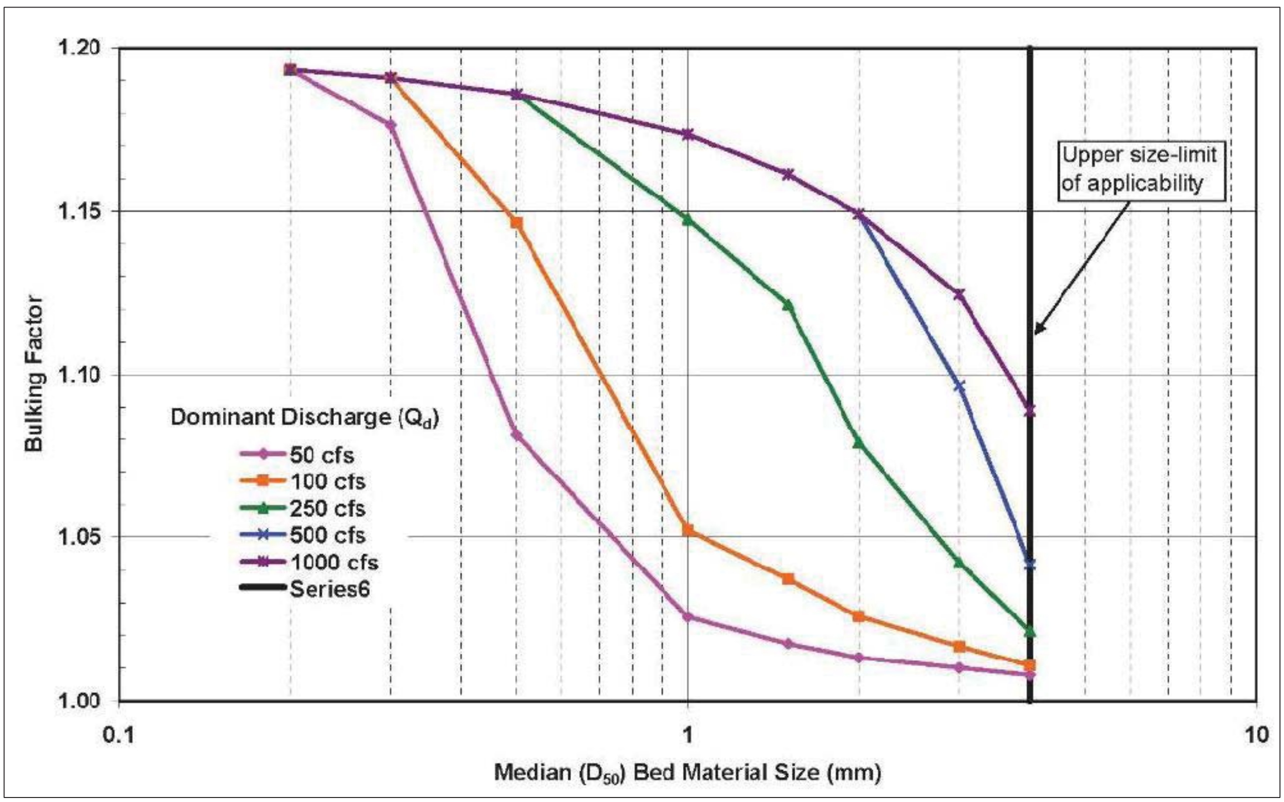
$$Y_{sm} = 0.055\,Y_{s100} + 0.245\,Y_{s10} + 0.45\,Y_{s2} \tag{3.28}$$



Source: SSCAFCA, November 2008, "Sediment and Erosion Design Guide", p. 3.28.
http://sscafca.org/development/documents/sediment_design_guide/Sediment%20Design%20Guide%2012-30-08.pdf

Figure 402-17 Annual Sediment Yield and Dominant Discharge

The assumed width-depth ratio (F_D) of 40 is based on data from a variety of existing, naturally adjusted arroyos (Leopold and Miller, 1956; Harvey et al., 1985). The assumption of critical flow is based on the observation that average Froude numbers (F_r) in stable sand-bed streams rarely exceed 0.7 to 1.0 (Richardson, personal communication) at high discharges. It should also be noted that current FEMA procedures for evaluating hydraulic conditions on alluvial fans is based on the assumption of critical flow (F_r = 1). Based on analysis of a wide range of arroyos in the greater Rio Rancho and Albuquerque area, the dominant discharge typically has a recurrence interval in the range of 5 to 10 years under relatively undeveloped conditions and decreases to 3 to 5 years under highly developed conditions due, primarily, to the increase in runoff during frequently occurring storms. The peak discharge associated with other recurrence interval flows was estimated using average ratios for conditions in the greater Rio Rancho and Albuquerque area. The 100-year peak discharge, for example, averages about five times the dominant discharge. Bulking factors estimated using the above assumptions for the 100-year peak are shown in **Figure 402-18** for channels with dominant discharge ranging from 50 to 1,000 cfs and median (D_{50}) bed-material sizes ranging from 0.5 to 4 mm. As shown in that figure, the bulking factors range from about 1.01 for small arroyos (W_d< = 50 cfs) with relatively coarse bed material (D_{50} = 4 mm) to a maximum of 1.19 for larger channels (Q_D> = 500 cfs) and relatively fine bed material (D_{50}<= 0.5 mm). Estimated bulking factors for other recurrence interval events for the median bed-material sizes are provided in **Figure 402-19**.



Source: SSCAFCA, November 2008, “Sediment and Erosion Design Guide”, Figure 3.9, p. 3.25. http://sscafca.org/development/documents/sediment_design_guide/Sediment%20Design%20Guide%2012-30-08.pdf

Figure 402-18 Bulking Factors for the 100-year Peak Discharge for Natural Channels

Table 3.6. Estimated sediment bulking factors for arroyos in the SSCAFCA jurisdictional area.					
Recurrence Interval (yrs)	Dominant Discharge (cfs)				
	50	100	250	500	1,000
D_{50} (mm) = 0.5 mm					
2	1.01	1.01	1.01	1.01	1.02
5	1.02	1.02	1.05	1.08	1.14
10	1.03	1.05	1.10	1.19	1.19
25	1.05	1.09	1.19	1.19	1.19
50	1.07	1.12	1.19	1.19	1.19
100	1.08	1.15	1.19	1.19	1.19
D_{50} (mm) = 1.0 mm					
2	1.01	1.01	1.01	1.01	1.01
5	1.01	1.01	1.01	1.03	1.05
10	1.01	1.01	1.03	1.07	1.16
25	1.02	1.03	1.08	1.17	1.17
50	1.02	1.04	1.12	1.17	1.17
100	1.03	1.05	1.15	1.17	1.17
D_{50} (mm) = 1.5 mm					
2	1.01	1.01	1.01	1.01	1.01
5	1.01	1.01	1.01	1.02	1.04
10	1.01	1.01	1.02	1.05	1.13
25	1.01	1.02	1.06	1.14	1.16
50	1.01	1.03	1.09	1.16	1.16
100	1.02	1.04	1.12	1.16	1.16
D_{50} (mm) = 2.0 mm					
2	1.01	1.01	1.01	1.01	1.01
5	1.01	1.01	1.01	1.01	1.03
10	1.01	1.01	1.02	1.04	1.08
25	1.01	1.01	1.04	1.09	1.15
50	1.01	1.02	1.06	1.15	1.15
100	1.01	1.03	1.08	1.15	1.15
D_{50} (mm) = 3.0 mm					
2	1.01	1.01	1.01	1.01	1.01
5	1.01	1.01	1.01	1.01	1.02
10	1.01	1.01	1.01	1.02	1.04
25	1.01	1.01	1.02	1.05	1.11
50	1.01	1.01	1.03	1.07	1.12
100	1.01	1.02	1.04	1.10	1.12
D_{50} (mm) = 4.0 mm					
2	1.01	1.01	1.01	1.01	1.01
5	1.01	1.01	1.01	1.01	1.01
10	1.01	1.01	1.01	1.02	1.03
25	1.01	1.01	1.02	1.03	1.06
50	1.01	1.01	1.02	1.04	1.10
100	1.01	1.01	1.03	1.06	1.10

Source: SSCAFCA, November 2008, "Sediment and Erosion Design Guide", Table 3.6, p. 3.26. http://sscafca.org/development/documents/sediment_design_guide/Sediment%20Design%20Guide%2012-30-08.pdf

Figure 402-19 Estimated Bulking Factors

402.11.2 New Mexico Institute of Mining and Technology

The NMDOT previously contracted with New Mexico Institute of Mining and Technology (NMIMT) to study the sediment bulking issue in New Mexico streams and arroyos. The resulting study report "Development of Watercourse Aggradation/Degradation Risk Index for New Mexico," May 2013, may be acquired from the NMDOT website at: http://dot.state.nm.us/content/dam/nmdot/Research/NM10DSN-01_Final_Report_Aggredation_Risk_with_Impl.pdf

The NMIMT report provides estimates for sediment bulking factors and risk maps for selected New Mexico Watersheds and for each of the NMDOT Maintenance Districts. **Figure 402-20** and **Figure 402-21** are examples of the maps found in this report. The NMIMT figures illustrate bulking factors up to 1.50 for some areas. Note that a sediment bulking factor greater than about 1.25 would be considered mud flow based on the reference presented in the previous Section.

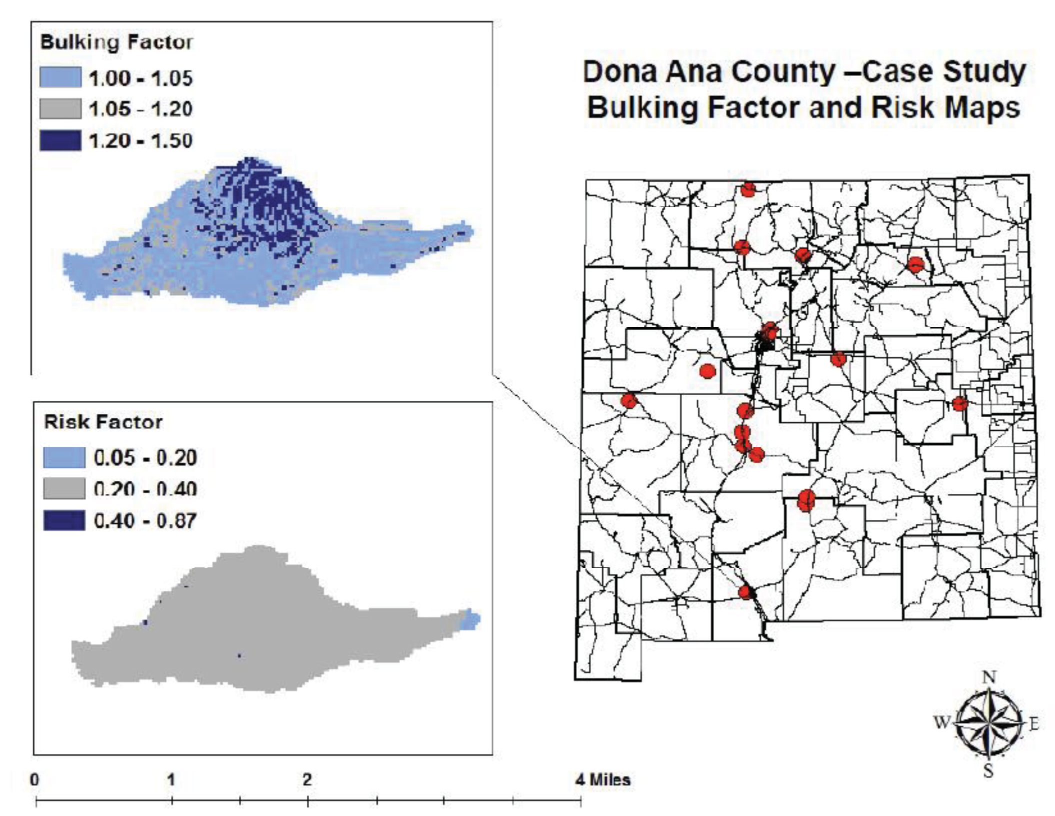


FIGURE 50: Bulking Factor and Risk Maps for Dona Ana County.

Source: New Mexico Institute of Mining and Technology, May 2013, Development of Watercourse Aggradation/Degradation Risk Index for New Mexico

Figure 402-20 Bulking and Risk Map Example

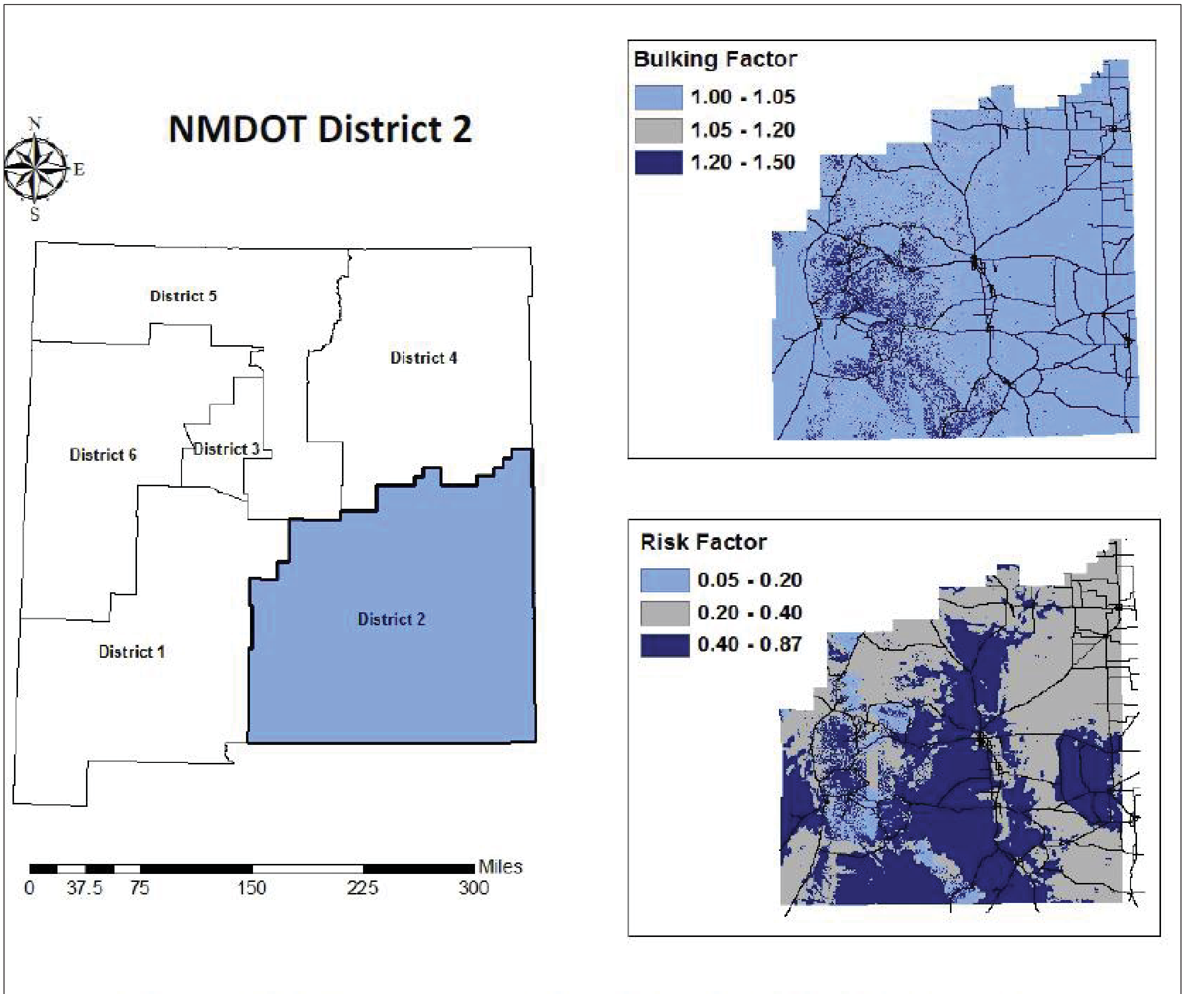


FIGURE 53: Bulking Factor and Risk Maps for NMDOT District 2.

Source: New Mexico Institute of Mining and Technology, May 2013,
Development of Watercourse Aggradation/Degradation Risk Index for New Mexico

Figure 402-21 District Bulking Factor and Risk Map Example

402.11.3 Guidance on Sediment Bulking Factor Selection

Sediment bulking factor selection is subjective and is driven by the basin land use type and condition, and also by the drainage conveyance system type and condition. General guidance, questions and items to consider that contribute or not, to bulking factor selection follow.

- Is the basin 100% urbanized without any exposed soil areas or landscape areas that will general sediment? If so, this would imply a bulking factor of 1.0 (no sediment load) from the basin surface. However, then the drainage conveyance system must also be evaluated.
- If the basin is 100% urbanized, does the drainage conveyance system consist of only storm drains and hard lined channels, or are there also unlined watercourses? A system that is totally lined would imply that no sediment bulking factor would be required (factor

of 1.0). However, if the urbanized basin contains unlined areas and unlined channels, a sediment bulking factor would be required.

- Mountain forest basins in good condition, with rock channels will generally contribute very minor sediment loads. However, if the land has been overgrazed, damaged by logging operations, damaged by recreational vehicular traffic and related activities, or burned by fire, the sediment yield to the watercourse must be considered and will obviously increase the sediment bulking factor compared to a healthy forest.
- Rangeland basins in good condition will contribute minor sediment loads, and rangelands generally outfall to natural unlined watercourses. The composition of the watercourse must be considered (clays, sands, gravels, cobbles, boulders). A bulking factor will be required for rangeland basins and the magnitude of the factor will depend on the basin and watercourse conditions. However, if the land has been overgrazed, damaged by logging operations, damaged by recreational vehicular traffic and related activities, or burned by fire, excess sediment yield to the watercourse must be considered and will obviously increase the sediment bulking factor compared to a healthy rangeland.

402.12 Rain on Snow

Snowmelt runoff is a major component of the hydrologic cycle in some parts of New Mexico and can be an important consideration for design flood analysis. Heavy rainfall on snow can result in runoff events that are significantly larger than would otherwise result from either the rainfall event or snowmelt event alone. Consult the Drainage Design Bureau when the drainage analysis is in a watershed with the potential for significant snow accumulations. The NRCS provides good guidance in “Part 630 Hydrology, National Engineering Handbook”, Chapter 11 Snowmelt” and in “Chapter 18, Selected Statistical Methods”.

402.13 Fire Related Impacts

Increased risk of severe wildfires has become increasingly frequent in New Mexico and the Western U.S. and are currently an area of intense study by a variety of Federal and State agencies. Much literature has been produced in recent years due to the number, size, and severity of wildfires in the west in general and in and around New Mexico specifically. While at this time no dependable analysis tools are available for estimating the runoff from a severely burned watershed, it is clear that severe wildfires in a watershed can result in flood flows that are orders of magnitude higher than would have been expected prior to the fire. While it may be unfeasible to design a highway crossing for a flood that is 10 to 100 times larger than would have resulted from the standard design storm, consideration should be given with respect to the potential flood risk after a severe wildfire. NRCS and the U.S. Forest Service are expected to produce planning, analysis, and design documents in the near future addressing this issue. The hope is that these tools will assist in planning for and defending against large post-fire flood events. Consult with the Drainage Design Bureau for guidance when simulating burned watersheds.

In the interim, Ventura County in California has conducted studies, and developed guidance for estimating the impacts of flood flows after a severe wildfire. The study is titled “Sediment/Debris Bulking Factors and Post-Fire Hydrology for Ventura County, Final Report – June 2011”. (A hotlink is not available.)

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403 Rational Formula Method

Hydrologic analyses performed on small (<160 acre) watersheds will normally be performed using the Rational Formula Method. The Rational Formula Method is a widely and long accepted procedure worldwide for estimating peak rates of runoff from small watersheds. The Rational Formula Method may be used on NMDOT projects for roadway drainage facilities and small drainage structures as described in **Section 401** (**Figure 401-1** and **Figure 401-2**) of this manual. The standard form of the Rational Formula Equation in English units is:

$$Q = C\,i\,A \qquad \textbf{403-1}$$

where:

Q	=	the peak rate of runoff, cfs
C	=	Runoff Coefficient
i	=	the rainfall intensity, in./hr
A	=	the watershed or drainage area, acres

The units in the Rational Formula do not yield peak discharge in cubic feet per second (cfs) directly, but rather are in acre-inches/hour. However, the conversion from acre-inches/hour to cfs is 1.008 which is commonly neglected because it does not introduce a significant error. The Rational Formula has several assumptions implicit to the method, including:

- The rainfall intensity is uniform for a duration equal to or greater than Tc
- Peak flow occurs when the entire watershed is contributing runoff
- The frequency of the resulting peak discharge is equal to the frequency of the rainfall event.
- Both the Runoff Coefficient (C) and the rainfall intensity (i) vary with the return period (both tend to increase as return period increases). Therefore, both must be determined separately for each design storm frequency.
- The Runoff Coefficient (C) is dependent on the Hydrologic Soil Group (HSG) and the vegetative cover or in the case of developed watersheds, the percentage of impervious cover. HSGs are divided into four soil groups and are described in **Section 402.4.**

Limitations for using the Rational Formula Method on NMDOT projects include the following:

- The total drainage area should not exceed 160 acres
- Land use, slope, and soils are fairly consistent throughout the watershed
- There are no diversions, detention basins, pump stations, or other structures in the watershed which would require the routing of a flood hydrograph
- The Time of Concentration (Tc) does not exceed one hour
- Runoff volumes may not be computed with the Rational Formula Method or Modified Rational Formula Method (not included in this Drainage Design Manual)

403.1 Time of Concentration (Tc) for Use in the Rational Formula Method

The assumptions within the Rational Formula Method are that the rainfall intensity is uniform for a duration equal to or greater than Tc and that the entire watershed is contributing runoff when the peak occurs. Therefore, in order to determine the appropriate rainfall intensity “i” for the watershed, the Tc must be determined. For NMDOT projects, Tc shall be calculated using the Kirpich Equation or Upland Method depending on specific circumstances.

The Upland Method was originally developed by the Soil Conservation Service (SCS), which is now the Natural Resources Conservation Service (NRCS). The Upland Method is described in Chapter 15 Time of Concentration of “Part 630 Hydrology, National Engineering Handbook” (NRCS, 2010). Note that in the current (2010) version of Chapter 15, the NRCS has renamed the “Upland Method” to the “Velocity Method.” However, many documents still refer to it as the “Upland Method” and, therefore, the name “Upland Method” is used in this Drainage Design Manual.

The Upland Method is used to estimate travel times for overland flow and shallow concentrated flow conditions. The Upland Method is limited to use in watersheds less than 2000 acres in size, or to the upper reaches of larger watersheds. For NMDOT projects, the Upland Method may be used for computing the Tc when using the Rational Formula Method or the Simplified Peak Discharge Method on an ***un-gullied*** watershed. The use of Upland Method is described in **Section 402.9.1.**

When using the Rational Formula, the Kirpich Equation should be used in watersheds ***when gullying is evident in more than 10% of the primary watercourse***. Gullying can be assumed if a blue line appears on the watercourse shown on the USGS quadrangle topographic map or is apparent from field reconnaissance or from inspection of aerial photography. The Kirpich Equation is given as:

$$Tc = 0.0078\, L^{0.77}\, S^{-0.385} \qquad \textbf{403-2}$$

(TxDOT, July 2016, “Hydraulic Design Manual,” Eq. 4-15, p. 4-39)
http://onlinemanuals.txdot.gov/txdotmanuals/hyd/index.htm

where:

Tc	=	Time of Concentration, minutes
L	=	maximum length of water travel, ft
S	=	surface slope, given by H/L, ft/ft
H	=	difference in elevation between the most hydraulically remote point in the drainage basin and the outlet, ft

In small watersheds where the slope is very flat, and the flow path of the hydraulically longest flow path is dominated by overland flow (> 300 ft), the Kerby Equation should be considered for the overland flow portion and Kirpich Equation for the channelized portion.

For small watersheds where overland flow is an important component of overall travel time, the Kerby Equation can be used. The Kerby Equation is:

$$T_{OV} = K\,(L\,N)^{0.467}\, S^{-0.235} \qquad \textbf{403-3}$$

(TxDOT, July 2016, “Hydraulic Design Manual”, Eq. 4-14, p. 4-37)
http://onlinemanuals.txdot.gov/txdotmanuals/hyd/index.htm

where:

T_{ov}	=	overland flow Time of Concentration, minutes
K	=	K = 0.828, a unit conversion factor
L	=	the overland-flow length, ft
N	=	a dimensionless retardance coefficient
S	=	the dimensionless slope of terrain conveying the overland flow

In the development of the Kerby Equation, the length of overland flow was as much as 1,200 feet. Hence, this length is considered an upper limit, and in practice, shorter values generally are expected. The dimensionless retardance coefficient used is similar in concept to the well-known Manning's roughness coefficient; however, for a given type of surface, the retardance coefficient for overland flow will be considerably larger than for open-channel flow. Typical values for the retardance coefficient are listed in **Table 402-9**. Roussel et al. (2005), recommends that the user should not interpolate the retardance coefficients shown in **Table 402-9.** If it is determined that a low slope condition or a transitional slope condition exists, the user should consider using an adjusted slope in calculating the Tc.

Time of Concentration with the Kerby-Kirpich Method

When the Kirpich Equation result and the Kerby Equation result are combined, it is referred to as the Kerby-Kirpich Method. The watershed should be divided between the channelized reach and the overland flow reach and the travel time across each reach calculated and combined to compute the total Tc.

- If the calculations (with either Kirpich Equation or with the Kerby Equation) yield a Tc less than 10 minutes, use 10 minutes
- If the resulting Tc is greater than 1 hour, do not use the Rational Formula Method, select another hydrologic analysis method

403.2 Rainfall

When developing Intensity-Duration-Frequency (IDF) curves and Depth-Duration (DD) values for Rational Formula Method from NOAA Precipitation Frequency Data Server (PFDS), the following approach is provided to develop the IDF curves, from which the rainfall intensity "i" is derived for the design frequency storm required.

1. Go to NOAA Precipitation Frequency Data Server (PFDS) http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=nm
    a. Click on New Mexico on the Map
    b. Data Description – use defaults
    c. Get Location Options
        i. Use navigation tools to either:
            1. Enter latitude and longitude or
            2. Select Station or
            3. Selection Location on map
    d. Data Description
        i. Data Type: Select "precipitation intensity"

   ii. Units: Select "*English*"
   iii. Time series type: Select "***partial duration***"
e. Scroll down to Depth-Duration-Frequency table below map
f. Scroll to bottom of table and in the "Estimates from the table in csv format" box select "***precipitation frequency estimates***"
g. Open in MS Excel and do a "save as" to your workspace as a .txt file
h. Open .txt file (it should open in Excel)
i. Insert Chart into the Excel spreadsheet (see **Table 403-1** example spreadsheet below)
   i. Insert a column adjacent to the durations and fill in with time values (*Excel doesn't recognize "5-min" as a value*)
   ii. Select X Y Scatter Chart Type
   iii. Select Data with duration (in minutes) on the x axis, intensity (in./hr) on the y axis for each frequency (1-year, 2-year, 5-year, 10-year, 25-year, 50-year, 100-year) as needed for project analyses. (See **Table 403-1**)
j. Format x axis to allow reading duration in 1 minute increments and y axis to read intensity in 0.1 in./hour increments. (See **Figure 403-1**)
k. Read rainfall intensity that matches basin Tc for the storm frequency required.
l. **Minimum Tc = 10 minutes for this purpose!**

Table 403-1 NOAA Data Server Sample IDF Spreadsheet-Lemitar NM

Point precipitation frequency estimates (inches/hour)
NOAA Atlas 14 Volume 1 Version 5
Data type: Precipitation intensity
Time series type: Partial duration
Project area: Southwest
Location name: Lemitar, New Mexico, US*
Station Name: -
Latitude: 34.1580°
Longitude: -106.9181°
Elevation: 4712 ft*
* source: Google Maps

PRECIPITATION FREQUENCY ESTIMATES
by
duration

for ARI:		1	2	5	10	25	50	100	200	500	1000 years
5-min:	5	2.45	3.18	4.26	5.09	6.23	7.1	8.04	9	10.31	11.35
10-min:	10	1.87	2.42	3.24	3.88	4.74	5.41	6.11	6.85	7.84	8.64
15-min:	15	1.54	2	2.68	3.2	3.92	4.46	5.05	5.66	6.48	7.14
30-min:	30	1.04	1.34	1.8	2.16	2.64	3.01	3.4	3.81	4.36	4.81
60-min:	60	0.64	0.83	1.11	1.33	1.63	1.86	2.1	2.36	2.7	2.98
2-hr:		0.37	0.48	0.64	0.76	0.95	1.11	1.29	1.49	1.8	2.06
3-hr:		0.27	0.34	0.45	0.54	0.67	0.78	0.9	1.04	1.25	1.43
6-hr:		0.16	0.2	0.25	0.3	0.36	0.42	0.48	0.55	0.66	0.75
12-hr:		0.08	0.11	0.13	0.16	0.19	0.22	0.25	0.29	0.34	0.38
24-hr:		0.05	0.06	0.08	0.09	0.11	0.12	0.14	0.16	0.18	0.2
2-day:		0.03	0.03	0.04	0.05	0.06	0.07	0.07	0.08	0.1	0.11
3-day:		0.02	0.02	0.03	0.03	0.04	0.05	0.05	0.06	0.07	0.08
4-day:		0.02	0.02	0.02	0.03	0.03	0.04	0.04	0.05	0.05	0.06
7-day:		0.01	0.01	0.02	0.02	0.02	0.02	0.03	0.03	0.03	0.04
10-day:		0.01	0.01	0.01	0.01	0.02	0.02	0.02	0.02	0.03	0.03
20-day:		0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.02
30-day:		0	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01
45-day:		0	0	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01
60-day:		0	0	0	0	0.01	0.01	0.01	0.01	0.01	0.01

Date/time (GMT): Fri Nov 13 22:14:03 2015
pyRunTime: 0.127875804901

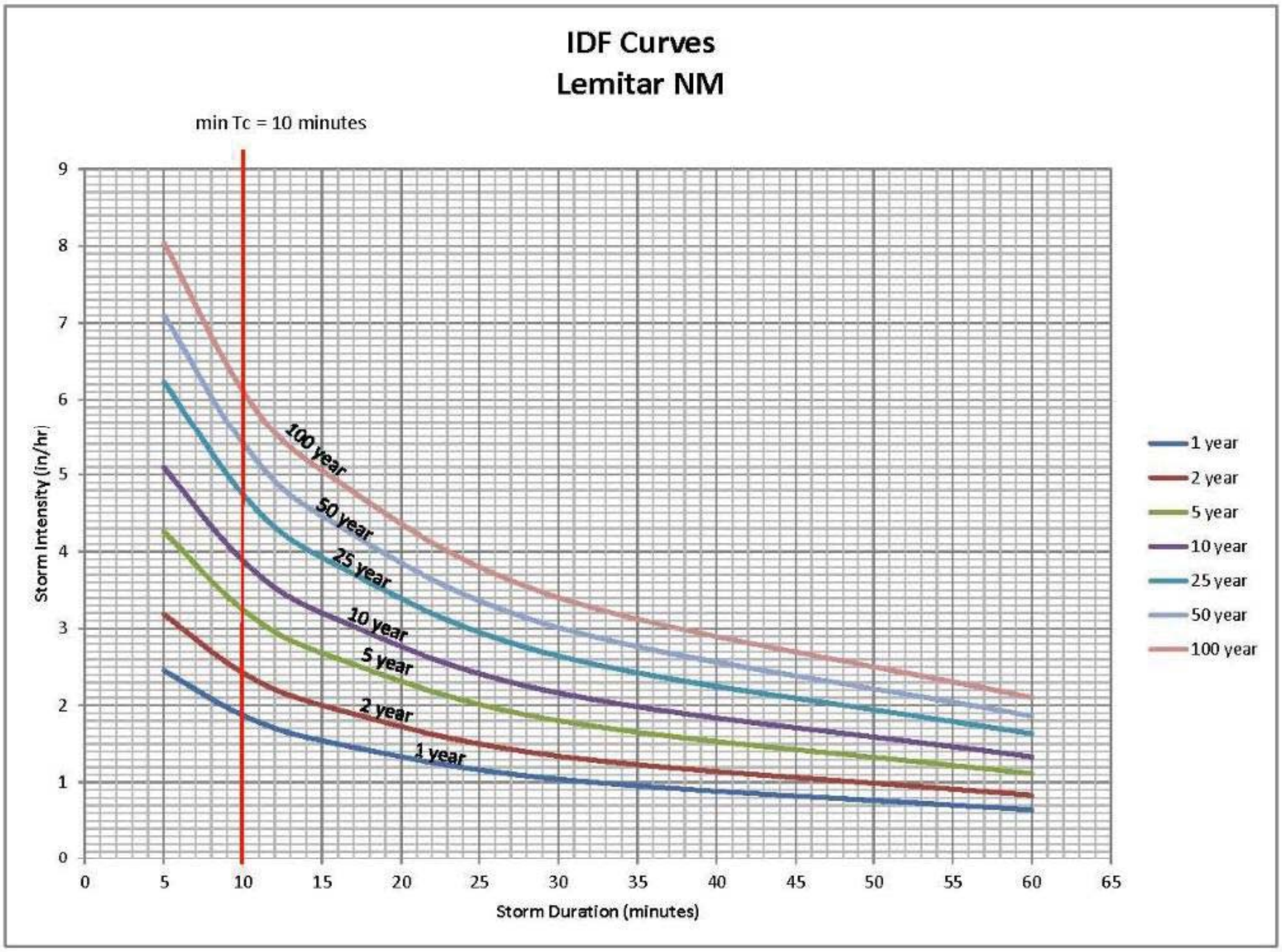


Figure 403-1 IDF Curves from NOAA Data Server-Lemitar, NM

To produce the Depth-Duration 1-hour precipitation values for use in determining the Rational Formula Runoff Coefficient "C", return to the NOAA Data Server for the same location as for the IDF Curve development (see **Table 403-2** from NOAA Data Server) http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=nm

Table 403-2 Depth-Duration-Frequency Table from NOAA Data Server

http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=nm

Point precipitation frequency estimates (inches)
NOAA Atlas 14 Volume 1 Version 5
Data type: Precipitation depth
Time series type: Partial duration
Project area: Southwest
Location name: Lemitar, New Mexico, US*
Station Name: -
Latitude: 34.1584°
Longitude: -106.9189°
Elevation: 4713 ft*
* source: Google Maps

PRECIPITATION FREQUENCY ESTIMATES

by duratior	1	2	5	10	25	50	100	200	500	1000 years
5-min:	0.2	0.27	0.35	0.42	0.52	0.59	0.67	0.75	0.86	0.95
10-min:	0.31	0.4	0.54	0.65	0.79	0.9	1.02	1.14	1.31	1.44
15-min:	0.39	0.5	0.67	0.8	0.98	1.12	1.26	1.41	1.62	1.78
30-min:	0.52	0.67	0.9	1.08	1.32	1.5	1.7	1.9	2.18	2.4
60-min:	0.64	0.83	1.11	1.33	1.63	1.86	2.1	2.36	2.7	2.98
2-hr:	0.75	0.96	1.27	1.52	1.9	2.22	2.58	2.98	3.59	4.13
3-hr:	0.81	1.03	1.35	1.61	2	2.33	2.7	3.12	3.75	4.3
6-hr:	0.93	1.18	1.51	1.78	2.18	2.52	2.9	3.31	3.93	4.48
12-hr:	1.01	1.28	1.63	1.91	2.31	2.65	3.03	3.44	4.06	4.59
24-hr:	1.16	1.45	1.82	2.12	2.55	2.9	3.29	3.72	4.35	4.88
2-day:	1.27	1.59	1.98	2.3	2.76	3.13	3.54	3.99	4.64	5.2
3-day:	1.36	1.7	2.12	2.46	2.94	3.34	3.78	4.25	4.95	5.55
4-day:	1.45	1.81	2.25	2.61	3.12	3.55	4.01	4.51	5.25	5.89
7-day:	1.67	2.08	2.57	2.96	3.52	3.97	4.46	4.99	5.77	6.41
10-day:	1.84	2.3	2.84	3.29	3.91	4.41	4.96	5.56	6.43	7.17
20-day:	2.33	2.9	3.54	4.03	4.71	5.25	5.81	6.39	7.2	7.89
30-day:	2.81	3.5	4.23	4.78	5.53	6.11	6.7	7.3	8.12	8.81
45-day:	3.41	4.23	5.08	5.7	6.61	7.12	7.71	8.29	9.11	9.78
60-day:	3.9	4.84	5.8	6.52	7.44	8.13	8.81	9.47	10.33	10.98

Date/time (GMT): Mon Nov 16 19:12:46 2015
pyRunTime: 0.126244068146

Procedure:

1. Data Description
   a. Data Type: Select "***precipitation depth***"
   b. Units: Select "***english***"
   c. Time series type: Select "***partial duration***"
2. Scroll down to Depth-Duration-Frequency table below map
3. Scroll to bottom of table and in the "Estimates from the table in csv format" box select "***precipitation frequency estimates***"
4. Open in MS Excel and do a "save as" to your workspace as a .txt file
5. Open .txt file (it should open in Excel) **Table 403-2**
6. Read point rainfall value for 1-hour design storm

403.3 Rational Formula Runoff Coefficient "C"

The Rational Formula Runoff Coefficient, "C" should be selected from **Figure 403-2** to **Figure 403-7** depending on the ground cover, Hydrologic Soil Group, type of development, and 1-hour rainfall depth for the design return period. The Runoff Coefficient "C" figures are adopted from the Arizona DOT Drainage Design Manual due to the similarities in climate, soils, vegetation and terrain between Arizona and New Mexico.

http://www.azdot.gov/docs/default-source/roadway-engineering-library/2014_adot_hydrology_manual.pdf?sfvrsn=16

Hydrologic Soil Groups are defined in **Section 402.4**. **Figure 403-2** to **Figure 403-7** show how "C" varies with 1-hour rainfall depth. This is because "C" is a function of infiltration and other hydrologic abstractions, relating the peak discharge to the theoretical peak discharge produced by 100% runoff.

Engineers are encouraged to review the supporting information provided in the Arizona manual before using these figures in order to familiarize themselves with their limitations and assumptions. When land use or other factors vary significantly throughout the watershed, an area weighted "C" value should be used. The weighted "C'" value is computed by the equation:

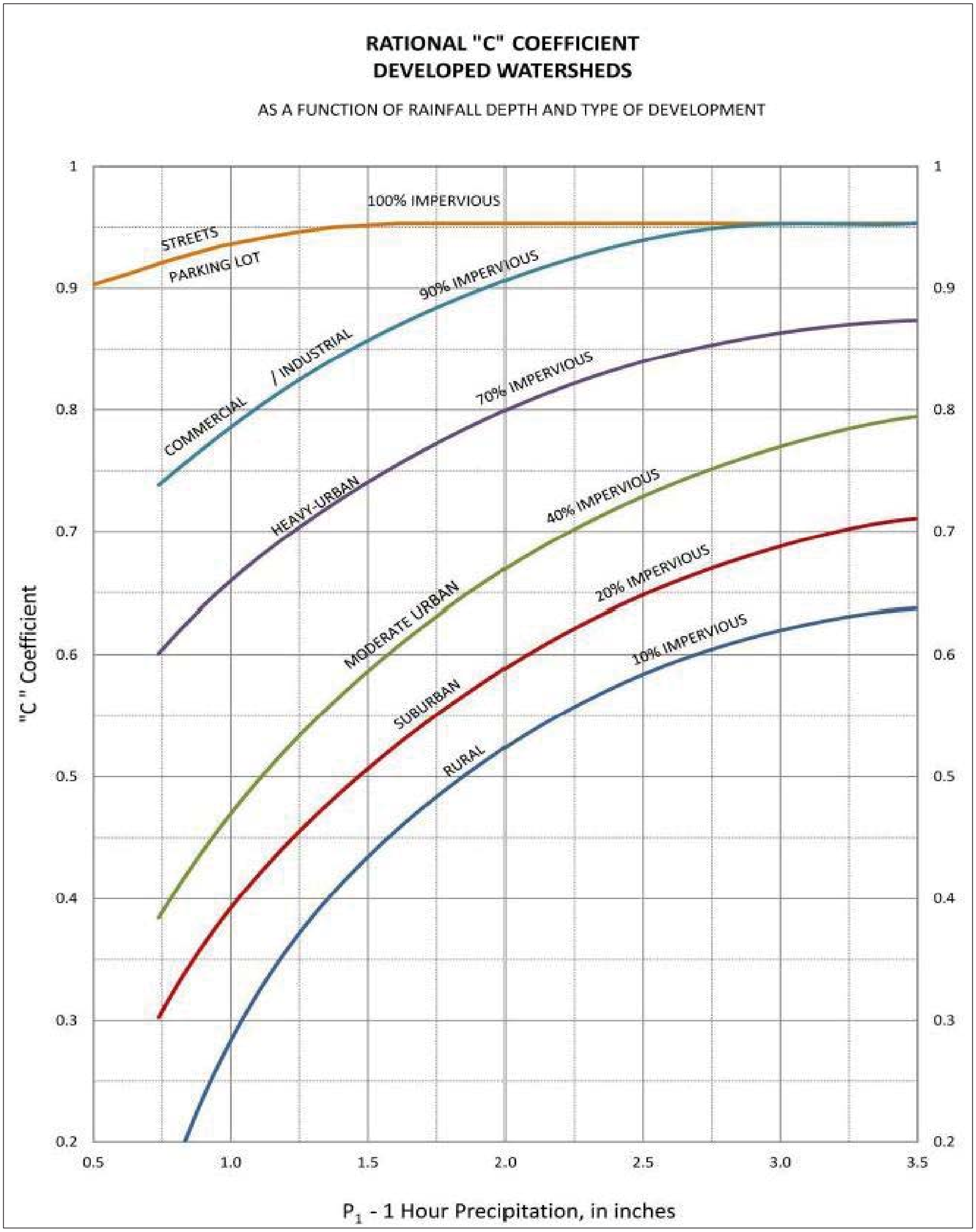
$$\text{Weighted C} = \frac{\text{C1 A1} + \text{C2 A2} + \text{C3 A3}\ldots}{\sum \text{A}} \qquad \textbf{403-4}$$

(Arizona Department of Transportation, 2014, "Highway Drainage Design Manual, Volume 2, Hydrology, Second Edition", Eq. 2.5, p. 2-7)
http://www.azdot.gov/docs/default-source/roadway-engineering-library/2014_adot_hydrology_manual.pdf?sfvrsn=16

where:

C1	=	"C" Runoff Coefficient for subbasin(s) 1, etc.
A1	=	area of subbasin(s) 1, etc., acres
$\sum$A	=	total basin area, acres

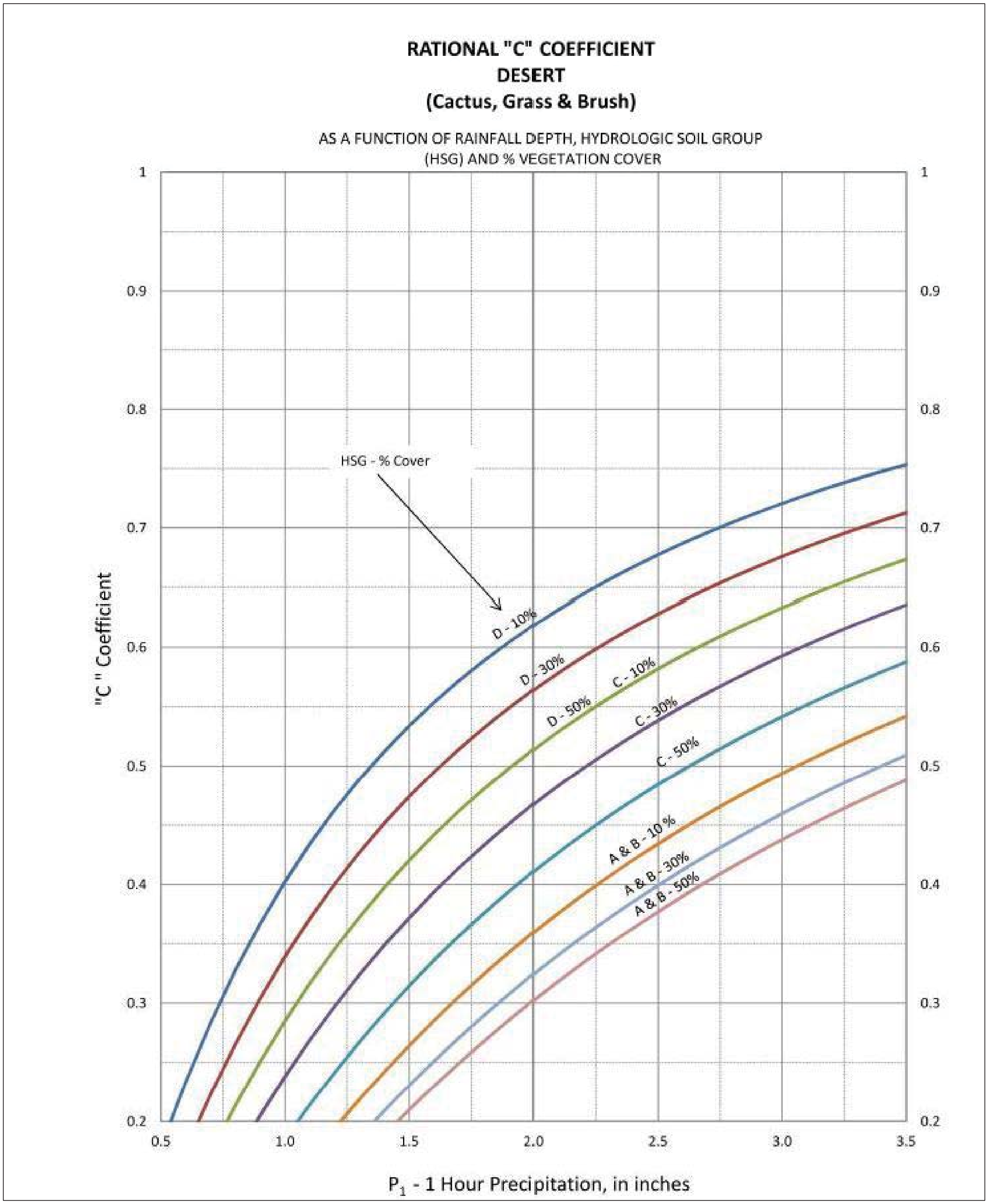
The designer should select the appropriate **Figure 403-2** to **Figure 403-7,** depending on the watershed location (desert, upland range, mountain or urban) and the predominant vegetation type (cactus, brush, grasses, juniper, pine). Enter the appropriate Figure with the design 1-hour rainfall depth. Move vertically up through the Figure until the appropriate curve is found, then move horizontally to find the design "C" value. The appropriate curve is selected based on the Hydrologic Soil Group (HSG) and the percent ground cover of the vegetation or percent imperviousness. When a value falls between two curves, interpolate linearly between the two nearest curves to the required percentage of cover or imperviousness.



Source: Arizona Department of Transportation, 2014, "Highway Drainage Design Manual, Volume 2, Hydrology, Second Edition", Figure 2-1, p. 2-8.

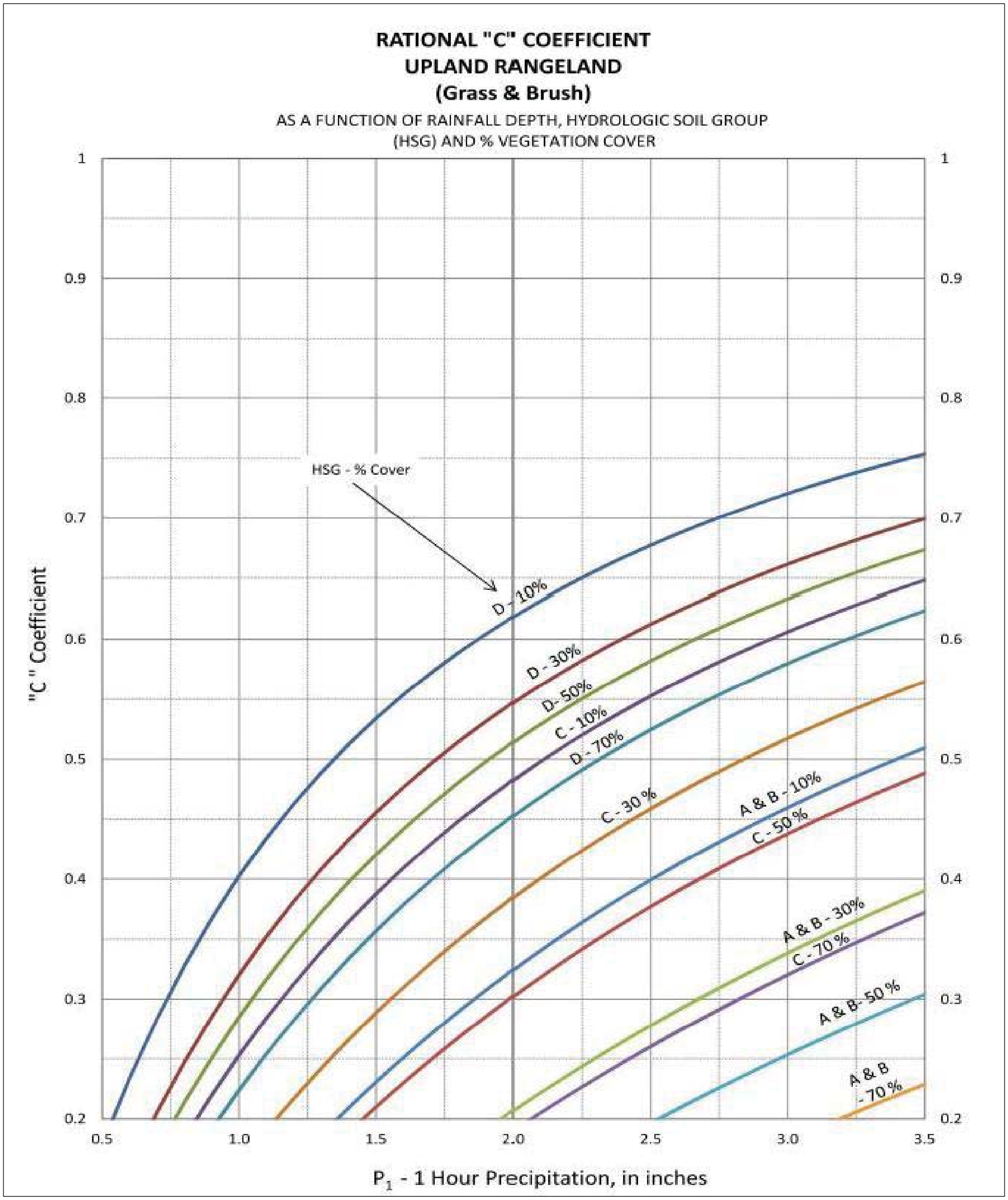
http://www.azdot.gov/docs/default-source/roadway-engineering-library/2014_adot_hydrology_manual.pdf?sfvrsn=16

Figure 403-2 Rational "C" Coefficient Developed Watersheds



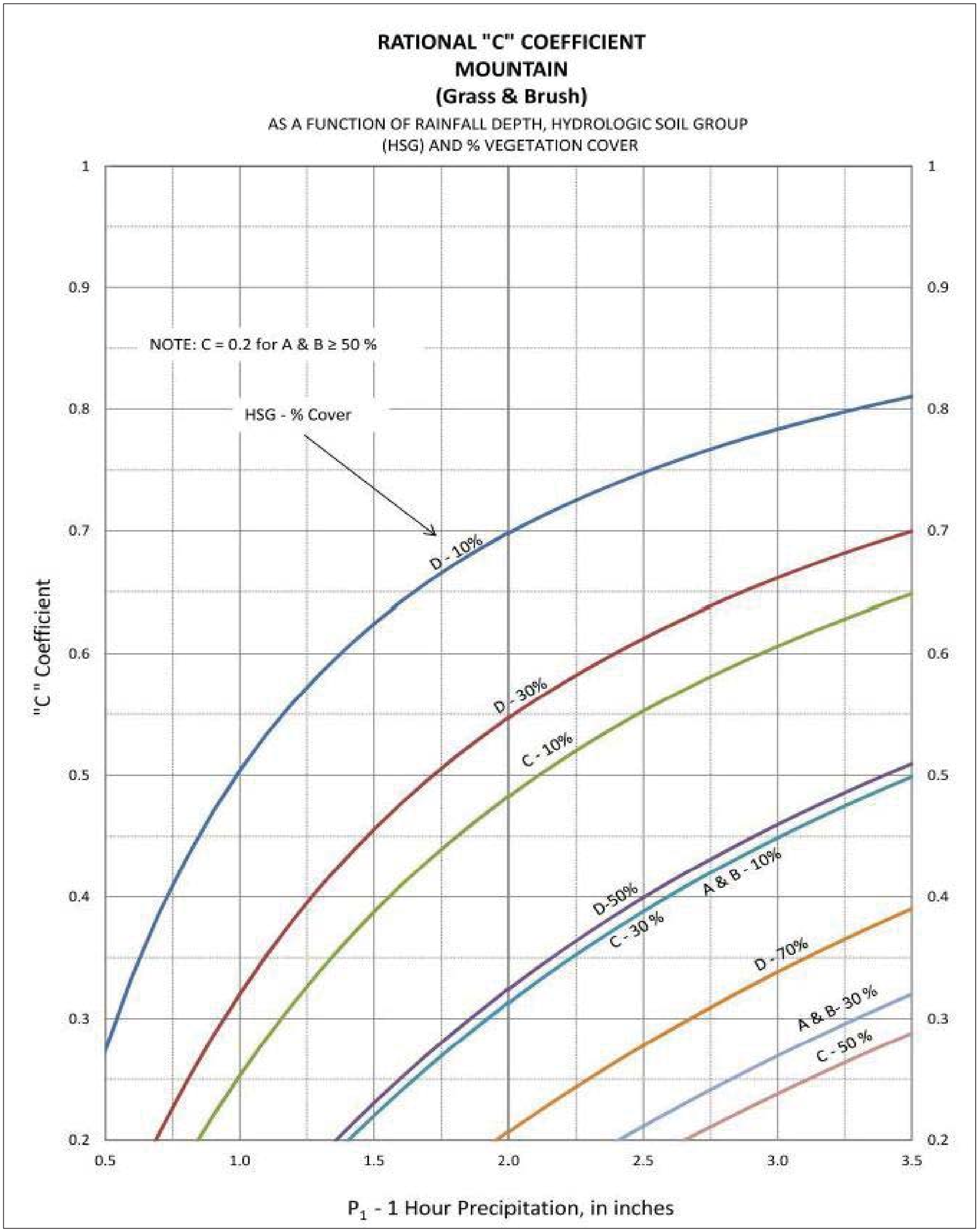
Source: Arizona Department of Transportation, 2014, “Highway Drainage Design Manual, Volume 2, Hydrology, Second Edition”, Figure 2-2, p. 2-9.
http://www.azdot.gov/docs/default-source/roadway-engineering-library/2014_adot_hydrology_manual.pdf?sfvrsn=16

Figure 403-3 Rational “C” Coefficient Desert (Cactus, Grass & Brush)



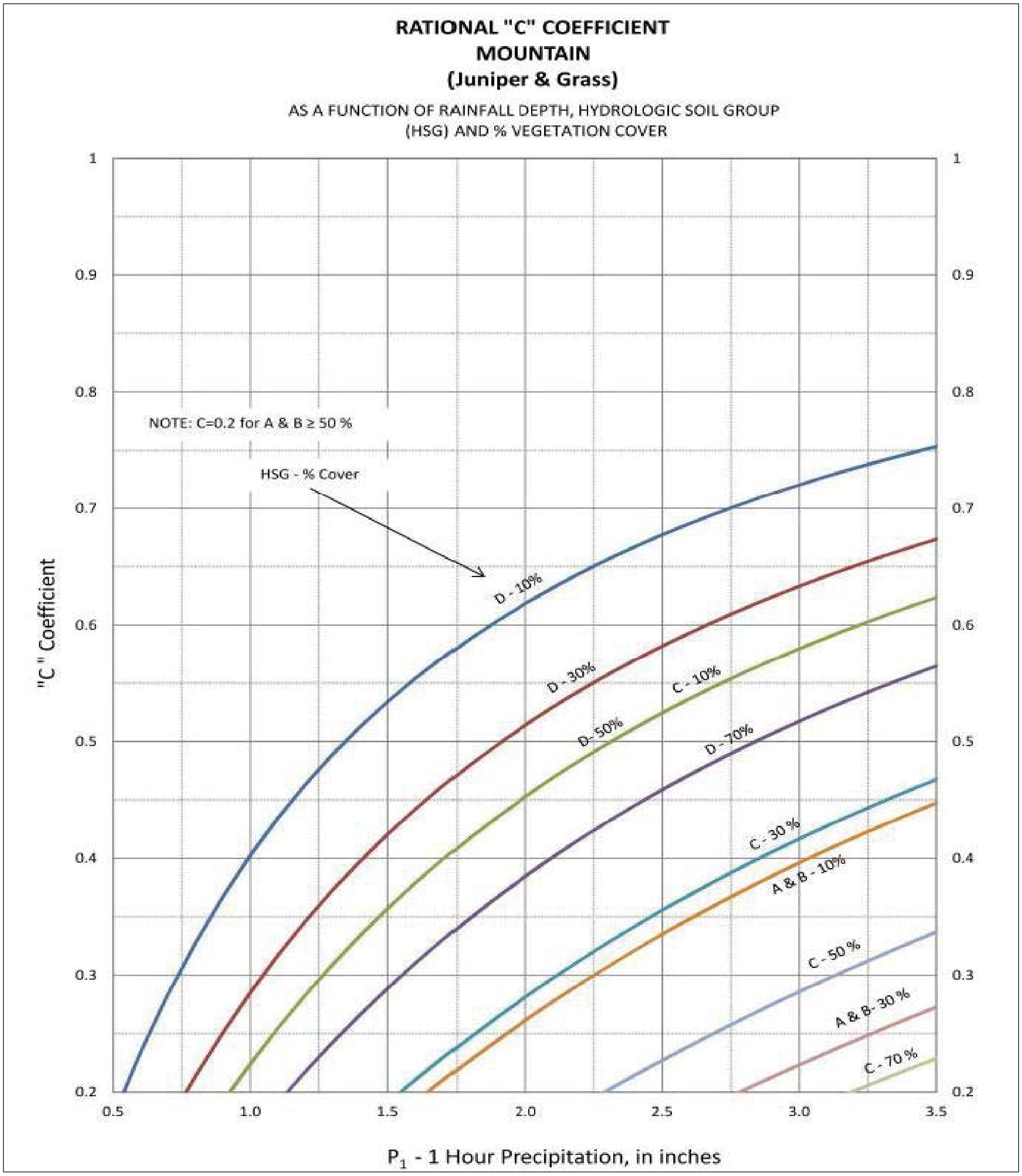
Source: Arizona Department of Transportation, 2014, “Highway Drainage Design Manual, Volume 2, Hydrology, Second Edition”, Figure 2-3, p. 2-10.
http://www.azdot.gov/docs/default-source/roadway-engineering-library/2014_adot_hydrology_manual.pdf?sfvrsn=16

Figure 403-4 Rational “C” Coefficient Upland Rangeland (Grass & Brush)



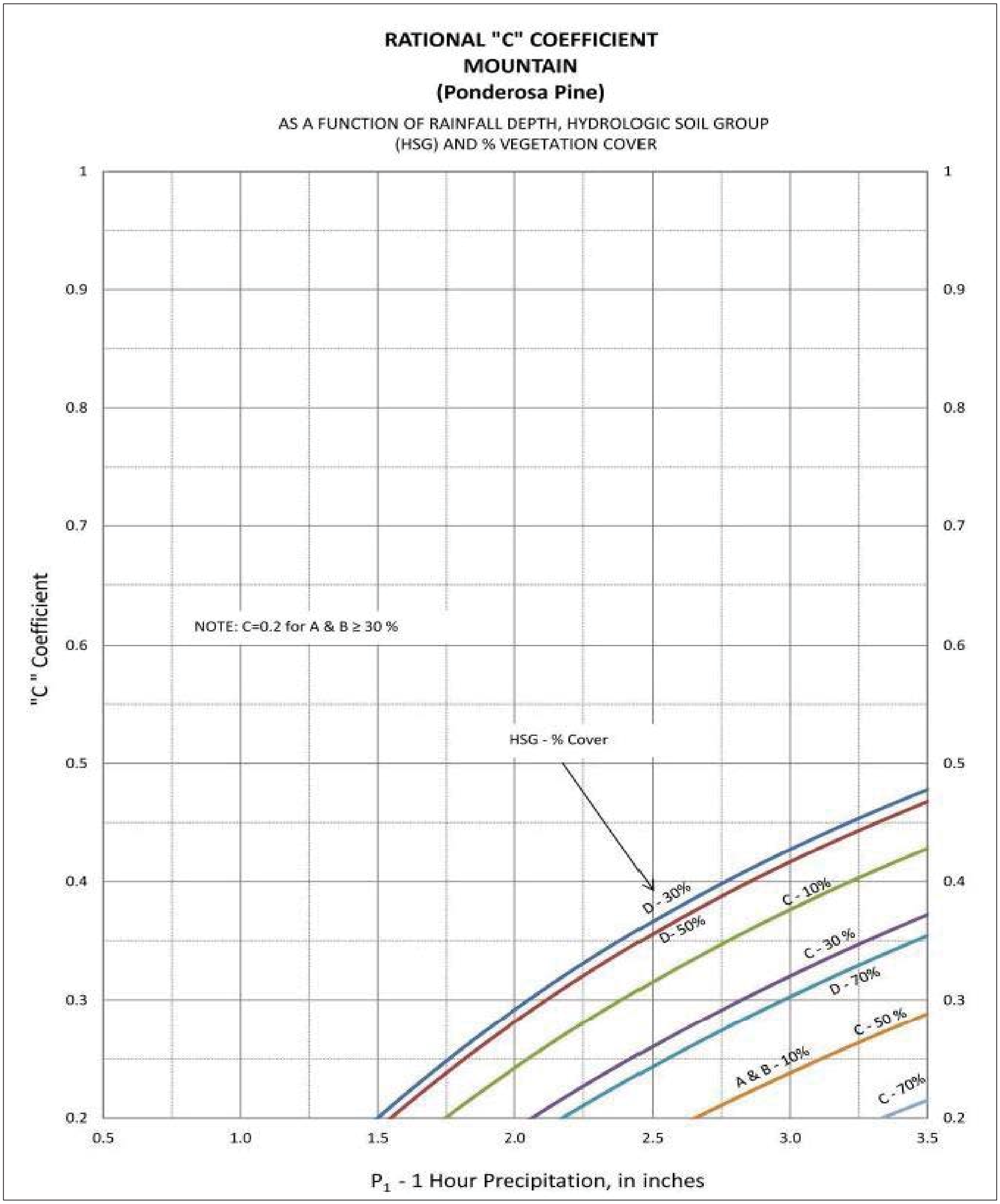
Source: Arizona Department of Transportation, 2014, “Highway Drainage Design Manual, Volume 2, Hydrology, Second Edition”, Figure 2-4, p. 2-11. http://www.azdot.gov/docs/default-source/roadway-engineering-library/2014_adot_hydrology_manual.pdf?sfvrsn=16

Figure 403-5 Rational “C” Coefficient Mountain (Grass and Brush)



Source: Arizona Department of Transportation, 2014, "Highway Drainage Design Manual, Volume 2, Hydrology, Second Edition", Figure 2-5, p. 2-12.
http://www.azdot.gov/docs/default-source/roadway-engineering-library/2014_adot_hydrology_manual.pdf?sfvrsn=16

Figure 403-6 Rational "C" Coefficient Mountain (Pinion, Juniper & Grass)



Source: Arizona Department of Transportation, 2014, “Highway Drainage Design Manual, Volume 2, Hydrology, Second Edition”, Figure 2-6, p. 2-13.
http://www.azdot.gov/docs/default-source/roadway-engineering-library/2014_adot_hydrology_manual.pdf?sfvrsn=16

Figure 403-7 Rational “C” Coefficient Mountain (Ponderosa)

Appendix 6 contains **Example Problem 6-1** and **Example Problem 6-2.**

Example Problem 6-1 and is a smaller site (34 acres) with 55% imperviousness located in central New Mexico. **Example Problem 6-2** is larger site (80 acres) with a more natural basin the demonstrates an area weighted Runoff Coefficient "C" calculation.

403.4 References

Arizona Department of Transportation, 2014, "Highway Drainage Design Manual, Volume 2, Hydrology, Second Edition".
http://www.azdot.gov/docs/default-source/roadway-engineering-library/2014_adot_hydrology_manual.pdf?sfvrsn=16

NOAA Hydrometeorological Design Studies Center Precipitation Frequency Data Server. (PFDS).
http://hdsc.nws.noaa.gov/hdsc/pfds/

NRCS, "Part 630 Hydrology, National Engineering Handbook". Note that various Chapters have different dates.
https://www.nrcs.usda.gov/wps/portal/nrcs/detailfull/national/water/manage/hydrology/?cid=stelprdb1043063

NRCS, 2010, "Part 630 Hydrology, National Engineering Handbook, Chapter 15 Time of Concentration".
http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=27002.wba

Roussel, M.C., Asquith, W. H., Thompson, D. B., Cleveland, T. G., and Fang, X., 2005, "Summary of Dimensionless Texas Hyetographs and Distribution of Storm Depth, Developed for Texas Department of Transportation Research Project 0-4194", U.S. Geological Survey, Austin, TX. (TxDOT 0-4194-4).
http://library.ctr.utexas.edu/digitized/texasarchive/phase1/4194-4-TxDOT.pdf

TxDOT, July 2016, "Hydraulic Design Manual," Chapter 4, Section 11.
http://onlinemanuals.txdot.gov/txdotmanuals/hyd/index.htm

404 NRCS Simplified Peak Discharge Method

404.1 General

The NRCS Simplified Peak Discharge Method estimates the peak rate of runoff and runoff volume from small to medium size watersheds (≤ 10 square miles). This method was developed by the Soil Conservation Service (now the NRCS) for use in New Mexico, and was originally developed in October 1973. This document was revised in 1985 titled "Peak Rates of Discharge for Small Watersheds, Chapter 2, Engineering Field Manual for Conservation Practices", SCS, February 1985. **APPENDIX 5** contains a copy that document. In April 2014, Supplemental Notice No. NM-36 was developed as a modification to the 1985 document. NM-36 only prescribed to replace the previous document (1985) rainfall data with NOAA Atlas 14 rainfall data.

The original Chapter 2 method (SCS, 1973) included unit peak discharge curves for different rainfall distributions, varying from 45% to 85% of the rainfall occurring in the peak hour.

After analysis of stream gage data, the 1985 update included only one peak discharge curve, representing a variable rainfall distribution depending on the Tc of the watershed. This curve is shown in **Figure 404-1**. Therefore, a separate estimate of rainfall distribution is not required to use this method. The analysis of gage data also showed that the method overestimated peak discharges at elevations above 7500 ft. Drainage structures above this elevation should be evaluated by the Unit Hydrograph Method (**Section 405**). The completion of the "Simplified Peak Discharge Method Worksheet" (**Figure 404-2**) is required when using this method. The NOAA Atlas 14 references and links are provided here.

NOAA, Rev. ed. 2011, "Atlas 14, Precipitation-Frequency Atlas of the United States Volume 1 Version 5.0".
http://www.nws.noaa.gov/oh/hdsc/PF_documents/Atlas14_Volume1.pdf

Precipitation Frequency Data Server (PFDS):
http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=nm

The use of the PFDS is preferred due to the accuracy with which point rainfall amounts may be determined using the digital map based tools.

Infiltration and other losses are estimated using the NRCS Curve Number (CN) methodology. Input parameters are consistent with those used in the NRCS Unit Hydrograph Method. The Simplified Peak Discharge Method is limited for NMDOT use to single basins less than 10 square miles in area and should not be used when Tc exceeds 10.0 hours. When Tc is less than 10 minutes, use 10 minutes. This method may be used on NMDOT projects for those conditions identified in **Section 401** (**Figure 401-1** and **Figure 401-2**) of this manual. This method should not be used for watersheds with perennial streamflow. In the case of perennial streams, use the method described in **Section 406** if a stream gage exists, or the method described in **Section 405**, and include base flow.

The NMDOT Drainage Design Bureau can be contacted to obtain a copy of a spreadsheet used to calculate flows via the SCS/NRCS Simplified Peak Discharge Method. Note that the

Engineer/Consultant is responsible for understanding the use of, and the accuracy of the results from this spreadsheet.

404.2 Limitations

The NRCS Simplified Peak Discharge Method limitations are as follows:

- Do not use on watersheds larger than 10 square miles
- Do not use when more than 30% of the drainage area is urban
- Do not use when more than 30% of the watershed is above 7500 feet in elevation
- Do not use a Tc of less than 10 minutes (0.16 hours) or greater than 10 hours
- Do not use on watersheds with perennial streams
- Do not use on areas impacted by significant snowmelt or recently impacted by severe wildfire

404.3 Factors Affecting Runoff

Precipitation is the source of runoff from small watersheds. The soils and vegetation of the watershed affect the amount of precipitation that runs off. Mechanical treatment on a watershed, along with its topography and shape, also affect the rate at which water runs off. Runoff Curve Numbers (CNs) represent the combined effect of soil, vegetative cover, and conservation practices in runoff determinations. Transmission or channel losses in sand and gravel bed channels can also significantly affect the volume and peak discharge arriving at the point of interest in a watershed.

NRCS, 2007, Part 630 National Engineering Handbook, Chapter 19, Transmission Losses, provides guidance for calculating the impacts of these losses on the flood hydrograph. If the engineer believes that transmission losses have a significant impact on flows in the basin, the analysis should not be performed using the Simplified Peak Discharge Method, but rather the Unit Hydrograph Method in HEC-HMS (**Section 405**).

404.4 Precipitation

The highest rates of runoff from small watersheds are usually caused by intense rainfall. The intensity of rainfall affects the rate of runoff more than it does the volume of runoff. Intense rainstorms that produce high rates of runoff in small watersheds usually do not extend over a large area. The same intense rainstorm that causes flooding in a small tributary is not likely to be the one that will cause major flooding in a main watercourse that drains many square miles. Data from recording rain gages were studied to determine an appropriate rainfall distribution for New Mexico. Generally, New Mexico has more intense, shorter duration rainfalls than other parts of the U.S.

The melting of accumulated snow in the mountains may result in a greater volume of runoff, but usually at a lesser rate than runoff caused by rainfall. The melting of a winter's snow accumulation over a large area may cause major flooding along rivers.

The Simplified Peak Discharge Method requires the 24-hour total precipitation depth, and the method is applicable to the 100-yr storm and all more frequent recurrence interval storms.

Obtain the 24-hour rainfall depth directly from the NOAA Precipitation Frequency Data Server (PFDS) as described in **Section 403.2**. For NMDOT projects, there is no reduction factor for partial series versus annual series applied to 2-year, 5-year, and 10-year rainfall depths. This represents a slight departure from the original NRCS Method (NRCS, 1985-2014) and adds a small percentage of safety factor for the more frequent return period events.

The time distribution of rainfall is built into the Simplified Peak Discharge Method. This statewide rainfall distribution varies from 45% to over 85% of the 24-hour rainfall occurring in the peak hour of the storm as the Time of Concentration (Tc) varies from 10 minutes to 10 hours.

For NMDOT drainage design, find the 24-hour rainfall depth from the NOAA Precipitation Frequency Data Server for the centroid of each watershed.

404.5 Antecedent Runoff Condition

The amount of precipitation occurring in the five days preceding the storm in question is an indication of the Antecedent Runoff Condition (ARC) of the soil. The CNs in **Table 402-2** to **Table 402-5** are for an average ARC II. Watersheds in New Mexico most often meet an ARC I or ARC II condition. NRCS has over 60 years of experience in the sizing of flood control dams around New Mexico using ARC II as the design condition. Experience has shown that the use of ARC II is conservative in that as it has been extremely rare for the emergency spillway on one of their dams to flow (a majority of these dams were designed for the 25-year or 50-year flood event). ARC III provides a very conservative assumption and generates significantly larger peak discharges and runoff volumes than ARC II for the same Curve Number and is typically not the case for most watersheds in New Mexico. Therefore, **use ARC II for NMDOT projects.**

404.6 Hydrologic Soil Groups

The texture, composition and density of soils have a direct impact on the amount and rate at which rainfall becomes runoff, and therefore, the soil type is a critical piece of information in the development of rainfall/runoff calculations. In general, soils are classified as sandy, silty, loamy or clayey. In nature, there can be an infinite number of combinations of these characteristics. The NRCS has divided the extremely wide range of soil textures by their hydrologic (runoff producing) characteristics into four Hydrologic Soils Groups: Type A, B, C and D. Type A soils are generally sandy soils and low runoff producers and Type D are clayey soils and high producers of runoff for a given rainfall volume. Types B and C soils runoff characteristics are subdivisions within the range of A to D.

Information regarding the soils in a watershed has been surveyed by NRCS and other agencies for almost the entire country including the State of New Mexico. This information is generally available from the NRCS by consulting the Natural Resources Conservation Service's (NRCS) Field Office Technical Guide; or the Web Soil Survey website.
http://websoilsurvey.nrcs.usda.gov/

Occasionally, when dealing with public lands (U.S. Forest Service, BLM, military bases) the soils information will not be shown in the NRCS database but may be available from the land management agency responsible for those lands.

For an expanded discussion and instructions on soils and their effects on runoff, see **Sections 402.4, 402.5,** and **402.6**. See also **Example Problem 6-7** located in Appendix 6 for a technical paper titled "Hatch Site 6 Runoff Methods Revisited" as an example of an approach for searching more deeply into predicted runoff results.

404.7 Vegetative Cover

Vegetation affects runoff in several ways including the following:

- The foliage and its litter maintain the soil's infiltration potential by preventing the sealing of the soil surface from raindrop impact
- Foliage retains some of the raindrops, increasing their chance of being evaporated
- Some of the moisture is intercepted on the plant and withheld from the initial period of runoff
- Vegetation transpires soil moisture leaving a greater void in the soil to be filled
- Vegetation, including its ground litter, forms numerous barriers along the path of the water flowing over the surface of the land (this lengthens the travel time and increases opportunity for infiltration)

The following information can be used as a guide in determining the vegetative cover conditions for range sites. Grass cover is evaluated on plant basal area while trees and shrubs are evaluated using canopy cover. Litter can be an effective cover and should be considered.

Cover Condition Class

Condition	Vegetative Cover
Poor	Less than 30% ground cover
Fair	About 30% to 70% ground cover
Good	More than 70% ground cover

Refer to NRCS NEH Part 630, (EFH) Amend. IA50, Nov. 2007 "Hydrologic Soil-Cover Complexes".
https://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/nrcs142p2_022388.pdf

For a more complete guide to determining the percentage of vegetative cover, see "Sampling Vegetation Attributes" Interagency Technical Reference 1996 (Rev. ed. 1997 and 1999) at:
http://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/stelprdb1044175.pdf

For a more detailed discussion and instructions on determining the appropriate Cover Conditions see **Sections 402.5 and 402.6** and the example Soil Cover Complex photographs presented in **APPENDIX 4**.

404.8 Conservation Practices

Conservation practices, in general, reduce sheet erosion and thereby maintain an open structure of the soil surface. Soil and water conservation practices are control measures

consisting of managerial, vegetative, and structural practices to reduce the loss of soil and water. The application of conservation practices across a watershed reduces the volume of runoff, but the effect diminishes rapidly with increased storm magnitude. Some types of these practices are discussed below. Visit the NRCS website for more detailed information regarding conservation practices.
https://www.nrcs.usda.gov/wps/portal/nrcs/detailfull/national/technical/cp/ncps/?cid=nrcs143_026849

Crop residue tilled into the soil and the residual root system from grasses that have been in the crop rotations produce a condition favoring greater infiltration and water storage in the soil profile. The effect of conservation tillage on reducing runoff ranges from slight to substantial.

Contouring and terracing reduce sheet erosion and increase the amount of rainfall withheld from runoff by the small reservoirs they form. Land areas in which level terraces have been constructed may be excluded from the drainage area above downstream measures if they store the design depth of runoff. Gradient terraces increase the distance water must travel and thereby increase the Time of Concentration. This, in turn, reduces the peak rate of discharge.

Watershed slopes affect the rate of runoff and the peak discharge rate at downstream points. Slopes have a smaller effect on the volume of runoff than conservation practices such as contouring and terracing.

Small depressions may trap an initial amount of rain, thus reducing the amount of expected runoff. Where ponding or swampy areas occur in the watershed, a considerable amount of surface runoff may be retained in temporary storage. NRCS Small Watershed Hydrology WinTR-55 User Guide, 2009 contains a procedure to adjust the peak discharge for ponded areas.
http://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/stelprdb1042897.pdf

404.9 Runoff Curve Number (CN)

The NRCS Runoff Curve Number (CN) is a lumped watershed parameter and most often serves as a proxy for all losses to precipitation from the time it hits the ground surface until it reaches the point of interest in a hydrologic analysis. As such, it should not be interpreted as a point infiltration value but rather as representing all losses (capture, infiltration, transmission, evaporation, etc.) unless separate calculations will be made for ponding and transmission losses.

Sections 402.5 and **402.6** contain important and useful excerpts from NRCS, June 1986, TR-55, Urban Hydrology for Small Watersheds, Chapter 2, which provides a complete and clear explanation of the CN, its determination, and its use in hydrologic analyses.
www.wcc.nrcs.usda.gov/ftpref/wntsc/H&H/other/TR55documentation.pdf

404.10 Time of Concentration

Calculate the Time of Concentration (Tc) for use in the Simplified Peak Discharge Method using the Upland Method for un-gullied watersheds and the upper, un-gullied portions of somewhat gullied watersheds. Use the Kirpich Equation for the gullied portions of the watershed and for watersheds that are almost entirely gullied. Follow the guidance in **Section 402.8**.

404.11 Peak Discharge Application Procedure

Step 1 – Gather input data.

Use the Simplified Peak Discharge Method worksheet **Figure 404-2**. Establish the appropriate Design Frequency Flood(s) for analysis (**Section 200**).

- Measure the drainage area, (A), in acres
- Compute the Time of Concentration, (Tc), in hours (**Sections 402.8** and **402.9**)
- Determine the appropriate Runoff Curve Number, CN, for the drainage basin (**Sections 402.5** and **402.6**)
- Obtain the 24-hour rainfall depth, P_{24}, in inches, for the appropriate design frequency, from NOAA Atlas 14 or online from the NOAA PFDS

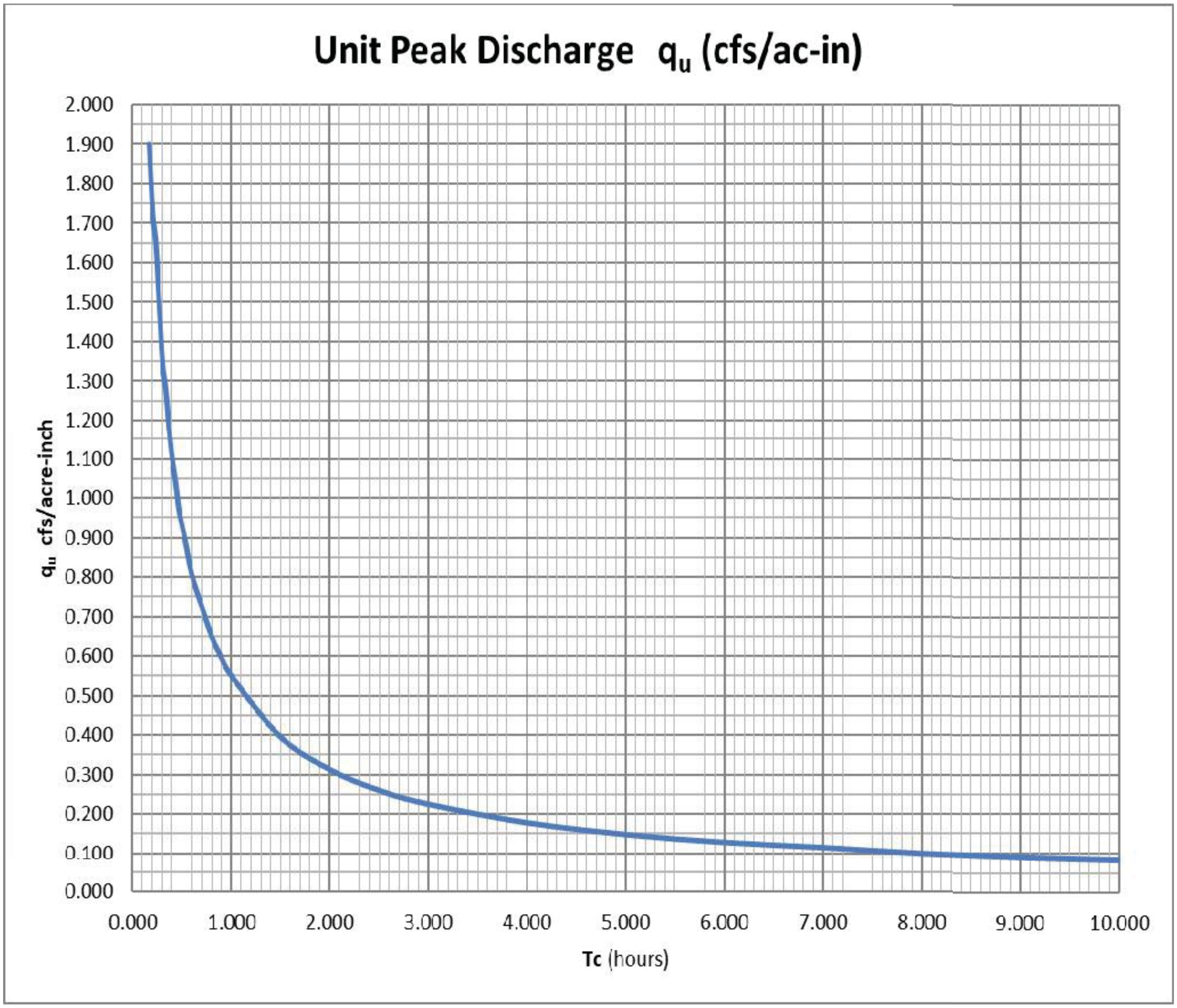
Step 2 – Determine the unit peak discharge, qu, for the watershed.

The unit peak discharge, $q_{u,}$ in cfs/ac-in. can be read from **Table 404-1** or **Figure 404-1**, given the Tc.

Table 404-1 Unit Peak Discharge Table for NRCS Simplified Peak Discharge Method

Source: Soil Conservation Service, 1973, revised by Luther McDougal, and Calvin Jackson, 1973, updated by Larry Goertz, February 1985, updated by Roger Ford, 2014, "Peak Rates of Discharge for Small Watersheds, Chapter 2, Engineering Field Manual for Conservation Practices".
(Not available on-line – see **APPENDIX 5**).

Tc		qu	Tc		qu
hours	min	cfs/ac-in	hours	min	cfs/ac-in
0.167	10.000	1.900	1.500	90	0.395
0.200	12.000	1.730	2.000	120	0.313
0.233	14.000	1.650	2.500	150	0.260
0.267	16.000	1.500	3.000	180	0.225
0.300	18.000	1.350	3.500	210	0.202
0.333	20.000	1.280	4.000	240	0.178
0.367	22.000	1.180	4.500	270	0.163
0.400	24.000	1.100	5.000	300	0.148
0.433	26.000	1.040	5.500	330	0.138
0.467	28.000	0.970	6.000	360	0.128
0.500	30.000	0.930	6.500	390	0.122
0.533	32.000	0.890	7.000	420	0.115
0.567	34.000	0.848	7.500	450	0.108
0.600	36.000	0.805	8.000	480	0.100
0.633	38.000	0.778	8.500	510	0.095
0.667	40.000	0.752	9.000	540	0.090
0.700	42.000	0.725	9.500	570	0.087
0.733	44.000	0.688	10.000	600	0.083
0.800	48.000	0.650			
0.867	52.000	0.623			
0.900	54.000	0.595			
1.000	60.000	0.550			



Source: Soil Conservation Service, 1973, revised by Luther McDougal, and Calvin Jackson, 1973, updated by Larry Goertz, February 1985, updated by Roger Ford, 2014, "Peak Rates of Discharge for Small Watersheds, Chapter 2, Engineering Field Manual for Conservation Practices".
(Not available on-line – see **APPENDIX 5**).

Figure 404-1 Unit Peak Discharge for NRCS Simplified Peak Discharge Method

If not using **Figure 404-1,** then read the unit peak discharge (q_u) value from **Table 404-1.**

Calculate the direct runoff depth (Q) from the watershed. The direct runoff is expressed as an average depth of runoff (Q) over the entire watershed, in inches. The direct runoff may be read from **Figure 402-8** using the 24-hour rainfall depth (P) in inches, and the Runoff Curve Number, CN.

The direct runoff depth (Q) may also be calculated from the following equation:

$$Q = \frac{\left[P - \left(\frac{200}{CN}\right) + 2\right]^2}{P + \left(\frac{800}{CN}\right) - 8} \qquad \textbf{404-1}$$

(Soil Conservation Service,1973, revised by Luther McDougal, and Calvin Jackson, 1973, updated by Larry Goertz, February 1985, updated by Roger Ford, 2014, “Peak Rates of Discharge for Small Watersheds, Chapter 2, Engineering Field Manual for Conservation Practices”)

where:

Q	=	direct runoff, inches
P	=	rainfall depth, inches
CN	=	Runoff Curve Number

Note that this method was developed based the 24-hour rainfall duration (P), with the maximum return period of 100-years, and is also applicable for more frequent return periods. The direct runoff depth (Q) may sometimes be shown as Q_d, to indicate depth, and to distinguish this term from the letter Q, which is also used often to designate discharge in cubic feet per second (cfs).

Step 3 – Compute the peak discharge

Compute the peak discharge (Q_p) from the watershed by the following equation:

$$Q_p = A\ Q\ q_u \qquad \textbf{404-2}$$

where:

Q_P	=	peak discharge, cfs
A	=	drainage area, acres
Q	=	direct runoff, inches
q_u	=	unit peak discharge, cfs/acre-inch

Step 4 – Compute the runoff volume, if required.

The runoff volume (Q) is obtained by the equation:

$$Q_v = (Q\ \ A) / 12 \qquad \textbf{404-3}$$

where:

Q	=	direct runoff, inches
Q_v	=	runoff volume from the watershed, ac-ft
A	=	drainage area, acres

Step 5 – Estimate Transmission Losses

Transmission losses shall not be applied when using the Simplified Peak Discharge Method except for water quality and sediment transport related applications. For small frequent rainfall events and water quality analyses, transmission losses can be significant and should be considered. For sediment transport analyses, transmission losses should be considered to avoid over estimation of sediment transport rates.

Simplified Peak Discharge Method Worksheet

Structure Location: MP: ____________________ County: ________________

District: ____________________

Structure Description: ____

Drainage Area: **A** = ______________ acres, ____________ mi^2

Elevation at Centroid of Watershed: **Elev** = __________________ ft *

Location of Centroid: **Lat:** _______________ **Long**: ___________________

Time of Concentration: **T_C** = _______________ hours

Method: ☐ Upland ☐ Kirpich ☐ Mixed

Weighted Runoff Curve Number: **CN** = ____________________

Method: ☐ Area ☐ Runoff

Unit Peak Discharge (from Figure 404-1): **q_u** = _______________ cfs/ac-in

Design Frequency Flood	___________ - year	___________ - year
24-hour Rainfall Depth (NOAA PFDS):	**P_{24}** ___________ in.	**P_{24}** = _____________ in.
Direct Runoff (Figure 402-8):	**Qd** = __________ in.	**Qd** = _____________ in.
Peak Discharge, Qp = A • Qd • qu:	**Qp** = __________ cfs	**Qp** = _____________ cfs
Discharge per acre	____________ cfs/ac	______________ cfs/ac
Runoff Volume, Qv = A • Qd/12:	**Qv** = _________ ac-ft	**Qv** = ______________ ac-ft

Project Location: ______________________________

CN#: ______________________________

Date: ______________________________

Computed By: ______________________________

Checked By: ______________________________

* If elevation is greater than 7500 ft, use NRCS Unit Hydrograph method

Source: Soil Conservation Service,1973, revised by Luther McDougal, and Calvin Jackson, 1973, updated by Larry Goertz, February 1985, updated by Roger Ford, 2014, "Peak Rates of Discharge for Small Watersheds, Chapter 2, Engineering Field Manual for Conservation Practices".

Figure 404-2 Simplified Peak Discharge Method Worksheet

Appendix 6 contains two example Simplified Peak Discharge Method problems. **Example Problem 6-3** is for a mid-size basin (7.6 sq mi) and **Example Problem 6-4** is for a small basin (1.07 sq mi).

404.12 References

NOAA, Rev. ed. 2011, "Atlas 14, Precipitation-Frequency Atlas of the United States Volume 1 Version 5.0".
http://www.nws.noaa.gov/oh/hdsc/PF_documents/Atlas14_Volume1.pdf

NOAA Hydrometeorological Design Studies Center Precipitation Frequency Data Server (PFDS).
http://hdsc.nws.noaa.gov/hdsc/pfds/

NRCS Website, "Conservation Practices".
https://www.nrcs.usda.gov/wps/portal/nrcs/detailfull/national/technical/cp/ncps/?cid=nrcs143_026849

NRCS Web Soil Survey Website.
https://websoilsurvey.sc.egov.usda.gov/App/HomePage.htm

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NRCS, 2007, "Part 630 Hydrology, National Engineering Handbook, Chapter 19, Transmission Losses".
http://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/16/stelprdb1043097.pdf

NRCS, November 2007, "Part 630 Hydrology, National Engineering Handbook, (EFH) Amend. IA50, Hydrologic Soil-Cover Complexes".
https://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/nrcs142p2_022388.pdf
http://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/nrcs142p2_011485.pdf.

NRCS, 2009, "Small Watershed Hydrology WinTR-55 User Guide".
http://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/stelprdb1042897.pdf

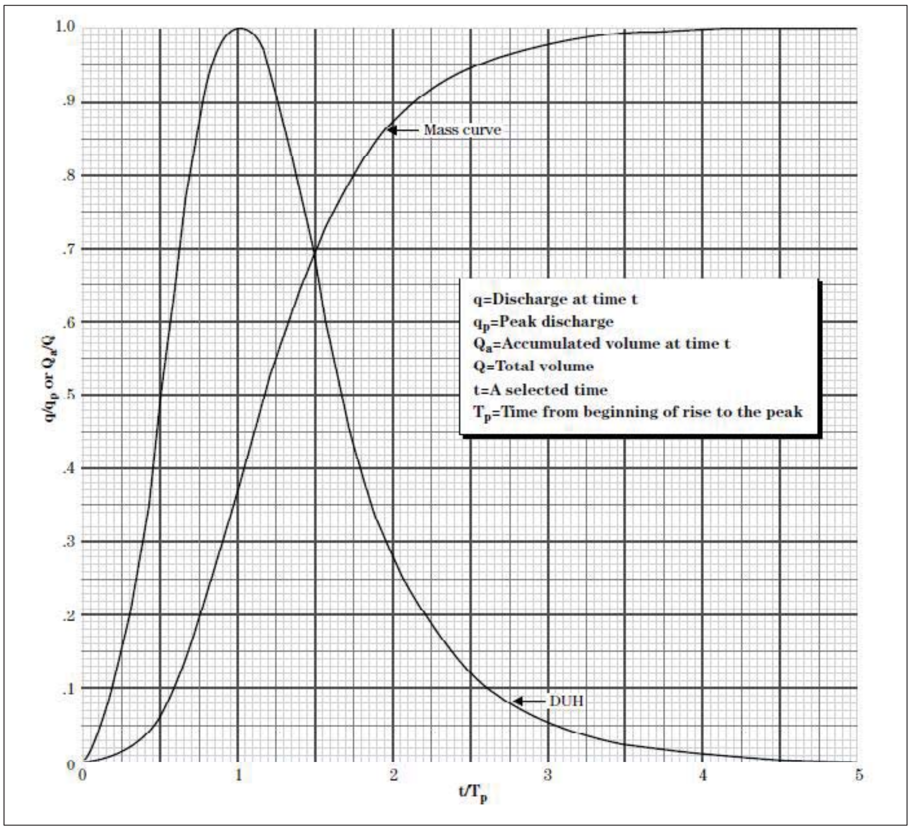
NRCS "Part 630 Hydrology, National Engineering Handbook, Chapters 8-12".
http://www.nrcs.usda.gov/wps/portal/nrcs/detailfull/national/water/?cid=stelprdb1043063

Soil Conservation Service, 1973, revised by Luther McDougal, and Calvin Jackson, 1973, updated by Larry Goertz, February 1985, updated by Roger Ford, 2014, "Peak Rates of Discharge for Small Watersheds, Chapter 2, Engineering Field Manual for Conservation Practices". (Not available on-line see **APPENDIX 5**).

405 NRCS (SCS) Unit Hydrograph Method within HEC-HMS

While there are multiple computer programs that can be used to develop a hydrograph, the NRCS Synthetic Unit Hydrograph Method has been selected for use on NMDOT projects in order to simplify reviews and to improve consistency. This method shall be used for watersheds over 10 square miles, or which have centroids above 7500 feet and whenever peak discharge calculations involve multiple subbasins and complex hydraulics within and among subbasins. The method should also be used whenever the analysis includes flood routing through detention facilities, pump stations, or long conveyance facilities. Synthetic unit hydrographs can be used to model drainage basins with or without base flow.

A hydrograph is a plot of discharge versus time. Synthetic unit hydrograph methods are used to adjust the shape of a generalized hydrograph to a particular drainage basin, usually at an ungaged site. A unit hydrograph is defined as the direct runoff hydrograph resulting from a rainfall event which has a specific temporal and spatial distribution, and which generates a unit depth of rainfall. The area beneath the unit hydrograph curve is equal to the volume of direct runoff from one inch of excess rainfall over the entire drainage basin or subbasin. **Figure 405-1** shows a dimensionless unit hydrograph and its associated cumulative mass curve.



Source: NRCS, 2007, "Part 630 Hydrology, National Engineering Handbook", Chapter 16, Hydrographs, Figure 16-1, p. 16-3. http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=17755.wba

Figure 405-1 Dimensionless Unit Hydrograph and Mass Curve

The NRCS Unit Hydrograph was developed through the analysis of a large number of natural (measured) unit hydrographs from a broad cross section of geographic locations and hydrologic regions around the continental United States.

Computer models are the preferred approach for application of the SCS (now NRCS) Synthetic Unit Hydrograph Method. These computation methods make creation, addition, and routing of multiple hydrographs a relatively easy task.

There are commercially available software programs such as WMS and AutoDesk that perform hydrologic modeling. However, the NMDOT model of choice for large and/or complex watersheds and anytime a hydrograph is needed, is the U.S. Army Corps of Engineers (USACE) program HEC-HMS. Appendix 6 contains **Example Problem 6-6** that presents an example of a HEC-HMS problem.

The program, the User's Manual, the Technical Reference Manual, the Application Guide and sample models are available as free downloads from the USACE Hydrologic Engineering Center at:
http://www.hec.usace.army.mil/software/hec-hms/

HEC-HMS version 4.2.1 (latest version at the time of the publication of this manual) is capable of performing a wide variety of hydrologic analyses. With the GIS companion product (HEC-GeoHMS) data collection, basin delineation and rainfall input parameters have been simplified and made reproducible.

Basic data for HEC-HMS is standard to nearly all hydrologic analyses models as follows:

- Drainage basin area
- Time of Concentration
- Rainfall/Runoff algorithm (in this case Runoff Curve Number)
- Total rainfall depth
- Rainfall temporal distribution
- Conveyance system hydraulic data

Detailed instructions for the construction of a HEC-HMS model are not included in this manual since they are extensive and well presented in the HEC-HMS User's and Technical Reference Manuals. HEC-HMS has been updated several times since its introduction, and its capabilities are modified and expanded with each version. Also, since the use of the most current version is recommended, the inclusion of detailed usage instructions which are subject to change in this manual is not practical.

There are some basic requirements for use of a hydrologic computer model on a NMDOT project.

- Use of a computer model other than HEC-HMS must be approved by the NMDOT Drainage Design Bureau prior to its use.
- **<u>The rainfall distribution used must be the 25% frequency</u>** produced by HEC-HMS from rainfall data from NOAA Atlas 14 or the NOAA Precipitation Frequency Data Server for the specific flood frequency and watershed under investigation, unless otherwise authorized by the Drainage Design Bureau (see **Section 405.3** for further explanation).
- Tc must be computed using the Iterative Method within the Stream Hydraulic Method, and/or the Upland Method as appropriate. The use of the Kirpich Equation is appropriate for checking the results from **Section 402.9.5**. Refer to **Table 402-6** for guidance on

selection of a Time of Concentration method. Complete input files, routing diagrams, and summary output files must be included (in an appendix) in every drainage report, as well as the HEC-HMS Method worksheet (see **Figure 405-9**).

- When hydrograph routing is required, the Muskingum-Cunge Method is preferred for use with the NRCS Unit Hydrograph procedure. On occasion, special circumstances may warrant the use of one of the other routing methods available within HEC-HMS. Consult with the Drainage Design Bureau before using an alternative method.

405.1 Basin Delineation

Regardless of the hydrologic analysis method selected including HEC-HMS, the area of a drainage basin and its subbasins are always required. Basic to all hydrologic methods is the assumption that the basin or subbasin can be reasonably characterized by one set of hydrologic characteristics (soils, slope, rainfall, vegetative cover, and land use). The further the basin and subbasin characteristics diverge from this assumption, the less accurate and reproducible the results will be. Good “rules of thumb” regarding basin and subbasin sizing are that the length of a basin or subbasin delineation should not exceed 4 times its width and that no subbasin should be more than 10 times larger than the smallest subbasin.

Section 402.2 contains a more detailed description of the hydrologic factors that should be considered when delineating basins and subbasins. Also refer to the discussion in **Section 405.9** regarding minimum Tc and model computation interval as they relate to basin size and modeling.

405.2 Rainfall Volume

The rainfall depths for the design frequency storm are to be found at the NOAA Precipitation Frequency Data Server for the centroid of the watershed being studied (using the Partial Duration Series). In very large watersheds, the use of different rainfall volumes for portions of the watershed may be appropriate (e.g. mountain faces might differ from the alluvial plains below). Rainfall depths for specific durations (i.e. 5 minute, 15 minute, 60 minute, etc.) are also provided. These values are inputs to HEC-HMS for development of the 25% design rainfall temporal distribution used in the NRCS Unit Hydrograph Method.

405.3 Rainfall Temporal Distribution

Proper application of this method requires use of a 24-hour rainfall event with the peak precipitation rate occurring at 6 hours. Rainfall data for the NRCS Unit Hydrograph Method consists of point precipitation depths for various durations up to and including the 24-hour point depth, and also requires a rainfall distribution. Point precipitation depths for the design return period may be obtained directly from NOAA Atlas 14 or the NOAA Precipitation Frequency Data Server.

Previously, the rainfall distribution prescribed for use on NMDOT projects with the NRCS (SCS) Unit Hydrograph Method was called the Modified NOAA-SCS rainfall distribution. This Modified NOAA-SCS rainfall distribution was a combination of the peak rainfall intensity defined by NOAA, with an NRCS Type II-a storm rearrangement. HEC-HMS does not have a built in NRCS Type II-a storm distribution.

However, the 25% frequency storm distribution available within HEC-HMS is a very close approximation and is prescribed for NMDOT hydrologic analyses wherever a rainfall distribution is required. Given that NOAA Atlas 14 has a greatly expanded database compared to the data available to the U.S. Weather Bureau at the time the Type II-a distribution was developed, the 25% distribution available in the HEC-HMS program should produce more accurate results throughout New Mexico.

For NMDOT drainage design projects, apply the 25% frequency storm distribution. The HEC-HMS User's Manual describes the method for creating model rainfall distributions. **Figure 405-2** and **Figure 405-3** are provided for additional guidance.

NOAA Atlas 14, Volume 1, Version 5
Location name: Mountainair, New Mexico, US*
Latitude: 34.3000°, Longitude: -106.1000°
Elevation: 6500 ft*
* source: Google Maps

POINT PRECIPITATION FREQUENCY ESTIMATES

Sanja Perica, Sarah Dietz, Sarah Heim, Lillian Hiner, Kazungu Maitaria, Deborah Martin, Sandra Pavlovic, Ishani Roy, Carl Trypaluk, Dale Unruh, Fenglin Yan, Michael Yekta, Tan Zhao, Geoffrey Bonnin, Daniel Brewer, Li-Chuan Chen, Tye Parzybok, John Yarchoan

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	0.246 (0.216–0.280)	0.318 (0.280–0.362)	0.422 (0.371–0.482)	0.502 (0.440–0.574)	0.608 (0.534–0.696)	0.689 (0.606–0.793)	0.775 (0.681–0.897)	0.862 (0.759–1.00)	0.979 (0.863–1.15)	1.07 (0.947–1.28)
10-min	0.375 (0.329–0.427)	0.484 (0.425–0.551)	0.643 (0.564–0.734)	0.764 (0.669–0.874)	0.925 (0.812–1.06)	1.05 (0.922–1.21)	1.18 (1.04–1.37)	1.31 (1.16–1.53)	1.49 (1.31–1.76)	1.63 (1.44–1.94)
15-min	0.464 (0.408–0.529)	0.599 (0.527–0.683)	0.797 (0.700–0.910)	0.947 (0.829–1.08)	1.15 (1.01–1.31)	1.30 (1.14–1.50)	1.46 (1.28–1.69)	1.63 (1.43–1.89)	1.85 (1.63–2.18)	2.02 (1.79–2.41)
30-min	0.625 (0.549–0.713)	0.807 (0.710–0.920)	1.07 (0.942–1.23)	1.28 (1.12–1.46)	1.54 (1.36–1.77)	1.75 (1.54–2.02)	1.97 (1.73–2.28)	2.19 (1.93–2.55)	2.49 (2.19–2.93)	2.73 (2.41–3.24)
60-min	0.774 (0.680–0.882)	0.999 (0.878–1.14)	1.33 (1.17–1.52)	1.58 (1.38–1.81)	1.91 (1.68–2.19)	2.17 (1.90–2.49)	2.44 (2.14–2.82)	2.71 (2.39–3.16)	3.08 (2.72–3.63)	3.37 (2.98–4.01)
2-hr	0.907 (0.799–1.04)	1.16 (1.02–1.33)	1.53 (1.35–1.75)	1.82 (1.60–2.07)	2.23 (1.95–2.54)	2.57 (2.23–2.91)	2.93 (2.52–3.32)	3.31 (2.83–3.75)	3.86 (3.26–4.37)	4.32 (3.61–4.90)
3-hr	0.956 (0.848–1.09)	1.21 (1.08–1.39)	1.58 (1.39–1.81)	1.88 (1.65–2.14)	2.30 (2.00–2.61)	2.64 (2.28–2.99)	3.00 (2.58–3.41)	3.40 (2.90–3.85)	3.96 (3.34–4.50)	4.43 (3.70–5.04)
6-hr	1.08 (0.961–1.23)	1.37 (1.22–1.56)	1.75 (1.55–1.99)	2.06 (1.82–2.34)	2.49 (2.19–2.82)	2.84 (2.47–3.21)	3.21 (2.78–3.63)	3.61 (3.10–4.08)	4.16 (3.54–4.71)	4.62 (3.89–5.24)
12-hr	1.23 (1.09–1.39)	1.55 (1.37–1.75)	1.96 (1.74–2.22)	2.29 (2.02–2.59)	2.75 (2.42–3.11)	3.12 (2.73–3.52)	3.51 (3.05–3.96)	3.92 (3.39–4.43)	4.50 (3.84–5.09)	4.97 (4.21–5.63)
24-hr	1.42 (1.31–1.55)	1.78 (1.65–1.94)	2.23 (2.06–2.44)	2.59 (2.38–2.83)	3.08 (2.82–3.36)	3.47 (3.16–3.78)	3.87 (3.51–4.22)	4.28 (3.87–4.68)	4.85 (4.34–5.32)	5.29 (4.71–5.83)
2-day	1.58 (1.45–1.72)	1.98 (1.82–2.16)	2.47 (2.27–2.69)	2.86 (2.63–3.13)	3.40 (3.10–3.71)	3.83 (3.48–4.18)	4.27 (3.87–4.67)	4.73 (4.25–5.18)	5.35 (4.76–5.89)	5.85 (5.16–6.45)
3-day	1.72 (1.58–1.87)	2.15 (1.98–2.34)	2.68 (2.47–2.92)	3.11 (2.86–3.38)	3.69 (3.38–4.02)	4.15 (3.78–4.52)	4.63 (4.20–5.04)	5.12 (4.62–5.59)	5.79 (5.17–6.35)	6.33 (5.60–6.96)
4-day	1.85 (1.71–2.01)	2.32 (2.14–2.53)	2.89 (2.67–3.15)	3.35 (3.09–3.64)	3.98 (3.65–4.32)	4.48 (4.09–4.86)	4.98 (4.54–5.42)	5.51 (4.98–6.00)	6.23 (5.58–6.81)	6.80 (6.04–7.46)
7-day	2.25 (2.08–2.43)	2.81 (2.59–3.05)	3.48 (3.21–3.78)	4.02 (3.70–4.35)	4.74 (4.35–5.14)	5.30 (4.84–5.75)	5.87 (5.35–6.37)	6.45 (5.84–7.02)	7.24 (6.50–7.90)	7.86 (7.00–8.60)
10-day	2.53 (2.34–2.75)	3.16 (2.92–3.44)	3.94 (3.63–4.29)	4.54 (4.18–4.95)	5.38 (4.92–5.86)	6.03 (5.49–6.57)	6.69 (6.08–7.31)	7.37 (6.66–8.08)	8.30 (7.43–9.12)	9.02 (8.01–9.97)
20-day	3.36 (3.12–3.62)	4.19 (3.89–4.53)	5.15 (4.78–5.57)	5.88 (5.44–6.36)	6.83 (6.30–7.39)	7.55 (6.93–8.17)	8.26 (7.56–8.95)	8.96 (8.16–9.73)	9.88 (8.95–10.8)	10.6 (9.52–11.6)
30-day	4.06 (3.79–4.35)	5.06 (4.72–5.44)	6.17 (5.74–6.63)	6.99 (6.50–7.52)	8.04 (7.46–8.65)	8.82 (8.16–9.50)	9.58 (8.84–10.3)	10.3 (9.49–11.2)	11.3 (10.3–12.2)	12.0 (10.9–13.0)
45-day	5.11 (4.78–5.47)	6.36 (5.94–6.81)	7.67 (7.16–8.22)	8.63 (8.04–9.26)	9.83 (9.13–10.6)	10.7 (9.91–11.5)	11.5 (10.6–12.4)	12.3 (11.3–13.3)	13.3 (12.2–14.4)	14.1 (12.8–15.3)
60-day	6.00 (5.59–6.42)	7.47 (6.97–8.02)	9.00 (8.39–9.66)	10.1 (9.41–10.8)	11.5 (10.7–12.3)	12.4 (11.5–13.4)	13.4 (12.4–14.4)	14.2 (13.1–15.4)	15.3 (14.1–16.6)	16.1 (14.7–17.5)

[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.

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Figure 405-2 Sample NOAA Precipitation Frequency Data Sever Output

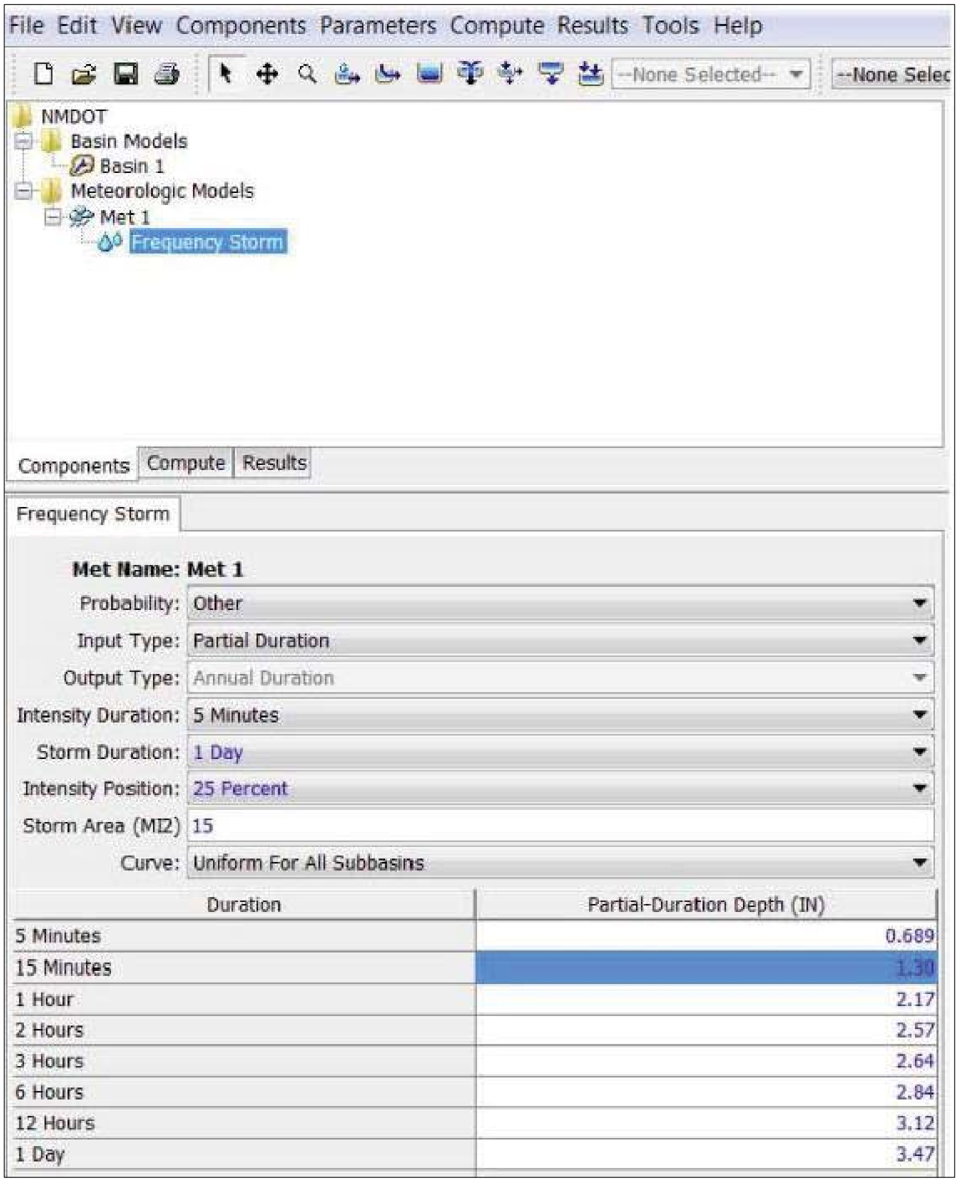


Figure 405-3 Sample HEC-HMS Precipitation Input Table

405.4 Soils Data

The NMDOT requires that hydrologic modeling within HEC-HMS utilize the NRCS Runoff Curve Number (CN) Method for determining a watershed's response to rainfall. Soils data (Hydrologic Soils Group) is integral to determining the CN.

The texture, composition and density of soils have a direct impact on the amount, and rate at which rainfall becomes runoff and, therefore, the soil type is a critical piece of information in the development of rainfall/runoff calculations. In general, soils are classified as sandy, silty, loamy or clayey. Of course, in nature, there can be an infinite number of combinations of these characteristics. The NRCS has divided the extremely wide range of soil textures by their hydrologic (runoff producing) characteristics into four Hydrologic Soils Groups: Type A, B, C, and D with: Type A being generally sandy soils and low runoff producers and Type D being clayey soils and high producers of runoff for a given rainfall volume. See **Section 402.4** for a more detailed description of soil classifications and their impact on the CN. Soils data are available for almost all of New Mexico from the NRCS Web Soil Survey at: http://websoilsurvey.nrcs.usda.gov/.

405.5 Hydrologic Soil Cover Complexes

A combination of a Hydrologic Soil Group (soil), land use and treatment class (cover) is a hydrologic soil-cover complex. A range of Runoff Curve Numbers (CNs) based on the combination of soil texture and cover has been developed by the NRCS from empirical data and is published by NRCS in their National Engineering Handbook, Chapter 9 as well in multiple other locations. **Section 402.5** contains a detailed description of the accepted process for determining appropriate soil cover complexes for use on NMDOT projects.

405.6 Runoff Curve Number

The NRCS Runoff Curve Number (CN) is a lumped watershed parameter and most often serves as a proxy for all losses from the beginning of precipitation until runoff reaches the point of interest in a hydrologic analysis. As such, it should not be interpreted as a point infiltration value but rather as representing all losses (initial abstraction, infiltration, transmission, evaporation, etc.) unless separate calculations will be made for ponding and transmission losses. **Section 402.6** contains a detailed description of the methods prescribed for determining the CN for NMDOT projects.

405.7 Other Land Use Effects

HEC-HMS has the ability to simulate the effects of directly connected impervious areas, ponds, dams, storm drains, and pump stations on the runoff hydrograph. The HEC-HMS User's Manual and the Technical Reference Manual should be consulted for the details regarding input data, limitations and capabilities of the software. Any NMDOT project that contains these elements and requires analyses of their impacts should utilize HEC-HMS unless approved by the Drainage Design Bureau.

Note that when modeling heavily urban basins, if the engineer inputs percentage impervious directly into the model, HEC-HMS assumes a CN=100 and produces 100% runoff from that area. <u>Impervious areas should be classified as CN=98. Do not use the percentage impervious option in HEC-HMS.</u>

405.8 Time of Concentration and Basin Lag

Time of Concentration (Tc), is defined as the time required for runoff to travel from the hydraulically most remote part of the watershed to the point of interest. The determination of Tc is one of the most important and sensitive drainage basin modeling needs when calculating the peak rate of runoff and hydrographs in HEC-HMS. Tc is a simplified proxy for the hydrologic response to precipitation by a watershed (capturing the interrelated effects of size, shape, and slope). The Tc for a watershed or subbasin has the most dramatic effect on the shape of the runoff hydrograph of any parameter. An accurate estimate of a watershed's Tc is therefore crucial to every type of hydrologic modeling. **Section 402.8** contains a detailed discussion and outlines the various methods approved to calculate and check Tc for a subbasin.

In the SCS (NRCS) Unit Hydrograph Method, basin lag (Lag or tlag) is defined as the time between the center of mass of excess rainfall and the peak of the unit hydrograph as:

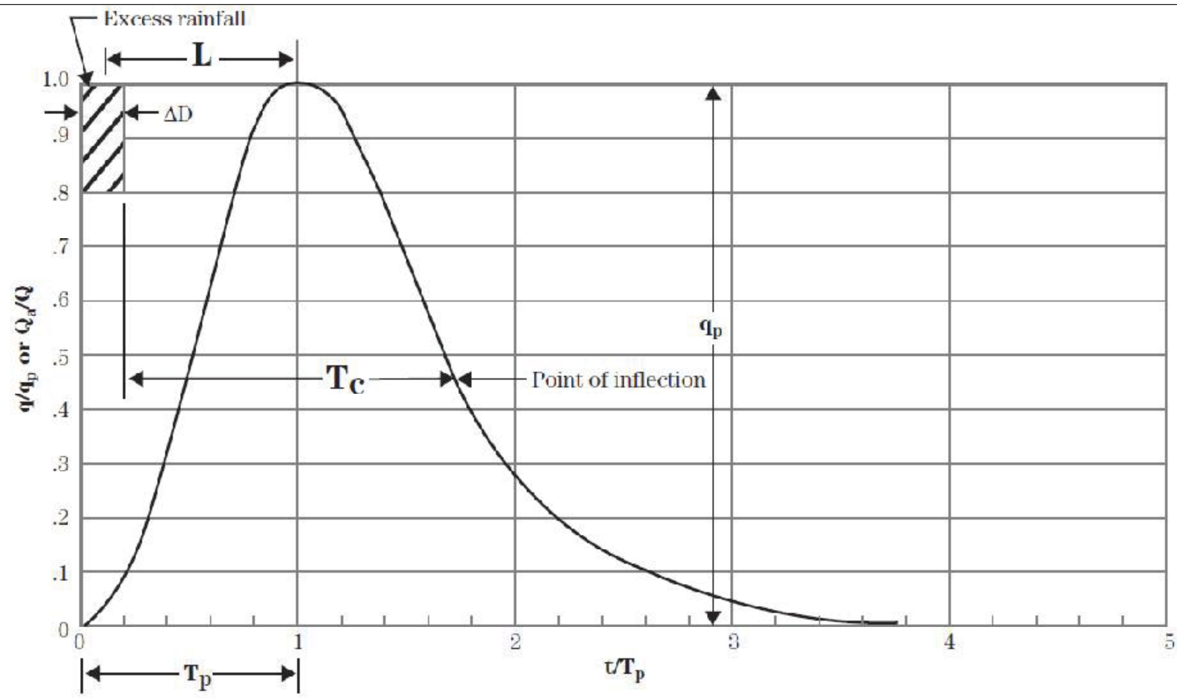
$$Lag = 0.6 \times Tc \qquad \textbf{405-1}$$

(NRCS, 2010, “Part 630 Hydrology, National Engineering Handbook, Chapter 15 Time of Concentration”, Eq. 15-3, p. 15-3)
https://directives.sc.egov.usda.gov/27002.wbas

where:

Lag	=	the time between the center of mass of excess runoff and the hydrograph peak, hr
Tc	=	time of concentration, hr

Figure 405-4 illustrates the various time relationships important to the development of the dimensionless unit hydrograph and resulting basin specific hydrographs within the NRCS Unit Hydrograph Method.



where:
L = Lag, h
T_c = time of concentration, h
T_p = time to peak, h
ΔD = duration of excess rainfall, h
t/T_p = dimensionless ratio of any time to time to peak
q = discharge rate at time t, ft^3/s
q_p = peak discharge rate at time T_p, ft^3/s
Q_a = runoff volume up to t, in
Q = total runoff volume, in

Source: NRCS, 2007, Part 630 Hydrology, National Engineering Handbook, Chapter 16 Hydrographs, p. 16A-1
https://www.nrcs.usda.gov/wps/portal/nrcs/detailfull/national/water/?&cid=stelprdb1043063

Figure 405-4 Graphical Representation of Relationships Between Lag, Tp and Tc

405.9 HEC-HMS Computation Interval and Duration Guidance

405.9.1 Computation Interval

The computation interval or time step for modeling within HEC-HMS can be specified for a range of intervals as follows:

1, 2, 3, 4, 5, 6, 10, 15, 20, 30 (minutes)

1, 2, 3, 6, 8, 12, 24 (hours)

Selection of the appropriate computation interval can also affect the modeling results. The HEC-HMS Technical Reference Manual (USACE, March 2000) states: "*that for adequate definition of the ordinates on the rising limb of the NRCS Unit Hydrograph, a computational interval, Δt, that is less than 29% of t_{lag} must be used (USACE 1998).*"

Therefore, if basin Lag=0.6 Tc, (Lag is the same as t_{lag}) then the maximum computational interval for use within HEC-HMS to adequately define the rising limb of the hydrograph (and often to capture the peak) is given by:

$$\Delta t < 0.29 \times 0.60\ Tc < 0.17\ Tc \qquad \textbf{405-2}$$

Note that 0.29 x 0.60 = 0.17, therefore this equation reduces to

$\Delta t < 0.17\ Tc$

(USACE, March 2000, "Hydrologic Engineering Center, "HEC-HMS Technical Reference Manual, p. 55)
http://www.hec.usace.army.mil/software/hec-hms/documentation/HEC-HMS_Technical%20Reference%20Manual_(CPD-74B).pdf

The following items are offered as additional guidance for selecting the minimum model computation interval selection:

1. Generally, the computation interval "*Δt*" should be based on the Tc of the smallest subbasin in the model.
2. Note that the shortest rainfall interval available from NOAA is 5 minutes, selecting a shorter computation interval will require HEC-HMS to extrapolate to find a smaller than 5-minute rainfall increment.
3. For 24-hour storm distributions, use a computation interval "*Δt*" of 5 minutes or greater, unless there are other compelling reasons for deviating from 5 minutes.
4. For basins with Tc shorter than 30 minutes, be aware that the computed runoff volume will be accurate but that the model may misstate the peak. Peak rates developed with HEC-HMS for basins with Tc shorter than 30 minutes should always be checked against other methods and experience.
5. Note that shorter and more numerous computation intervals do not always result in better answers (accuracy versus precision).

405.9.2 Duration of Simulation

The model simulation duration (the beginning and ending date and time) should be long enough to capture the entire storm runoff hydrograph. After an initial model run duration of 24 hours, the engineer should review the terminal basin outfall hydrograph to determine if the discharge has returned to zero. If zero discharge is not achieved, extend the model duration and simulate again to obtain zero discharge. Durations greater than 24 hours will generally be required for larger basins (greater than 10 square miles) and for models which contain reservoir routings with long detention times.

405.10 Transmission Losses (Channel Losses)

HEC-HMS has the ability to include the effects of channel losses to the hydrograph. This function is available only in the Modified Puls and Muskingum-Cunge hydrograph routing Methods. Channel losses are included in the "Reach" description within the Basin Model Manager within HEC-HMS. Generally, channel losses do not significantly affect the peak rate of discharge for larger, infrequent flood events, but may have a significant and measurable effect on floods up to the 5-year flood. Therefore, transmission losses should not be considered in the modeling of floods events equal to or greater than the 10-year event. Models constructed for the purpose of evaluating water quality and for determining channel stability and sediment transport will benefit from consideration of transmission losses. If the need to determine the values for use in calculating channel losses on NMDOT projects should arise, use the Percolation Loss/Gain method as outlined in the HEC-HMS User's Manual (p. 234) and the NRCS, 2007, Part 630 Hydrology National Engineering Handbook, Chapter 19, Appendix 19C "Estimating Transmission Losses When No Observed Data are Available".
http://www.hec.usace.army.mil/software/hec-hms/

http://www.wcc.nrcs.usda.gov/ftpref/wntsc/H&H/NEHhydrology/ch19.pdf

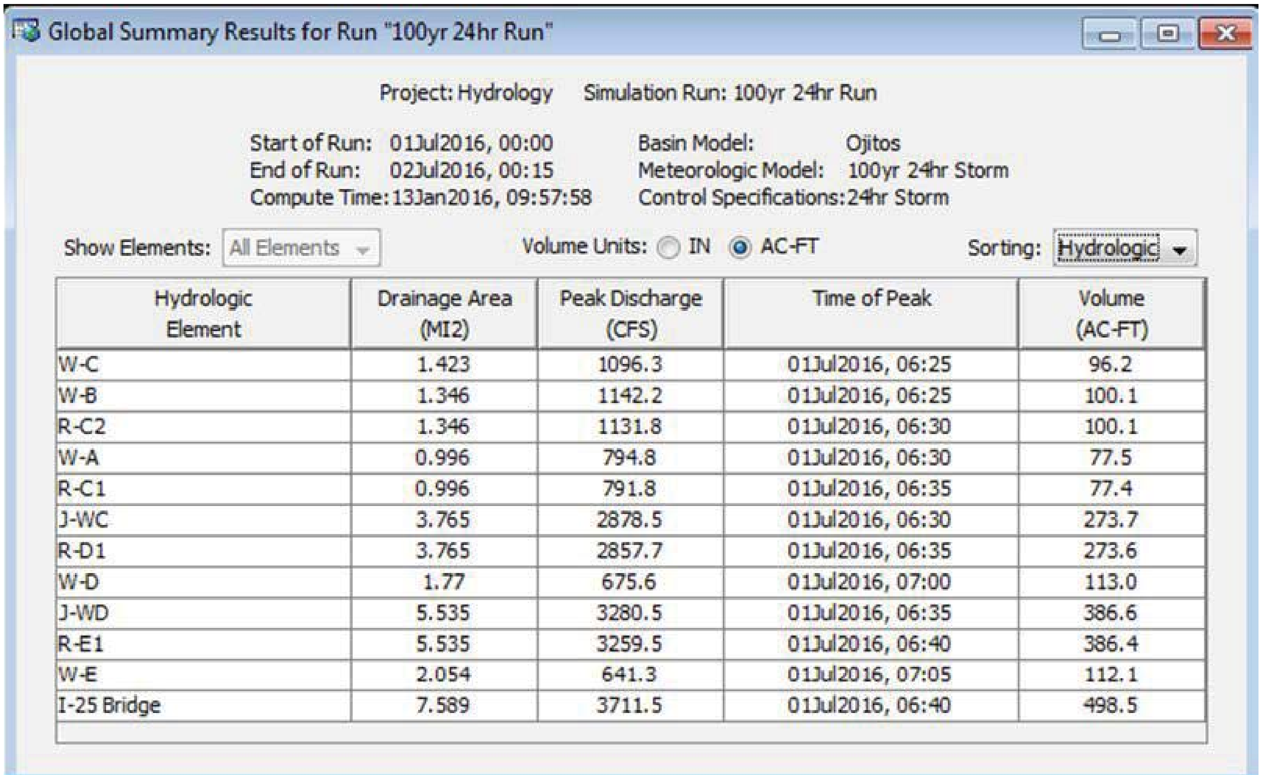
405.11 Flood Routing

HEC-HMS offers a total of six hydrologic routing methods for simulating flow in open channels. For most NMDOT project applications, the Muskingum-Cunge Method is the preferred method. HEC-HMS can also include flood hydrograph routings through diversions, reservoirs, and pump stations.

The Muskingum-Cunge Routing Method is based on the combination of the conservation of momentum and the conservation of mass. This Method relates storage to both inflow and outflow discharges from both the channel and floodplain within each analysis reach. This Method is sometimes referred to as a Variable Coefficient Method because routing parameters are recalculated every time step based on channel properties and the flow depth. The computations attempt to simulate the attenuation of flood waves and can be used in reaches with a mild slope.

405.12 Model Results Reporting

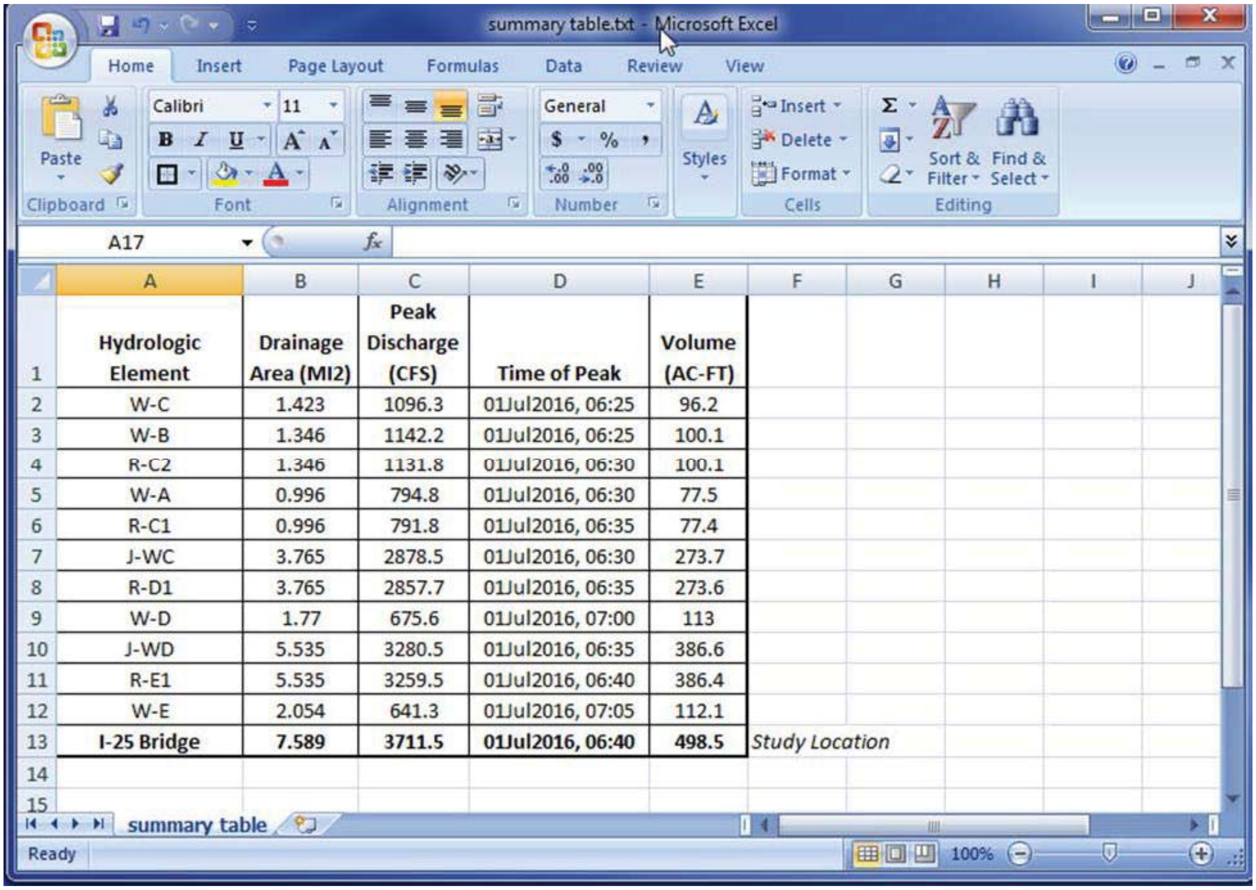
Once the model has been run and the results have been checked for reasonableness, the engineer must include the summary results for each storm frequency simulated in the report. See **Figure 405-5** for the HEC-HMS "Global Summary Table".



Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (AC-FT)
W-C	1.423	1096.3	01Jul2016, 06:25	96.2
W-B	1.346	1142.2	01Jul2016, 06:25	100.1
R-C2	1.346	1131.8	01Jul2016, 06:30	100.1
W-A	0.996	794.8	01Jul2016, 06:30	77.5
R-C1	0.996	791.8	01Jul2016, 06:35	77.4
J-WC	3.765	2878.5	01Jul2016, 06:30	273.7
R-D1	3.765	2857.7	01Jul2016, 06:35	273.6
W-D	1.77	675.6	01Jul2016, 07:00	113.0
J-WD	5.535	3280.5	01Jul2016, 06:35	386.6
R-E1	5.535	3259.5	01Jul2016, 06:40	386.4
W-E	2.054	641.3	01Jul2016, 07:05	112.1
I-25 Bridge	7.589	3711.5	01Jul2016, 06:40	498.5

Figure 405-5 HEC-HMS Global Summary Results Example

Sort the results in the Global Summary Table using “Hydrologic” order, and also select the “Volume Units” to be in ac-ft. Then the HEC-HMS “Global Summary Table” can be exported as a text file to any number of spreadsheet programs for formatting needs as shown in **Figure 405-6**.



Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (AC-FT)	
W-C	1.423	1096.3	01Jul2016, 06:25	96.2	
W-B	1.346	1142.2	01Jul2016, 06:25	100.1	
R-C2	1.346	1131.8	01Jul2016, 06:30	100.1	
W-A	0.996	794.8	01Jul2016, 06:30	77.5	
R-C1	0.996	791.8	01Jul2016, 06:35	77.4	
J-WC	3.765	2878.5	01Jul2016, 06:30	273.7	
R-D1	3.765	2857.7	01Jul2016, 06:35	273.6	
W-D	1.77	675.6	01Jul2016, 07:00	113	
J-WD	5.535	3280.5	01Jul2016, 06:35	386.6	
R-E1	5.535	3259.5	01Jul2016, 06:40	386.4	
W-E	2.054	641.3	01Jul2016, 07:05	112.1	
I-25 Bridge	**7.589**	**3711.5**	**01Jul2016, 06:40**	**498.5**	*Study Location*

Figure 405-6 HEC-HMS Discharge Summary Table Example

In addition, a Basin Model map generated in HEC-HMS (**Figure 405-7**) should be included in the report. This can be created simply by utilizing a screen capture program to copy the screen from HEC-HMS. This Basin Model Map is a schematic that is valuable to assist in understanding the model organization, and the order that basin elements were applied to simulate the basin storm runoff.

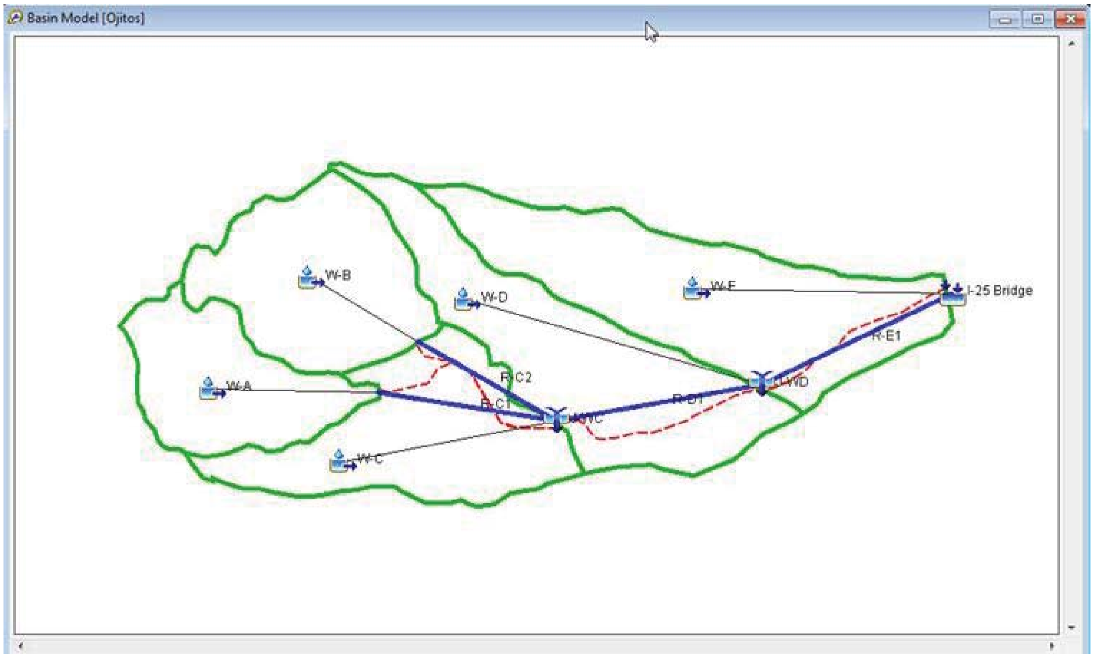


Figure 405-7 HEC-HMS Basin Model Example

The hydrograph shape can be found under the element results **(Figure 405-8).**

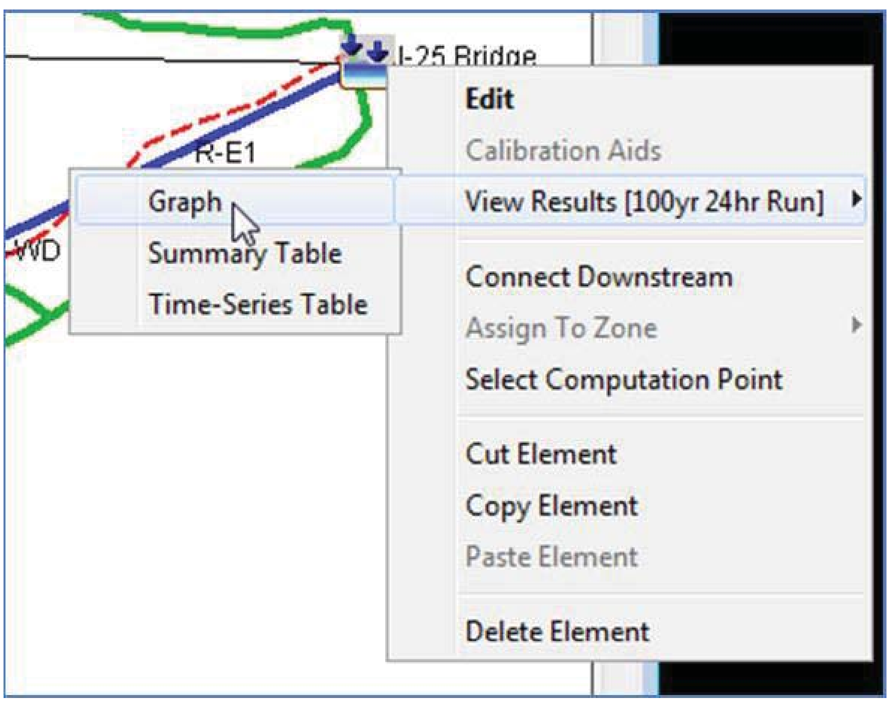


Figure 405-8 HEC-HMS Display Hydrograph Menu Example

The HEC-HMS Method Worksheet (**Figure 405-9**) should be filled out as well.

HEC-HMS Method Worksheet

Structure Location: MP: ______________________ County: ________________
District: ______________________
Structure Description: ______________________________
Drainage Area: **A** = ______________ acres ______________ mi^2

Meteorological Model Summary
Elevation at Centroid of Watershed: **Elev** = __________________ ft *
Location of Centroid: **Lat:** ________________ **Long**: ____________________

Design Frequency Flood	____________ - year	____________ - year
24-hour Rainfall Depth (NOAA PFDS):	**P_{24}** ____________ in.	**P_{24} =** ______________ in.

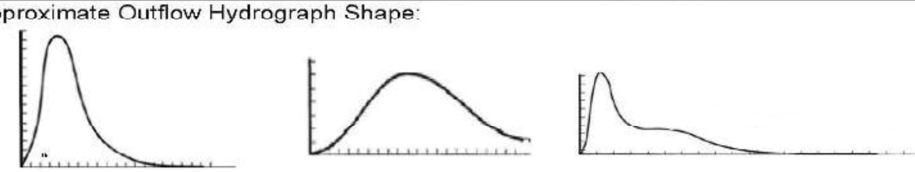
Basin Model Summary
Number of Sub-Basins ________________
Curve Number Range Used for modeling Low: __________ High: ______________
Basin Lag Range Used for modeling Low: __________ min High: ______________ min

Control Specifications Summary
Total Model Duration _________ Hrs:Min Time Interval ________ (min\hrs)

Summary Output (at Structure Location

Design Frequency Flood	____________ - year	____________ - year
Peak Discharge (cfs)	**Q** = ____________ cfs	**Q** = ______________ cfs
Discharge per acre	___________ cfs/ac	______________ cfs/ac
Total Volume (ac-ft)	**V** = __________ ac-ft	**V** = _____________ ac-ft
Total Runoff (in)	**V** = __________ in	V = _______________ in

Approximate Outflow Hydrograph Shape:



Peaky" "Broad" "Mixed"

Project Location: ______________________________
CN#: ______________________________
Date: ______________________________
Computed By: ______________________________
Checked By: ______________________________
* If elevation is greater than 7500 ft, use NRCS Unit Hydrograph method

Figure 405-9 HEC-HMS Method Worksheet

405.13 References

Easterling, Charles, M. 1979, "Urban Watershed Modeling with HYMO", ASCE Irrigation and Drainage Division Specialty Conference".

NOAA Hydrometeorological Design Studies Center Precipitation Frequency Data Server (PFDS).
http://hdsc.nws.noaa.gov/hdsc/pfds/

NRCS Web Soil Survey Website.
https://websoilsurvey.sc.egov.usda.gov/App/HomePage.htm

NRCS, 2004, "Part 630 Hydrology, National Engineering Handbook, Chapter 9 Hydrologic Soil-Cover Complexes".
http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=17758.wba

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http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=17755.wba

NRCS, 2007, "Part 630 Hydrology, National Engineering Handbook, Chapter 19 Transmission Losses".
http://www.wcc.nrcs.usda.gov/ftpref/wntsc/H&H/NEHhydrology/ch19.pdf

NRCS, 2007, "Part 630 Hydrology, National Engineering Handbook, Chapter 19, Appendix 19C, Estimating Transmission Losses When No Observed Data are Available".
http://www.wcc.nrcs.usda.gov/ftpref/wntsc/H&H/NEHhydrology/ch19.pdf

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USACE Hydrologic Engineering Center - HEC-HMS, "HEC-HMS User's Manual", "HEC-HMS Technical Reference Manual", "HEC-HMS Application Guide".
http://www.hec.usace.army.mil/software/hec-hms/

USACE, March 2000, "Hydrologic Engineering Center, "HEC-HMS Technical Reference Manual".
http://www.hec.usace.army.mil/software/hec-hms/documentation/HEC-HMS_Technical%20Reference%20Manual_(CPD-74B).pdf

406 Watersheds with Stream Gage Data

When considering the use of statistical analysis of gage data for design purposes, it is important to determine if the present watershed conditions are represented by the stream gage record or if there has been a significant change in land use. If there has been a significant increase in urbanization or change in agricultural practices, the historical record may not represent current conditions. While many hydrologic techniques are available for the prediction of frequency of flow events, this section presents concepts and techniques for analyzing peak flows using stream gage data and, to a lesser extent, low flows, following the recommendations of

USGS, England, J.F., Jr., Cohn, T.A., Faber, B.A., Stedinger, J.R., Thomas, W.O., Jr., Veilleux, A.G., Kiang, J.E., Mason, R.R., Jr., 2018, "Guidelines for Determining Flood Flow Frequency, Bulletin #17C, Chapter 5 of Section B, Surface Water, Book 4, Hydrologic Analysis and Interpretation, Techniques and Methods 4-B5".
https://pubs.er.usgs.gov/publication/tm4B5

Elements of risk and uncertainty are inherent in any flood frequency analysis. It is possible to standardize many elements of flood frequency analysis, but reliable results are only possible where available records are adequate to warrant statistical analysis of the data.

Flow frequency analysis relates the magnitude of a given flow event with the frequency or probability of that event's exceedance. If a stream gage is available and the conditions applicable, a gage analysis is generally considered preferable to deterministic methods (Rational Formula Method, Simplified Peak Discharge Method or NRCS Unit Hydrograph Method within HEC-HMS). Since a gage represents the actual rainfall-runoff behavior of the watershed in relation to the stream. A variety of Federal, state, and local agencies operate and maintain stream gages. Currently, the USGS operates about 7,000 active stream gaging stations across the country. Data are also available for about 13,000 discontinued gaging stations. Data is available for 155 currently active sites in New Mexico and for a total of 495 sites when the discontinued sites are included.

The USGS has determined station specific flood frequency data for 293 gage locations for recurrence intervals of 2-, 5-, 10-, 25-, 50-, 100- and 500-years that generally have 10 or more years of record (through 2004). Historical peak flow data for both active and discontinued gages can be found at the following USGS website at:
http://nwis.waterdata.usgs.gov/usa/nwis/peak.

This information is also found in Appendix 1 of the USGS report prepared for New Mexico in cooperation with the NMDOT: "Analysis of the Magnitude and Frequency of Peak Discharge and Maximum Observed Peak Discharge in New Mexico and Surrounding Areas", Scientific Investigations Report 2008-5119, USGS, Waltemeyer, Scott, D., 2008.
http://pubs.usgs.gov/sir/2008/5119/

The USGS has also developed a web-based flood-frequency analysis tool called "PeakFQ-Flood-Frequency Analysis", for determining the stream flood statistics at gaging stations with sufficiently long records. This program is available at:
https://water.usgs.gov/software/PeakFQ/

Streamflow data from gages other than USGS gages should not be used for design of NMDOT projects (unless approved by the NMDOT), but may be useful for checking against peak discharge estimates derived from other methods and sources. There are several general scenarios in which data from a non-USGS streamflow gage may be utilized:

1. The gage has been in place for a sufficient number of years (Bulletin 17C recommends at least 10 years)
2. The gage data is reasonably representative of the average watershed conditions during the period of record
3. The gage is located at the highway drainage structure
4. The gage is located upstream or downstream at some distance from the highway

The majority of the gage data in New Mexico has been collected by the USGS. For most of their active streamflow gage sites and many of their inactive sites, the USGS has computed flood frequency estimates. These estimates can be used directly for design if the gage is located at or near (as defined below) the highway crossing. The current USGS study of peak stream flows in New Mexico (USGS, Waltemeyer, Scott, D., 2008) includes tabulated flood frequency estimates for most USGS gage sites in New Mexico.

If the gage data set represents a relatively short period of record, a correction weighting procedure is recommended. The gage frequency distribution peak flood estimate is weighted according to the length of record and equivalent years from the USGS regression analysis. Waltemeyer (USGS, 1996) describes a procedure for improving flood frequency estimates at gaged sites, using USGS regression equations. In the event that the USGS gage at the highway drainage structure was not included in Waltemeyer's study, then a frequency distribution analysis is necessary. A comprehensive discussion of frequency analysis is beyond the scope of this manual. There are several publications which describe the process in great detail. References for two such publications are provided below:

USGS, England et al., 2018, "Guidelines for Determining Flood Flow Frequency, Bulletin #17C, Chapter 5 of Section B, Surface Water, Book 4, Hydrologic Analysis and Interpretation, Techniques and Methods 4-B5".
https://pubs.er.usgs.gov/publication/tm4B5

U.S. Army Corps of Engineers, 1993, "Engineering and Design, Hydrologic Frequency Analysis".
http://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-1415.pdf

Typically, a Log-Pearson Type III probability distribution is fit to the set of streamflow data. The use of a partial duration series may be appropriate rather than an annual series depending on data availability and quality.

When the USGS streamflow gage is located on the same stream but some distance upstream or downstream of the highway, the gage site can still be used to provide a weighted flood frequency estimate. The area weighted correction procedure (USGS, Waltemeyer, Scott, D., 1996) includes a drainage area ratio adjustment which can be used when the ratio of ungaged watershed area to gaged watershed area is within the limits 0.5 to 1.5. The following excerpt from Waltemeyer explains that process.

406.1 Ungaged Site on a Stream Having a Nearby Gaging Station

This information in this section was obtained from "Analysis of the Magnitude and Frequency of Peak Discharge and Maximum Observed Peak Discharge in New Mexico and Surrounding Areas", Scientific Investigations Report 2008-5119, USGS, Waltemeyer, Scott, D., 2008.
http://pubs.usgs.gov/sir/2008/5119/

Flood-frequency estimates can be made for ungaged sites upstream or downstream from gaging stations by using a method developed by Sauer (1974). Using this method, flood-frequency data at the gaging station is transferred to the ungaged site by using the following drainage-area ratio adjustment equation:

$$Q_{T(u)} = Q_{T(g)}\,(A_u / A_g)^{x} \qquad \textbf{406-1}$$

(USGS, Waltemeyer, Scott, D., 2008, "Analysis of the Magnitude and Frequency of Peak Discharge and Maximum Observed Peak Discharge in New Mexico and Surrounding Areas", Scientific Investigations Report 2008-5119, Eq. 3, p.11)

where:

$Q_{T(u)}$	=	weighted flood-frequency estimate at the ungaged site, ft^3/s
$Q_{T(g)}$	=	flood-frequency estimate at the gaging station, ft^3/s
A_u	=	drainage area at the ungaged site, square miles
A_g	=	drainage area at the gaging station, square miles
x	=	exponent of the drainage area of the applicable regional regression equation is listed in Table 2 found on pages 9 and 10 of the USGS document "Analysis of the Magnitude and Frequency of Peak Discharge and Maximum Observed Peak Discharge in New Mexico and Surrounding Areas", by Scott D. Waltemeyer 2008

According to Sauer (1974), the equation is applicable when the drainage-area ratio (A_u/A_g), is between 0.5 and 1.5. For example, to estimate a 50-year peak discharge at an ungaged site in Region 2 upstream from gaging station Cisco Wash near Cisco, Utah (09163700), the station value listed in Appendix 1 is 4,670 ft^3/s. Note that the weighted value of 5,500 ft^3/s was not used because when using this technique, a regional adjustment is made by using the exponent from the regional equation. The weighted value is considered the best flood-frequency value, but when using this technique, a double weight would be made based on the regional flood information. The drainage area at the gaging station is 90.7 square miles (Appendix 1, USGS, 2008). The 50-year recurrence interval regression equation exponent for the drainage area is 0.308 for Region 2 (Table 2, USGS, 2008). The drainage area at the ungaged site is 75.5 square miles, and when equation 4 (USGS, 2008) is used (equation below), the peak discharge at the ungaged site is:

$$Q_{50u} = Q_{50g}\left(A_u / A_g\right)^{x}$$

$$Q_{50u} = (4{,}670)\,(75.5 / 90.7)^{0.308} = 4{,}410\ ft^3/s \qquad \textbf{406-2}$$

(USGS, Waltemeyer, Scott, D., 2008, "Analysis of the Magnitude and Frequency of Peak Discharge and Maximum Observed Peak Discharge in New Mexico and Surrounding Areas", Scientific Investigations Report 2008-5119, Eq. 3, p.12)
http://pubs.usgs.gov/sir/2008/5119/

Note: The USGS has developed a web application called "StreamStats". StreamStats incorporates a Geographic Information System (GIS) to provide users with access to an assortment of analytical tools that are useful for a variety of water resources planning and management purposes, and for engineering and design purposes.
https://water.usgs.gov/osw/streamstats/

406.2 References

Sauer, V.B., 1974, “Flood Characteristics of Oklahoma Streams: U.S. Geological Survey Water-Resources Investigations Report 52-73”.

U.S. Army Corps of Engineers, March 1993, “EM 1110-2- 1415 Hydrologic Frequency Analysis”. http://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-1415.pdf

USGS, Website, "PeakFQ-Flood-Frequency Analysis". https://water.usgs.gov/software/PeakFQ/

USGS, Website, “StreamStats”. https://water.usgs.gov/osw/streamstats/

USGS, Website, “Stream Flow Gage Data, Active and Discontinued Gages” http://nwis.waterdata.usgs.gov/usa/nwis/peak.

USGS, Waltemeyer, Scott, D., 1996, “Analysis of the Magnitude and Frequency of Peak Discharge and Maximum Observed Peak Discharge in New Mexico”, Water-Resources Investigations Report 96-4112. http://pubs.er.usgs.gov/publication/wri964112

USGS, England, J.F., Jr., Cohn, T.A., Faber, B.A., Stedinger, J.R., Thomas, W.O., Jr., Veilleux, A.G., Kiang, J.E., Mason, R.R., Jr., 2018, “Guidelines for Determining Flood Flow Frequency, Bulletin #17C, Chapter 5 of Section B, Surface Water, Book 4, Hydrologic Analysis and Interpretation, Techniques and Methods 4-B5”. https://pubs.er.usgs.gov/publication/tm4B5

USGS, Waltemeyer, Scott, D., 2008, “Analysis of the Magnitude and Frequency of Peak Discharge and Maximum Observed Peak Discharge in New Mexico and Surrounding Areas”, Scientific Investigations Report 2008-5119. http://pubs.usgs.gov/sir/2008/5119/

407 Statistical Methods in Watersheds without Stream Gage Data

The USGS’s (Waltemeyer, 2008) report titled “Analysis of the Magnitude and Frequency of Peak Discharge and Maximum Observed Peak Discharge in New Mexico and Surrounding Areas”, Scientific Investigations Report 2008-5119, was prepared in cooperation with the NMDOT. The report summarized the analyses and equations developed for estimating peak discharges for recurrence intervals of 2-, 5-,10-, 25-, 50-, 100- and 500-years at ungaged sites by use of data collected through 2004 for 293 gaging stations on unregulated streams that have 10 or more years of record.

The regional flood frequency equation values shown in Table 2 of the above-referenced report list the “Average Standard Error of Estimates” for each of the nine hydrologic regions and for recurrence intervals of 2-, 5-, 10-, 25-, 50-, 100- and 500-years. Flood magnitude estimates from the USGS are based on information collected from stream gage data as well as from estimates of flood magnitude using high water marks and eyewitness accounts when gages were damaged or destroyed by the flood. Many records are relatively short compared to the exceedance frequency projected by the statistics. There are also inherent accuracy problems with some of the data collected by means other than from a properly functioning gage. Hence the estimates produced may differ from those that would have been produced if the records were long and accurate.

It is important to consider the Standard Error when using USGS regression estimates as it affects the accuracy of the estimates and, therefore, the reliance that can be placed on the interpretations drawn from the data.

The USGS states in the above-referenced report: The average Standard Error of prediction, which includes average sampling error and average Standard Error of regression, ranged from 38 to 93 percent (mean value is 62, and median value is 59) for the 100-year flood. The 1996 investigation Standard Error of prediction for the flood regions ranged from 41 to 96 percent (mean value is 67, and median value is 68) for the 100-year flood that was analyzed by using generalized least-squares regression analysis. Overall, the equations based on generalized least-squares regression techniques are more reliable than those in the 1996 report because of the increased length of record and improved geographic information system (GIS) method to determine basin and climatic characteristics.

The Standard Error measure indicates the extent to which a regression estimate is likely to deviate from the true population and is expressed as a number. The Relative Standard Error (RSE) is the Standard Error expressed as a fraction of the estimate and is usually displayed as a percentage. Estimates with a RSE of 25% or greater are subject to high sampling and regression error and should be used with caution.

The average Standard Error of estimates listed in Table 2 of the above referenced USGS report all exceed 25% (with some exceeding 100%). Therefore, the use of the USGS regional regression equations for New Mexico should be limited to:

1. Determination that the peak discharges calculated using one of the three approved hydrologic peak discharge analyses methods are within reason and supported by the exercise of judgment, and
2. For very preliminary peak discharge estimation when scoping a project
3. USGS regional regression equations may be used for design when checked against one of the hydrologic peak discharge analysis methods and approved by the NMDOT Drainage Engineer

The tabulation of maximum observed peak discharges for sites within each of the nine hydrologic regions around New Mexico are listed in Appendix 3 of the Waltemeyer 2008 report. The engineer is encouraged to review that Appendix when performing drainage analyses to gain further understanding of the hydrologic response of the various regions around the state. An excerpt from Appendix 3 is shown below (**Figure 407-1**) for reference.

Appendix 3. Miscellaneous sites, maximum peak discharge, drainage area, and site elevations.

[DMS, degrees minutes seconds; NGVD 29, National Geodetic Vertical Datum of 1929]

Map identi-fier (fig. 2)	Latitude (DMS)	Longi-tude (DMS)	Miscellaneous site name	Date	Maximum peak discharge (cubic feet per second)	Drainage area (square miles)	Site elevation (feet above NGVD 29)	Region
1	362958	1030731	Apache Creek at State Highway 18 near Clayton, New Mexico	1952	2,500	50.8	4,780	1
2	354906	1034621	Arroyo del Alamo near Mosquero, New Mexico	05/16/54	3,440	27.4	4,380	1
3	350110	1040426	Arroyo Laguana tributary near Montoya, New Mexico	07/05/60	2,660	3.40	4,460	1
4	350216	1033407	Barranca Creek near Norton, New Mexico	08/23/59	3,870	147	4,040	1
5	350714	1035416	Blanco Creek tributary at Palomas, New Mexico	07/05/60	1,540	2.90	4,330	1
6	365451	1030515	Bontz Arroyo near Guy, New Mexico	07/06/58	2,410	4.90	4,450	1
7	364546	1042933	Canadian River tributary near Hebron, New Mexico	06/17/65	2,130	2.01	6,300	1
8	364818	1030100	Carrizozo Creek tributary near Moses, New Mexico	07/06/58	307	0.15	4,730	1
9	352646	1034249	Carros Creek near Gallegos, New Mexico	07/08/60	2,590	9.40	4,050	1
10	361550	1031235	Carrizo Creek near Clayton, New Mexico	05/28/57	29,500	305	4,780	1
11	365210	1042246	Chicorica Creek near Raton, New Mexico	06/17/65	12,800	78.8	6,460	1
12	365100	1042125	Chicorica Creek tributary near Raton, New Mexico	06/17/65	1,810	1.33	6,470	1
13	362122	1032812	Carrizo Creek above State Road 56 near Clayton, New Mexico	06/17/65	9,270	305	5,245	1
14	353616	1045352	Conchas River tributary near Trujillo, New Mexico	07/05/67	4,530	3.43	6,480	1
15	353622	1045208	Conchas River near Trujillo, New Mexico	07/05/67	9,810	18.6	6,430	1
16	364216	1043536	Crow Creek below Waldron Canyon near Koehler, New Mexico	06/17/65	30,400	59.8	6,260	1
17	363755	1043225	Crow Creek near Maxwell, New Mexico	06/17/65	13,100	78.4	6,030	1
18	351813	1041843	Cuervo Creek near Conchas Dam, New Mexico	06/18/69	12,800	135	4,280	1
19	360020	1033331	Del Muerto Creek near Bueyeros, New Mexico	07/16/72	34,600	29.0	4,690	1
20	365540	1042130	East Fork Chicorica Creek at Yankee, New Mexico	06/17/65	13,500	22.7	6,780	1
21	351256	1031216	Frost Creek near Porter, New Mexico	07/16/58	2,910	10.0	3,900	1
22	354430	1040722	La Cinta Creek near Roy, New Mexico	08/17/77	13,300	116.5	4,700	1
23	361249	1043900	Ocate Creek at Colmar, New Mexico	07/04/51	25,000	434	5,900	1
24	354113	1030746	Minneosa Creek near Nara Visa, New Mexico	07/23/54	20,400	118	4,300	1
25	345838	1034502	Paris Creek near Quay, New Mexico	07/17/56	3,410	9.20	4,220	1

Source: USGS, Waltemeyer, Scott, D., 2008, "Analysis of the Magnitude and Frequency of Peak Discharge and Maximum Observed Peak Discharge in New Mexico and Surrounding Areas", Appendix 3, p. 91.
http://pubs.usgs.gov/sir/2008/5119/

Figure 407-1 USGS Appendix 3 Excerpt

407.1 References

USGS, Waltemeyer, Scott, D., 2008, "Analysis of the Magnitude and Frequency of Peak Discharge and Maximum Observed Peak Discharge in New Mexico and Surrounding Areas", Scientific Investigations Report 2008-5119.
http://pubs.usgs.gov/sir/2008/5119/

408 Risk and Uncertainty in Hydrologic Analysis

Highway drainage structures are designed to safely pass a certain magnitude flood. On most New Mexico highways, the Design Flood will be the "50-year" frequency flood. This flood is theoretically equivalent to the largest flood which will occur at that location on average at least once every fifty years. By designing drainage structures to safely pass relatively rare events, the risk to users of the highway is reduced to an acceptable level. There is always some chance, or risk, that a flood will occur which exceeds the design flood used to size a particular drainage structure. While it might be desirable to design all drainage structures to pass the largest possible flood, economic realities prevent this option. Instead, a level of protection must be provided which is both responsible and reasonable.

Design exceptions or variances may be required as a result of budget impacts, right-of-way limitations, environmental and property impacts, or other constraints. Such variances are only allowed when all other options have been considered and found to be inadequate. If deviation from the criteria and design standards for major drainage structures or systems is necessary, a risk assessment may be required. If a jurisdiction or organization has more stringent criteria than the NMDOT criteria, those criteria shall govern the drainage design. Even though the 50-year flood occurs on average at least once every 50 years, there is some small, but very real possibility (2% chance) that this flood could occur in any given year. Stated another way, just because a 50-year flood occurred last year, does not mean that it could not occur again this year. The probability of a 50-year flood occurring or being exceeded this year and every year is remains at 2%.

In order to better quantify the risk associated with a certain design frequency the following example is provided:

Consider a drainage structure capable of passing the 100-year frequency event with a structural design life of 50-years. What is the probability or risk, that the structure will see a 100-year flood (or greater) during its design life? The logical answer might be 1 chance in 2, or 50%. However statistical analyses show that the risk is lower, actually at 39.5%. Statistically, the concept of risk is described by a binomial distribution

USGS, England et al., 2018, “Guidelines for Determining Flood Flow Frequency, Bulletin #17C, Chapter 5 of Section B, Surface Water, Book 4, Hydrologic Analysis and Interpretation, Techniques and Methods 4-B5”.
https://pubs.er.usgs.gov/publication/tm4B5

Equation 408-1 describes this statistical relationship.

$$R = 1 - \left(1 - \frac{1}{T_r}\right)^m \times 100 \qquad \textbf{408-1}$$

where:

R	=	the risk of design discharge being exceeded at least once during the design life, percent
T_r	=	the recurrence interval or frequency of the design flood, years
m	=	the design life of the structure, years

$R = 1 - \left(1 - \frac{1}{100}\right)^{50} \times 100 = 39.5\%$ for the example above.

Assuming that the structure is designed for the 50-year flood and has a design life of 50 years, then **Equation 408-1** predicts that the structure's capacity has a 63.6% chance of being equaled or exceeded during the structure's design life.

$$R = 1 - \left(1 - \frac{1}{50}\right)^{50} \times 100 = 63.6\%$$

Table 408-1 lists computed values of risk for a range of structure design lives.

Table 408-1 Tabulation of Risk of at Least One Exceedance during the Design Life

	Design Life - Years					
Recurrence Interval	**2**	**5**	**10**	**25**	**50**	**100**
2	75.0%	97.0%	100.0%	100.0%	100.0%	100.0%
5	36.0%	67.0%	89.0%	100.0%	100.0%	100.0%
10	19.0%	41.0%	65.0%	93.0%	99.0%	100.0%
25	8.0%	18.0%	34.0%	64.0%	87.0%	98.0%
50	4.0%	10.0%	18.0%	40.0%	64.0%	87.0%
100	2.0%	5.0%	10.0%	22.0%	39.0%	63.0%
500	0.4%	1.0%	2.0%	5.0%	10.0%	18.0%
1000	0.2%	0.5%	1.0%	2.0%	5.0%	10.0%

Another way of looking at the concept of risk is to define an acceptable level of risk and then compute the design flood which would have to be accommodated by the drainage structure to satisfy that level of risk. **Equation 408-1** can be rearranged to solve for the required return period, yielding **Equation 408-2**.

$$Tr = \frac{1}{1 - \left(1 - \frac{R}{100}\right)^{1/m}} \qquad \textbf{408-2}$$

Assume that a 10% level of risk is desirable, or stated another way, there is a 90% confidence level that the structure is adequate. Then **Equation 408-2** predicts that the structure with the design life of 50 years must be capable of passing the 475-year flood.

$$Tr = \frac{1}{1 - \left(1 - \frac{10}{100}\right)^{\frac{1}{50}}} = 475 \text{ years}$$

It becomes apparent that risk cannot be completely eliminated, but may be reduced to a level acceptable to society. Even if there were unlimited funds to build drainage structures, the ability to accurately calculate the magnitude of flood events decreases as the design flood magnitude increases. All of the current flood prediction methods, whether analytical or parametric, are based on observed flood flows from watersheds with measured response characteristics, and occasionally rain gage data. The effective period of recorded data in New Mexico reaches 100 years in only a few locations. Thus, the prediction of a 475-year flood is done by extrapolating the data, since the desired flood has only a small chance of being included in the data set. The uncertainty in predicted flood flows increases as the return period lengthens.

The accuracy of predicted flood magnitudes up to the 100-year event is, while not perfect, certainly much better. For the analytic methods presented in this manual, risk takes the form of uncertainty in the input parameters. A drainage area can be measured by multiple engineers and the answers from each, should all be within two or three percent. Use of a consistent method to compute Tc reduces variability in the estimation of Tc. However, the selection of a Rational Formula Method Runoff Coefficient "C", or a NRCS Runoff Curve Number “CN” involves considerable judgement. Even meticulous measurement of watershed areas, land uses, and Hydrologic Soil Groups may not accurately describe the response of the watershed for every storm. There is some inherent variability of the data, and of its interpretation, leading to uncertainty in the selection of the correct “C” or “CN”. This uncertainty cannot be universally quantified, and thus becomes part of the overall risk and uncertainty in predicting peak flood magnitudes.

With the analytic methods in this manual, one approach to qualitatively assess the risk is to perform a sensitivity analysis. This is done by varying a particular input parameter across its range of reasonable values and comparing the resulting range of predicted peak flows. The most sensitive analytic parameter in larger watersheds will probably be the “C” or “CN”. Use the “C” or “CN” value obtained by normal design methods to compute a peak flow, as well as the lowest and highest “C” or “CN” values which could occur in the watershed. (Note: In small watersheds, Tc can be the most sensitive input value, but the process is the same.)

The resulting three computed peak flow values provide an estimate of the range of most probable peak flood flows. This is not a precise computed range of risk, but it does help to bracket the most likely peak flow value. The middle peak flood flow value will often be used to size the structure, while the upper limit peak flood flow can be used to assess the "worst case" headwater or overtopping condition. If the risk and consequences of an overtopping or significant backwater are unacceptably adverse to the roadway or nearby property, consider an alternate design.

408.1 Reference

USGS, England, J.F., Jr., Cohn, T.A., Faber, B.A., Stedinger, J.R., Thomas, W.O. Jr., Veilleux, A.G., Kiang, J.E., Mason, R.R., Jr., 2018, “Guidelines for Determining Flood Flow Frequency, Bulletin #17C, Chapter 5 of Section B, Surface Water, Book 4, Hydrologic Analysis and Interpretation, Techniques and Methods 4-B5”.
https://pubs.er.usgs.gov/publication/tm4B5
https://water.usgs.gov/osw/bulletin17b/dl_flow.pdf

ATTACHMENT III.3.B

U.S. DEPT. OF COMMERCE NATIONAL OCEANIC AND ATMOSPHERIC ADMINISTRATION NATIONAL WEATHER SERVICE OFFICE OF HYDROLOGIC DEVELOPMENT HYDROMETEOROLOGICAL DESIGN STUDIES CENTER, NOAA ATLAS 14, VOLUME 1, VERSION 5 , POINT PRECIPITATION FREQUENCY ESTIMATES FOR LATITUDE: 32.5297°, LONGITUDE: -103.7884°, PDS-BASED POINT PRECIPITATION FREQUENCY ESTIMATES WITH 90% CONFIDENCE INTERVALS (IN INCHES)

NOAA Atlas 14, Volume 1, Version 5
Location name: Lovington, New Mexico, USA*
Latitude: 32.5297°, Longitude: -103.7884°
Elevation: 3522.46 ft**
* source: ESRI Maps
** source: USGS

POINT PRECIPITATION FREQUENCY ESTIMATES

Sanja Perica, Sarah Dietz, Sarah Heim, Lillian Hiner, Kazungu Maitaria, Deborah Martin, Sandra Pavlovic, Ishani Roy, Carl Trypaluk, Dale Unruh, Fenglin Yan, Michael Yekta, Tan Zhao, Geoffrey Bonnin, Daniel Brewer, Li-Chuan Chen, Tye Parzybok, John Yarchoan

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	**0.317** (0.280-0.361)	**0.410** (0.361-0.467)	**0.546** (0.480-0.621)	**0.652** (0.571-0.739)	**0.794** (0.691-0.900)	**0.905** (0.784-1.03)	**1.02** (0.880-1.16)	**1.14** (0.978-1.29)	**1.31** (1.11-1.48)	**1.44** (1.21-1.63)
10-min	**0.483** (0.425-0.550)	**0.624** (0.550-0.711)	**0.831** (0.730-0.944)	**0.992** (0.868-1.13)	**1.21** (1.05-1.37)	**1.38** (1.19-1.56)	**1.56** (1.34-1.76)	**1.74** (1.49-1.97)	**1.99** (1.69-2.25)	**2.19** (1.84-2.48)
15-min	**0.598** (0.527-0.682)	**0.773** (0.682-0.881)	**1.03** (0.905-1.17)	**1.23** (1.08-1.40)	**1.50** (1.30-1.70)	**1.71** (1.48-1.94)	**1.93** (1.66-2.18)	**2.16** (1.85-2.44)	**2.47** (2.09-2.79)	**2.71** (2.28-3.07)
30-min	**0.806** (0.710-0.918)	**1.04** (0.918-1.19)	**1.39** (1.22-1.58)	**1.66** (1.45-1.88)	**2.02** (1.76-2.29)	**2.30** (1.99-2.61)	**2.60** (2.24-2.94)	**2.90** (2.48-3.28)	**3.32** (2.81-3.76)	**3.65** (3.07-4.13)
60-min	**0.997** (0.879-1.14)	**1.29** (1.14-1.47)	**1.72** (1.51-1.95)	**2.05** (1.79-2.32)	**2.50** (2.17-2.83)	**2.85** (2.46-3.23)	**3.22** (2.77-3.64)	**3.59** (3.08-4.07)	**4.11** (3.48-4.65)	**4.52** (3.80-5.12)
2-hr	**1.14** (0.998-1.31)	**1.47** (1.29-1.69)	**1.99** (1.74-2.28)	**2.39** (2.09-2.73)	**2.96** (2.56-3.36)	**3.40** (2.93-3.86)	**3.87** (3.31-4.39)	**4.37** (3.70-4.95)	**5.05** (4.23-5.73)	**5.60** (4.65-6.36)
3-hr	**1.21** (1.06-1.38)	**1.57** (1.38-1.79)	**2.11** (1.84-2.39)	**2.53** (2.21-2.87)	**3.13** (2.72-3.54)	**3.60** (3.11-4.06)	**4.11** (3.52-4.63)	**4.64** (3.94-5.23)	**5.38** (4.52-6.08)	**5.99** (4.97-6.77)
6-hr	**1.40** (1.24-1.59)	**1.80** (1.59-2.05)	**2.40** (2.11-2.71)	**2.87** (2.53-3.25)	**3.55** (3.10-4.00)	**4.09** (3.55-4.60)	**4.67** (4.02-5.24)	**5.28** (4.51-5.93)	**6.14** (5.18-6.90)	**6.84** (5.71-7.70)
12-hr	**1.57** (1.38-1.78)	**2.01** (1.77-2.28)	**2.66** (2.33-3.02)	**3.18** (2.78-3.61)	**3.92** (3.40-4.43)	**4.51** (3.89-5.09)	**5.15** (4.41-5.80)	**5.82** (4.93-6.55)	**6.76** (5.67-7.62)	**7.52** (6.25-8.50)
24-hr	**1.74** (1.57-1.94)	**2.24** (2.02-2.50)	**3.00** (2.70-3.34)	**3.61** (3.25-4.01)	**4.48** (4.00-4.97)	**5.18** (4.59-5.74)	**5.93** (5.22-6.56)	**6.73** (5.86-7.45)	**7.85** (6.76-8.72)	**8.77** (7.47-9.78)
2-day	**1.91** (1.71-2.14)	**2.46** (2.20-2.77)	**3.32** (2.96-3.72)	**4.02** (3.57-4.50)	**5.03** (4.43-5.61)	**5.86** (5.13-6.54)	**6.76** (5.87-7.54)	**7.73** (6.64-8.65)	**9.14** (7.73-10.2)	**10.3** (8.60-11.6)
3-day	**2.03** (1.81-2.28)	**2.62** (2.33-2.95)	**3.54** (3.14-3.98)	**4.30** (3.80-4.82)	**5.39** (4.73-6.03)	**6.29** (5.48-7.04)	**7.27** (6.29-8.14)	**8.33** (7.13-9.35)	**9.87** (8.32-11.1)	**11.1** (9.27-12.6)
4-day	**2.15** (1.91-2.42)	**2.77** (2.46-3.13)	**3.76** (3.33-4.24)	**4.57** (4.03-5.14)	**5.75** (5.03-6.45)	**6.72** (5.84-7.54)	**7.78** (6.71-8.73)	**8.93** (7.62-10.1)	**10.6** (8.90-12.0)	**12.0** (9.94-13.6)
7-day	**2.42** (2.15-2.72)	**3.12** (2.77-3.52)	**4.23** (3.74-4.76)	**5.12** (4.52-5.76)	**6.40** (5.60-7.18)	**7.45** (6.47-8.36)	**8.58** (7.40-9.65)	**9.79** (8.37-11.0)	**11.5** (9.73-13.1)	**13.0** (10.8-14.8)
10-day	**2.68** (2.39-3.02)	**3.46** (3.09-3.90)	**4.68** (4.16-5.27)	**5.67** (5.02-6.37)	**7.09** (6.23-7.94)	**8.25** (7.20-9.23)	**9.50** (8.23-10.6)	**10.8** (9.30-12.2)	**12.8** (10.8-14.4)	**14.4** (12.0-16.3)
20-day	**3.40** (3.05-3.78)	**4.36** (3.91-4.85)	**5.75** (5.15-6.39)	**6.83** (6.10-7.58)	**8.32** (7.39-9.23)	**9.49** (8.39-10.5)	**10.7** (9.40-11.9)	**12.0** (10.4-13.3)	**13.7** (11.8-15.3)	**15.1** (12.9-17.0)
30-day	**3.99** (3.59-4.42)	**5.10** (4.58-5.66)	**6.64** (5.96-7.36)	**7.82** (7.00-8.66)	**9.41** (8.38-10.4)	**10.6** (9.44-11.8)	**11.9** (10.5-13.2)	**13.2** (11.6-14.6)	**14.9** (12.9-16.6)	**16.3** (14.0-18.2)
45-day	**4.68** (4.20-5.20)	**5.99** (5.38-6.65)	**7.79** (6.98-8.65)	**9.16** (8.19-10.2)	**11.0** (9.81-12.2)	**12.4** (11.0-13.8)	**13.9** (12.2-15.4)	**15.4** (13.4-17.1)	**17.4** (15.0-19.4)	**18.9** (16.2-21.3)
60-day	**5.37** (4.85-5.92)	**6.86** (6.19-7.56)	**8.82** (7.95-9.73)	**10.3** (9.25-11.3)	**12.2** (10.9-13.4)	**13.6** (12.2-15.0)	**15.0** (13.4-16.6)	**16.5** (14.6-18.2)	**18.3** (16.1-20.4)	**19.7** (17.1-22.0)

[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.

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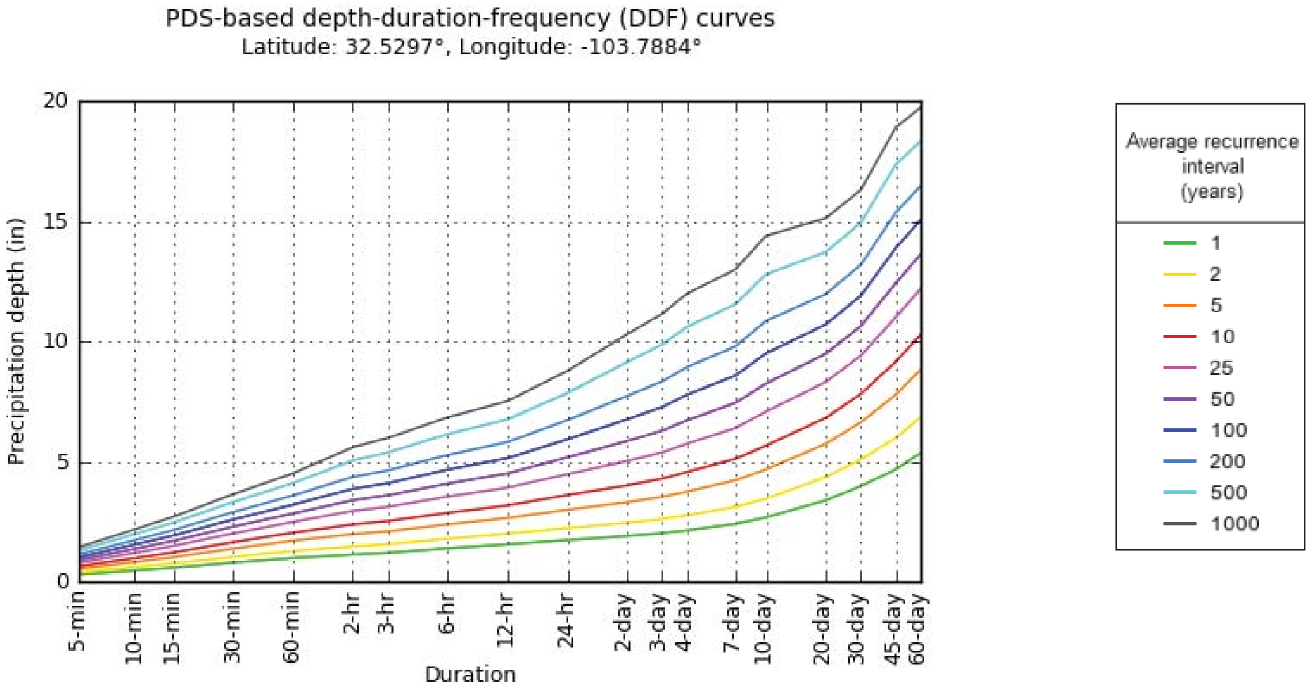
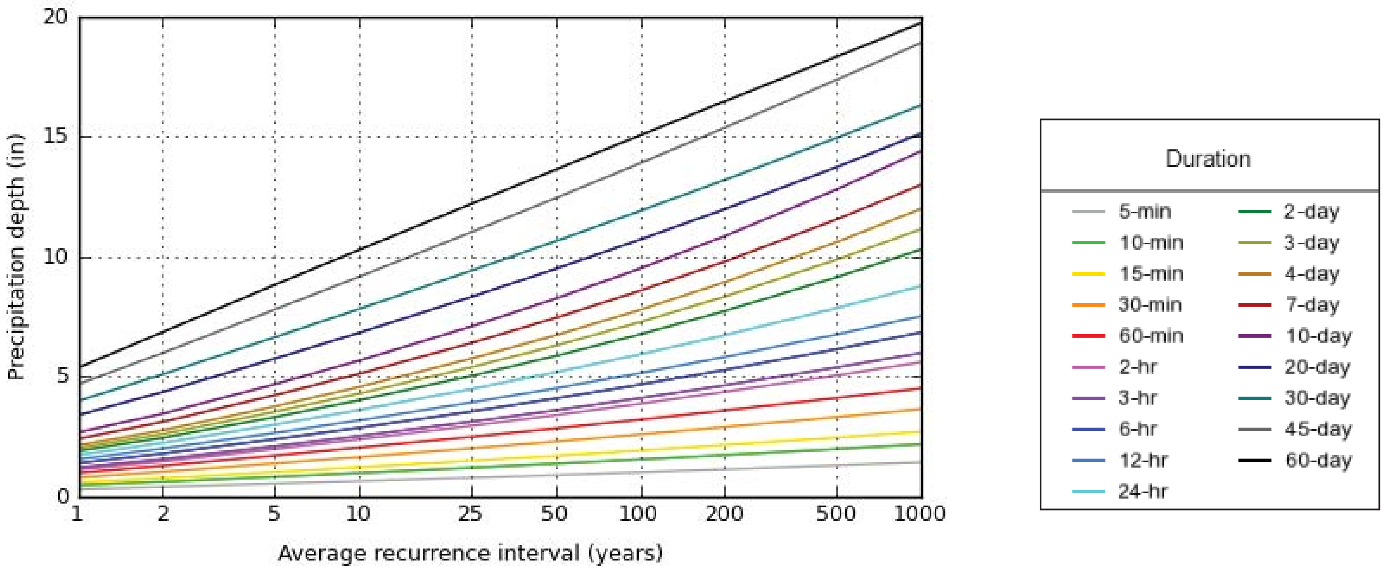
PF graphical





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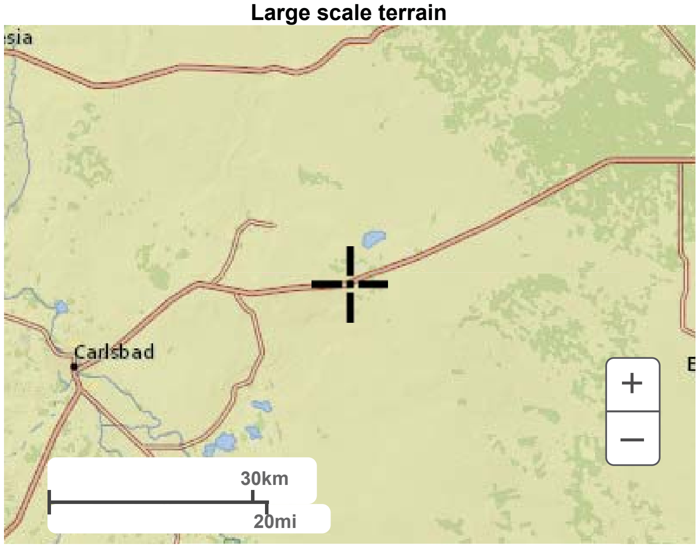
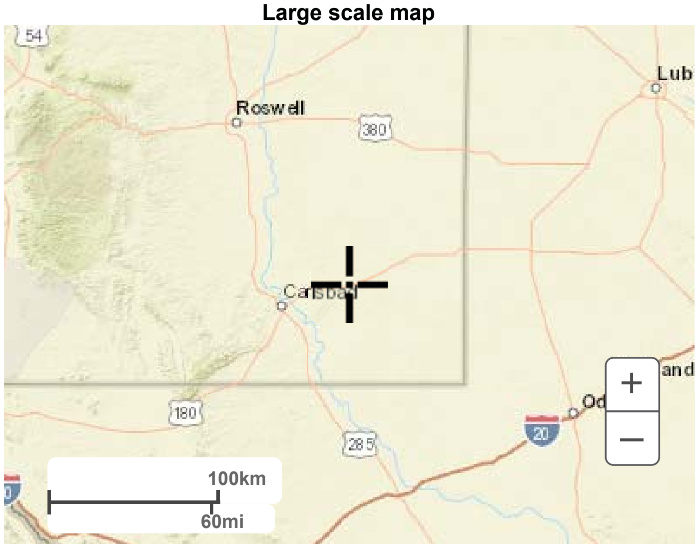
Maps & aerials

Small scale terrain

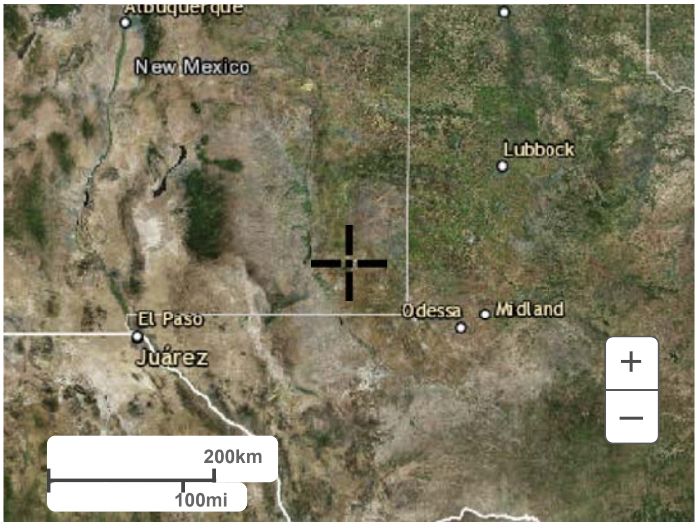
Large scale terrain



Large scale map



Large scale aerial



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National Oceanic and Atmospheric Administration
National Weather Service
National Water Center
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Silver Spring, MD 20910
Questions?: HDSC.Questions@noaa.gov

Disclaimer

ATTACHMENT III.3.C

AUTODESK® INC, 2017, STORM AND SANITARY ANALYSIS, MODEL OUTPUT – PRE-DEVELOPMENT CONDITION

Project Description

File Name .... PreConstruction.SPF

Project Options

Flow Units	CFS
Elevation Type	Elevation
Hydrology Method	SCS TR-20
Time of Concentration (TOC) Method	SCS TR-55
Link Routing Method	Kinematic Wave
Enable Overflow Ponding at Nodes	NO
Skip Steady State Analysis Time Periods ...	NO

Analysis Options

Start Analysis On	Apr 30, 2019	00:00:00
End Analysis On	May 01, 2019	00:00:00
Start Reporting On	Apr 30, 2019	00:00:00
Antecedent Dry Days	0	days
Runoff (Dry Weather) Time Step	0 01:00:00	days hh:mm:ss
Runoff (Wet Weather) Time Step	0 00:05:00	days hh:mm:ss
Reporting Time Step	0 00:05:00	days hh:mm:ss
Routing Time Step	30	seconds

Number of Elements

	Qty
Rain Gages	1
Subbasins...	8
Nodes...	9
Junctions	4
Outfalls	0
Flow Diversions	0
Inlets	0
Storage Nodes	5
Links...	8
Channels	3
Pipes	1
Pumps	0
Orifices	4
Weirs	0
Outlets	0
Pollutants	0
Land Uses	0

Rainfall Details

SN	Rain Gage ID	Data Source	Data Source ID	Rainfall Type	Rain Units	State	County	Return Period (years)	Rainfall Depth (inches)	Rainfall Distribution
1	LL-PFtable	Time Series	RG est 1	Cumulative	inches	New Mexico	Lea	25	4.48	NM Type IIA 65

Subbasin Summary

SN	Subbasin ID	Area (ac)	Weighted Curve Number	Total Rainfall (in)	Total Runoff (in)	Total Runoff Volume (ac-ft)	Peak Runoff (cfs)	Time of Concentration (days hh:mm:ss)
1	Runoff-Central	161.63	77.00	4.48	2.19	29.55	148.97	0 01:52:23
2	Runoff-East	161.25	77.00	4.48	2.19	29.48	232.63	0 01:07:52
3	Runoff-North	158.86	77.00	4.48	2.19	29.05	200.79	0 01:18:52
4	Runoff-West	127.44	77.00	4.48	2.19	23.30	176.25	0 01:11:15
5	RunOn-South	94.60	77.00	4.48	2.19	17.30	65.68	0 02:33:52
6	RunOn-SouthEast	75.01	77.00	4.48	2.19	13.71	30.52	0 04:39:06
7	RunOn-SouthWest	43.95	77.00	4.48	2.19	8.04	69.02	0 01:01:42
8	RunOn-West	31.10	77.00	4.48	2.19	5.69	78.93	0 00:34:54

Node Summary

SN	Element ID	Element Type	Invert Elevation (ft)	Ground/Rim (Max) Elevation (ft)	Initial Water Elevation (ft)	Surcharge Elevation (ft)	Ponded Area (ft²)	Peak Inflow (cfs)	Max HGL Elevation Attained (ft)	Max Surcharge Depth Attained (ft)	Min Freeboard Attained (ft)	Time of Peak Flooding Occurrence (days hh:mm)	Total Flooded Volume (ac-in)	Total Time Flooded (min)
1	HwyClvt-Cen-Out	Junction	3520.00	3523.00	3520.00	3523.00	0.00	69.66	3520.67	0.00	3.83	0 00:00	0.00	0.00
2	HwyClvt-E-Out	Junction	3514.00	3516.50	3514.00	3516.50	0.00	40.19	3514.63	0.00	4.37	0 00:00	0.00	0.00
3	NWBasinIn	Junction	3513.00	3516.00	3513.00	3516.00	50000.00	109.85	3514.67	0.00	1.33	0 00:00	0.00	0.00
4	WHwyCllctr	Junction	3552.00	3557.00	3552.00	3557.00	2000.00	20.11	3552.37	0.00	4.63	0 00:00	0.00	0.00
5	HwyClvt-Cen	Storage Node	3519.00	3526.00	3519.00		2000.00	195.95	3526.00				85.68	69.00
6	HwyClvt-E	Storage Node	3510.00	3515.00	3511.00		100.00	390.75	3515.00				574.29	294.00
7	NWBasin	Storage Node	3498.00	3508.00	3498.00		1800000.00	310.44	3498.00				0.00	0.00
8	RRClvt-Mid	Storage Node	3554.00	3557.00	3554.00		2500.00	77.84	3557.00				31.92	60.00
9	RRClvt-S	Storage Node	3564.00	3567.00	3564.00		2500.00	68.71	3567.00				46.07	100.00

Link Summary

SN	Element ID	Element Type	From (Inlet) Node	To (Outlet) Node	Length (ft)	Inlet Invert Elevation (ft)	Outlet Invert Elevation (ft)	Average Slope (%)	Diameter or Height (in)	Manning's Roughness	Peak Flow (cfs)	Design Flow Capacity (cfs)	Peak Flow/ Design Flow Ratio	Peak Flow Velocity (ft/sec)	Peak Flow Depth (ft)	Peak Flow Depth/ Total Depth Ratio	Total Time Surcharged (min)	Reported Condition
1	NWcollctn_NWbsn	Pipe	NWBasinIn	NWBasin	5342.82	3514.00	3498.00	0.3000	0.000	0.0320	109.85	0.00	0.45	0.00	0.63	0.63	0.00	Calculated
2	HwaySouthConveyance	Channel	WHwyCllctr	HwyClvt-Cen	1788.63	3552.00	3519.00	1.8400	36.000	0.0320	20.18	817.46	0.02	3.24	0.37	0.12	0.00	
3	HwyClvtC-Out_NWcollctn	Channel	HwyClvt-Cen-Out	NWBasinIn	400.00	3520.00	3514.00	1.5000	12.000	0.0320	69.66	138.05	0.50	4.72	0.67	0.67	0.00	
4	HwyClvtE-Out_NWcollctn	Channel	HwyClvt-E-Out	NWBasinIn	400.00	3514.00	3513.00	0.2500	12.000	0.0320	40.19	88.98	0.45	4.49	0.63	0.63	0.00	
5	HwyClvtC-Ctrl_NWcollctn	Orifice	HwyClvt-Cen	HwyClvt-Cen-Out		3519.00	3520.00		36.000		69.66							
6	HwyClvtE-Ctrl_NWcollctn	Orifice	HwyClvt-E	HwyClvt-E-Out		3510.00	3514.00		24.000		40.19							
7	RRClvtC_HwyClvtC	Orifice	RRClvt-Mid	WHwyCllctr		3554.00	3552.00		24.000		10.05							
8	RRClvtS_HwyClvtC	Orifice	RRClvt-S	WHwyCllctr		3564.00	3552.00		24.000		10.05							

Subbasin Hydrology

Subbasin : Runoff-Central

Input Data

Area (ac) 161.63
Weighted Curve Number 77.00
Rain Gage ID .... LL-PFtable

Composite Curve Number

Soil/Surface Description	Area (acres)	Soil Group	Curve Number
Desert shrub range, Poor	161.63	B	77.00
Composite Area & Weighted CN	161.63		77.00

Time of Concentration

TOC Method : SCS TR-55

Sheet Flow Equation :

Tc = (0.007 * ((n * Lf)^0.8)) / ((P^0.5) * (Sf^0.4))

Where :

Tc = Time of Concentration (hr)
n = Manning's roughness
Lf = Flow Length (ft)
P = 2 yr, 24 hr Rainfall (inches)
Sf = Slope (ft/ft)

Shallow Concentrated Flow Equation :

V = 16.1345 * (Sf^0.5) (unpaved surface)
V = 20.3282 * (Sf^0.5) (paved surface)
V = 15.0 * (Sf^0.5) (grassed waterway surface)
V = 10.0 * (Sf^0.5) (nearly bare & untilled surface)
V = 9.0 * (Sf^0.5) (cultivated straight rows surface)
V = 7.0 * (Sf^0.5) (short grass pasture surface)
V = 5.0 * (Sf^0.5) (woodland surface)
V = 2.5 * (Sf^0.5) (forest w/heavy litter surface)
Tc = (Lf / V) / (3600 sec/hr)

Where:

Tc = Time of Concentration (hr)
Lf = Flow Length (ft)
V = Velocity (ft/sec)
Sf = Slope (ft/ft)

Channel Flow Equation :

V = (1.49 * (R^(2/3)) * (Sf^0.5)) / n
R = Aq / Wp
Tc = (Lf / V) / (3600 sec/hr)

Where :

Tc = Time of Concentration (hr)
Lf = Flow Length (ft)
R = Hydraulic Radius (ft)
Aq = Flow Area (ft²)
Wp = Wetted Perimeter (ft)
V = Velocity (ft/sec)
Sf = Slope (ft/ft)
n = Manning's roughness

Sheet Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.15	0.00	0.00
Flow Length (ft) :	100	0.00	0.00
Slope (%) :	1	0.00	0.00
2 yr, 24 hr Rainfall (in) :	2.24	0.00	0.00
Velocity (ft/sec) :	0.11	0.00	0.00
Computed Flow Time (min) :	15.45	0.00	0.00

Shallow Concentrated Flow Computations	Subarea A	Subarea B	Subarea C
Flow Length (ft) :	1000	0.00	0.00
Slope (%) :	1	0.00	0.00
Surface Type :	Unpaved	Unpaved	Unpaved
Velocity (ft/sec) :	1.61	0.00	0.00
Computed Flow Time (min) :	10.35	0.00	0.00

Channel Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.1	0.00	0.00
Flow Length (ft) :	4400	0.00	0.00
Channel Slope (%) :	1	0.00	0.00
Cross Section Area (ft²) :	3	0.00	0.00
Wetted Perimeter (ft) :	7	0.00	0.00
Velocity (ft/sec) :	0.85	0.00	0.00
Computed Flow Time (min) :	86.58	0.00	0.00

Total TOC (min) 112.39

Subbasin Runoff Results

Total Rainfall (in) 4.48
Total Runoff (in) 2.19
Peak Runoff (cfs) 148.97
Weighted Curve Number 77.00
Time of Concentration (days hh:mm:ss) 0 01:52:23

Subbasin : Runoff-East

Input Data

Area (ac) 161.25
Weighted Curve Number 77.00
Rain Gage ID .... LL-PFtable

Composite Curve Number

Soil/Surface Description	Area (acres)	Soil Group	Curve Number
Desert shrub range, Poor	161.25	B	77.00
Composite Area & Weighted CN	161.25		77.00

Time of Concentration

Sheet Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.15	0.00	0.00
Flow Length (ft) :	100	0.00	0.00
Slope (%) :	.8	0.00	0.00
2 yr, 24 hr Rainfall (in) :	2.24	0.00	0.00
Velocity (ft/sec) :	0.10	0.00	0.00
Computed Flow Time (min) :	16.90	0.00	0.00

Shallow Concentrated Flow Computations	Subarea A	Subarea B	Subarea C
Flow Length (ft) :	1200	0.00	0.00
Slope (%) :	.8	0.00	0.00
Surface Type :	Unpaved	Unpaved	Unpaved
Velocity (ft/sec) :	1.44	0.00	0.00
Computed Flow Time (min) :	13.89	0.00	0.00

Channel Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.1	0.00	0.00
Flow Length (ft) :	1800	0.00	0.00
Channel Slope (%) :	1	0.00	0.00
Cross Section Area (ft²) :	2	0.00	0.00
Wetted Perimeter (ft) :	5	0.00	0.00
Velocity (ft/sec) :	0.81	0.00	0.00
Computed Flow Time (min) :	37.09	0.00	0.00

Total TOC (min) 67.87

Subbasin Runoff Results

Total Rainfall (in) 4.48
Total Runoff (in) 2.19
Peak Runoff (cfs) 232.63
Weighted Curve Number 77.00
Time of Concentration (days hh:mm:ss) 0 01:07:52

Subbasin : Runoff-North

Input Data

Area (ac) 158.86
Weighted Curve Number 77.00
Rain Gage ID .... LL-PFtable

Composite Curve Number

Soil/Surface Description	Area (acres)	Soil Group	Curve Number
Desert shrub range, Poor	158.86	B	77.00
Composite Area & Weighted CN	158.86		77.00

Time of Concentration

Sheet Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.15	0.00	0.00
Flow Length (ft) :	100	0.00	0.00
Slope (%) :	.5	0.00	0.00
2 yr, 24 hr Rainfall (in) :	2.24	0.00	0.00
Velocity (ft/sec) :	0.08	0.00	0.00
Computed Flow Time (min) :	20.39	0.00	0.00

Shallow Concentrated Flow Computations	Subarea A	Subarea B	Subarea C
Flow Length (ft) :	4000	0.00	0.00
Slope (%) :	.5	0.00	0.00
Surface Type :	Unpaved	Unpaved	Unpaved
Velocity (ft/sec) :	1.14	0.00	0.00
Computed Flow Time (min) :	58.48	0.00	0.00

Total TOC (min) 78.87

Subbasin Runoff Results

Total Rainfall (in) 4.48
Total Runoff (in) 2.19
Peak Runoff (cfs) 200.79
Weighted Curve Number 77.00
Time of Concentration (days hh:mm:ss) 0 01:18:52

Subbasin : Runoff-West

Input Data

Area (ac) 127.44
Weighted Curve Number 77.00
Rain Gage ID .... LL-PFtable

Composite Curve Number

Soil/Surface Description	Area (acres)	Soil Group	Curve Number
Desert shrub range, Poor	127.44	B	77.00
Composite Area & Weighted CN	127.44		77.00

Time of Concentration

Sheet Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.15	0.00	0.00
Flow Length (ft) :	100	0.00	0.00
Slope (%) :	1.4	0.00	0.00
2 yr, 24 hr Rainfall (in) :	2.24	0.00	0.00
Velocity (ft/sec) :	0.12	0.00	0.00
Computed Flow Time (min) :	13.51	0.00	0.00

Shallow Concentrated Flow Computations	Subarea A	Subarea B	Subarea C
Flow Length (ft) :	2000	0.00	0.00
Slope (%) :	1.38	0.00	0.00
Surface Type :	Unpaved	Unpaved	Unpaved
Velocity (ft/sec) :	1.90	0.00	0.00
Computed Flow Time (min) :	17.54	0.00	0.00

Channel Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.1	0.00	0.00
Flow Length (ft) :	2400	0.00	0.00
Channel Slope (%) :	1.38	0.00	0.00
Cross Section Area (ft^2) :	3	0.00	0.00
Wetted Perimeter (ft) :	7	0.00	0.00
Velocity (ft/sec) :	0.99	0.00	0.00
Computed Flow Time (min) :	40.20	0.00	0.00

Total TOC (min) 71.25

Subbasin Runoff Results

Total Rainfall (in) 4.48
Total Runoff (in) 2.19
Peak Runoff (cfs) 176.25
Weighted Curve Number 77.00
Time of Concentration (days hh:mm:ss) 0 01:11:15

Subbasin : RunOn-South

Input Data

Area (ac) 94.60
Weighted Curve Number 77.00
Rain Gage ID .... LL-PFtable

Composite Curve Number

Soil/Surface Description	Area (acres)	Soil Group	Curve Number
Desert shrub range, Poor	94.60	B	77.00
Composite Area & Weighted CN	94.60		77.00

Time of Concentration

Sheet Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.15	0.00	0.00
Flow Length (ft) :	100	0.00	0.00
Slope (%) :	1.5	0.00	0.00
2 yr, 24 hr Rainfall (in) :	2.24	0.00	0.00
Velocity (ft/sec) :	0.13	0.00	0.00
Computed Flow Time (min) :	13.14	0.00	0.00

Shallow Concentrated Flow Computations	Subarea A	Subarea B	Subarea C
Flow Length (ft) :	3400	0.00	0.00
Slope (%) :	1.5	0.00	0.00
Surface Type :	Unpaved	Unpaved	Unpaved
Velocity (ft/sec) :	1.98	0.00	0.00
Computed Flow Time (min) :	28.62	0.00	0.00

Channel Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.1	0.00	0.00
Flow Length (ft) :	5200	0.00	0.00
Channel Slope (%) :	.77	0.00	0.00
Cross Section Area (ft²) :	5	0.00	0.00
Wetted Perimeter (ft) :	11	0.00	0.00
Velocity (ft/sec) :	0.77	0.00	0.00
Computed Flow Time (min) :	112.12	0.00	0.00

Total TOC (min) 153.88

Subbasin Runoff Results

Total Rainfall (in) 4.48
Total Runoff (in) 2.19
Peak Runoff (cfs) 65.68
Weighted Curve Number 77.00
Time of Concentration (days hh:mm:ss) 0 02:33:53

Subbasin : RunOn-SouthEast

Input Data

Area (ac) 75.01
Weighted Curve Number 77.00
Rain Gage ID .... LL-PFtable

Composite Curve Number

Soil/Surface Description	Area (acres)	Soil Group	Curve Number
Desert shrub range, Poor	75.01	B	77.00
Composite Area & Weighted CN	75.01		77.00

Time of Concentration

Sheet Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.15	0.00	0.00
Flow Length (ft) :	100	0.00	0.00
Slope (%) :	1.9	0.00	0.00
2 yr, 24 hr Rainfall (in) :	2.24	0.00	0.00
Velocity (ft/sec) :	0.14	0.00	0.00
Computed Flow Time (min) :	11.95	0.00	0.00

Shallow Concentrated Flow Computations	Subarea A	Subarea B	Subarea C
Flow Length (ft) :	2300	0.00	0.00
Slope (%) :	1.9	0.00	0.00
Surface Type :	Unpaved	Unpaved	Unpaved
Velocity (ft/sec) :	2.22	0.00	0.00
Computed Flow Time (min) :	17.27	0.00	0.00

Channel Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.2	0.00	0.00
Flow Length (ft) :	4600	0.00	0.00
Channel Slope (%) :	.5	0.00	0.00
Cross Section Area (ft²) :	4	0.00	0.00
Wetted Perimeter (ft) :	9	0.00	0.00
Velocity (ft/sec) :	0.31	0.00	0.00
Computed Flow Time (min) :	249.89	0.00	0.00

Total TOC (min) 279.11

Subbasin Runoff Results

Total Rainfall (in) 4.48
Total Runoff (in) 2.19
Peak Runoff (cfs) 30.52
Weighted Curve Number 77.00
Time of Concentration (days hh:mm:ss) 0 04:39:07

Subbasin : RunOn-SouthWest

Input Data

Area (ac) 43.95
Weighted Curve Number 77.00
Rain Gage ID .... LL-PFtable

Composite Curve Number

Soil/Surface Description	Area (acres)	Soil Group	Curve Number
Desert shrub range, Poor	43.95	B	77.00
Composite Area & Weighted CN	43.95		77.00

Time of Concentration

Sheet Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.15	0.00	0.00
Flow Length (ft) :	100	0.00	0.00
Slope (%) :	1.35	0.00	0.00
2 yr, 24 hr Rainfall (in) :	2.24	0.00	0.00
Velocity (ft/sec) :	0.12	0.00	0.00
Computed Flow Time (min) :	13.70	0.00	0.00

Shallow Concentrated Flow Computations	Subarea A	Subarea B	Subarea C
Flow Length (ft) :	1000	0.00	0.00
Slope (%) :	1.35	0.00	0.00
Surface Type :	Unpaved	Unpaved	Unpaved
Velocity (ft/sec) :	1.87	0.00	0.00
Computed Flow Time (min) :	8.91	0.00	0.00

Channel Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.1	0.00	0.00
Flow Length (ft) :	2400	0.00	0.00
Channel Slope (%) :	1.35	0.00	0.00
Cross Section Area (ft^2) :	5	0.00	0.00
Wetted Perimeter (ft) :	11	0.00	0.00
Velocity (ft/sec) :	1.02	0.00	0.00
Computed Flow Time (min) :	39.08	0.00	0.00

Total TOC (min) 61.70

Subbasin Runoff Results

Total Rainfall (in) 4.48
Total Runoff (in) 2.19
Peak Runoff (cfs) 69.02
Weighted Curve Number 77.00
Time of Concentration (days hh:mm:ss) 0 01:01:42

Subbasin : RunOn-West

Input Data

Area (ac) 31.10
Weighted Curve Number 77.00
Rain Gage ID .... LL-PFtable

Composite Curve Number

Soil/Surface Description	Area (acres)	Soil Group	Curve Number
Desert shrub range, Poor	31.10	B	77.00
Composite Area & Weighted CN	31.10		77.00

Time of Concentration

Sheet Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.15	0.00	0.00
Flow Length (ft) :	100	0.00	0.00
Slope (%) :	1.85	0.00	0.00
2 yr, 24 hr Rainfall (in) :	2.24	0.00	0.00
Velocity (ft/sec) :	0.14	0.00	0.00
Computed Flow Time (min) :	12.08	0.00	0.00

Shallow Concentrated Flow Computations	Subarea A	Subarea B	Subarea C
Flow Length (ft) :	3000	0.00	0.00
Slope (%) :	1.85	0.00	0.00
Surface Type :	Unpaved	Unpaved	Unpaved
Velocity (ft/sec) :	2.19	0.00	0.00
Computed Flow Time (min) :	22.83	0.00	0.00

Total TOC (min) 34.91

Subbasin Runoff Results

Total Rainfall (in) 4.48
Total Runoff (in) 2.19
Peak Runoff (cfs) 78.93
Weighted Curve Number 77.00
Time of Concentration (days hh:mm:ss) 0 00:34:55

Junction Input

SN	Element ID	Invert Elevation (ft)	Ground/Rim (Max) Elevation (ft)	Ground/Rim (Max) Offset (ft)	Initial Water Elevation (ft)	Initial Water Depth (ft)	Surcharge Elevation (ft)	Surcharge Depth (ft)	Ponded Area (ft²)	Minimum Pipe Cover (in)
1	HwyClvt-Cen-Out	3520.00	3523.00	3.00	3520.00	0.00	3523.00	0.00	0.00	0.00
2	HwyClvt-E-Out	3514.00	3516.50	2.50	3514.00	0.00	3516.50	0.00	0.00	0.00
3	NWBasinIn	3513.00	3516.00	3.00	3513.00	0.00	3516.00	0.00	50000.00	0.00
4	WHwyCllctr	3552.00	3557.00	5.00	3552.00	0.00	3557.00	0.00	2000.00	0.00

Junction Results

SN	Element ID	Peak Inflow (cfs)	Peak Lateral Inflow (cfs)	Max HGL Elevation Attained (ft)	Max HGL Depth Attained (ft)	Max Surcharge Depth Attained (ft)	Min Freeboard Attained (ft)	Average HGL Elevation Attained (ft)	Average HGL Depth Attained (ft)	Time of Max HGL Occurrence (days hh:mm)	Time of Peak Flooding Occurrence (days hh:mm)	Total Flooded Volume (ac-in)	Total Time Flooded (min)
1	HwyClvt-Cen-Out	69.66	0.00	3520.67	0.67	0.00	3.83	3520.18	0.18	0 06:44	0 00:00	0.00	0.00
2	HwyClvt-E-Out	40.19	0.00	3514.63	0.63	0.00	4.37	3514.35	0.35	0 06:46	0 00:00	0.00	0.00
3	NWBasinIn	109.85	0.00	3514.67	1.67	0.00	1.33	3514.18	1.18	0 07:38	0 00:00	0.00	0.00
4	WHwyCllctr	20.11	0.00	3552.37	0.37	0.00	4.63	3552.10	0.10	0 06:29	0 00:00	0.00	0.00

Channel Input

SN	Element ID	Length (ft)	Inlet Invert Elevation (ft)	Inlet Invert Offset (ft)	Outlet Invert Elevation (ft)	Outlet Invert Offset (ft)	Total Drop (ft)	Average Slope (%)	Shape	Height (ft)	Width (ft)	Manning's Roughness	Entrance Losses	Exit/Bend Losses	Additional Losses	Initial Flow (cfs)	Flap Gate
1	HwaySouthConveyance	1788.63	3552.00	0.00	3519.00	0.00	33.00	1.8400	Trapezoidal	3.000	34.000	0.0320	0.5000	0.5000	0.0000	0.00	No
2	HwyClvtC-Out_NWcollctn	400.00	3520.00	0.00	3514.00	1.00	6.00	1.5000	Trapezoidal	1.000	26.000	0.0320	0.5000	0.5000	0.0000	0.00	No
3	HwyClvtE-Out_NWcollctn	400.00	3514.00	0.00	3513.00	0.00	1.00	0.2500	Trapezoidal	1.000	17.000	0.0320	0.5000	0.5000	0.0000	0.00	No

Channel Results

SN	Element ID	Peak Flow (cfs)	Time of Peak Flow Occurrence (days hh:mm)	Design Flow Capacity (cfs)	Peak Flow/ Design Flow Ratio	Peak Flow Velocity (ft/sec)	Travel Time (min)	Peak Flow Depth (ft)	Peak Flow Depth/ Total Depth Ratio	Total Time Surcharged (min)	Froude Number	Reported Condition
1	HwaySouthConveyance	20.18	0 07:11	817.46	0.02	3.24	9.20	0.37	0.12	0.00		
2	HwyClvtC-Out_NWcollctn	69.66	0 07:38	138.05	0.50	4.72	1.41	0.67	0.67	0.00		
3	HwyClvtE-Out_NWcollctn	40.19	0 11:25	88.98	0.45	4.49	1.48	0.63	0.63	0.00		

Storage Nodes

Storage Node : HwyClvt-Cen

Input Data

Invert Elevation (ft)	3519.00
Max (Rim) Elevation (ft)	3526.00
Max (Rim) Offset (ft)	7.00
Initial Water Elevation (ft)	3519.00
Initial Water Depth (ft)	0.00
Ponded Area (ft²)	2000.00
Evaporation Loss	0.00

Output Summary Results

Peak Inflow (cfs)	195.95
Peak Lateral Inflow (cfs)	175.84
Peak Outflow (cfs)	69.66
Peak Exfiltration Flow Rate (cfm)	0.00
Max HGL Elevation Attained (ft)	3526.00
Max HGL Depth Attained (ft)	7
Average HGL Elevation Attained (ft)	3521.22
Average HGL Depth Attained (ft)	2.22
Time of Max HGL Occurrence (days hh:mm)	0 06:47
Total Exfiltration Volume (1000-ft³)	0.000
Total Flooded Volume (ac-in)	85.68
Total Time Flooded (min)	69
Total Retention Time (sec)	0.00

Storage Node : HwyClvt-E

Input Data

Invert Elevation (ft)	3510.00
Max (Rim) Elevation (ft)	3515.00
Max (Rim) Offset (ft)	5.00
Initial Water Elevation (ft)	3511.00
Initial Water Depth (ft)	1.00
Ponded Area (ft^2)	100.00
Evaporation Loss	0.00

Output Summary Results

Peak Inflow (cfs)	390.75
Peak Lateral Inflow (cfs)	390.75
Peak Outflow (cfs)	40.19
Peak Exfiltration Flow Rate (cfm)	0.00
Max HGL Elevation Attained (ft)	3515.00
Max HGL Depth Attained (ft)	5
Average HGL Elevation Attained (ft)	3513.57
Average HGL Depth Attained (ft)	3.57
Time of Max HGL Occurrence (days hh:mm)	0 06:50
Total Exfiltration Volume (1000-ft^3)	0.000
Total Flooded Volume (ac-in)	574.29
Total Time Flooded (min)	294
Total Retention Time (sec)	0.00

Storage Node : NWBasin

Input Data

Invert Elevation (ft) 3498.00
Max (Rim) Elevation (ft) 3508.00
Max (Rim) Offset (ft) 10.00
Initial Water Elevation (ft) 3498.00
Initial Water Depth (ft) 0.00
Ponded Area (ft^2) 1800000.00
Evaporation Loss 0.00

Output Summary Results

Peak Inflow (cfs) 310.44
Peak Lateral Inflow (cfs) 200.59
Peak Outflow (cfs) 0.00
Peak Exfiltration Flow Rate (cfm) 0.00
Max HGL Elevation Attained (ft) 3498.00
Max HGL Depth Attained (ft) 0
Average HGL Elevation Attained (ft) 3498.00
Average HGL Depth Attained (ft) 0
Time of Max HGL Occurrence (days hh:mm) 0 00:00
Total Exfiltration Volume (1000-ft^3) 0.000
Total Flooded Volume (ac-in) 0
Total Time Flooded (min) 0
Total Retention Time (sec) 0.00

Storage Node : RRClvt-Mid

Input Data

Invert Elevation (ft) 3554.00
Max (Rim) Elevation (ft) 3557.00
Max (Rim) Offset (ft) 3.00
Initial Water Elevation (ft) 3554.00
Initial Water Depth (ft) 0.00
Ponded Area (ft²) 2500.00
Evaporation Loss 0.00

Output Summary Results

Peak Inflow (cfs) 77.84
Peak Lateral Inflow (cfs) 77.84
Peak Outflow (cfs) 10.05
Peak Exfiltration Flow Rate (cfm) 0.00
Max HGL Elevation Attained (ft) 3557.00
Max HGL Depth Attained (ft) 3
Average HGL Elevation Attained (ft) 3555.47
Average HGL Depth Attained (ft) 1.47
Time of Max HGL Occurrence (days hh:mm) 0 06:27
Total Exfiltration Volume (1000-ft³) 0.000
Total Flooded Volume (ac-in) 31.92
Total Time Flooded (min) 60
Total Retention Time (sec) 0.00

Storage Node : RRClvt-S

Input Data

Invert Elevation (ft) 3564.00
Max (Rim) Elevation (ft) 3567.00
Max (Rim) Offset (ft) 3.00
Initial Water Elevation (ft) 3564.00
Initial Water Depth (ft) 0.00
Ponded Area (ft²) 2500.00
Evaporation Loss 0.00

Output Summary Results

Peak Inflow (cfs) 68.71
Peak Lateral Inflow (cfs) 68.71
Peak Outflow (cfs) 10.05
Peak Exfiltration Flow Rate (cfm) 0.00
Max HGL Elevation Attained (ft) 3567.00
Max HGL Depth Attained (ft) 3
Average HGL Elevation Attained (ft) 3565.55
Average HGL Depth Attained (ft) 1.55
Time of Max HGL Occurrence (days hh:mm) 0 06:33
Total Exfiltration Volume (1000-ft³) 0.000
Total Flooded Volume (ac-in) 46.07
Total Time Flooded (min) 100
Total Retention Time (sec) 0.00

ATTACHMENT III.3.D
AUTODESK® INC, 2017, STORM AND SANITARY ANALYSIS,
MODEL OUTPUT – FINAL CONDITION

Project Description

File Name .... FinalCondition.SPF

Project Options

Flow Units	CFS
Elevation Type	Elevation
Hydrology Method	SCS TR-20
Time of Concentration (TOC) Method	SCS TR-55
Link Routing Method	Kinematic Wave
Enable Overflow Ponding at Nodes	NO
Skip Steady State Analysis Time Periods ...	NO

Analysis Options

Start Analysis On	Mar 31, 2019	00:00:00
End Analysis On	Apr 01, 2019	00:00:00
Start Reporting On	Mar 31, 2019	00:00:00
Antecedent Dry Days	0	days
Runoff (Dry Weather) Time Step	0 01:00:00	days hh:mm:ss
Runoff (Wet Weather) Time Step	0 00:05:00	days hh:mm:ss
Reporting Time Step	0 00:05:00	days hh:mm:ss
Routing Time Step	30	seconds

Number of Elements

	Qty
Rain Gages	1
Subbasins..	11
Nodes..	19
Junctions	10
Outfalls	0
Flow Diversions	0
Inlets	0
Storage Nodes	9
Links..	16
Channels	10
Pipes	0
Pumps	0
Orifices	4
Weirs	2
Outlets	0
Pollutants	0
Land Uses	0

Rainfall Details

SN	Rain Gage ID	Data Source	Data Source ID	Rainfall Type	Rain Units	State	County	Return Period (years)	Rainfall Depth (inches)	Rainfall Distribution
1	LL-PFtable	Time Series	RG est 1	Cumulative	inches	New Mexico	Lea	25	4.48	NM Type IIA 65

Subbasin Summary

SN	Subbasin ID	Area (ac)	Weighted Curve Number	Total Rainfall (in)	Total Runoff (in)	Total Runoff Volume (ac-ft)	Peak Runoff (cfs)	Time of Concentration (days hh:mm:ss)
1	Processing	57.98	77.00	4.48	2.19	10.60	54.06	0 01:50:56
2	RoadsideEast	26.63	77.00	4.48	2.19	4.87	90.89	0 00:23:19
3	RoadsideW	12.87	77.00	4.48	2.19	2.35	21.99	0 00:56:03
4	Runoff-Central	103.88	77.00	4.48	2.19	18.99	183.00	0 00:54:04
5	Runoff-East	57.16	77.00	4.48	2.19	10.45	82.47	0 01:07:52
6	Runoff-North	158.86	77.00	4.48	2.19	29.05	200.79	0 01:18:52
7	Runoff-West	115.39	77.00	4.48	2.19	21.10	159.59	0 01:11:15
8	RunOn-South	94.60	77.00	4.48	2.19	17.30	65.68	0 02:33:52
9	RunOn-SouthEast	75.01	77.00	4.48	2.19	13.71	30.52	0 04:39:06
10	RunOn-SouthWest	43.95	77.00	4.48	2.19	8.04	69.02	0 01:01:42
11	RunOn-West	31.10	77.00	4.48	2.19	5.69	78.93	0 00:34:54

Node Summary

SN	Element ID	Element Type	Invert Elevation (ft)	Ground/Rim (Max) Elevation (ft)	Initial Water Elevation (ft)	Surcharge Elevation (ft)	Ponded Area (ft²)	Peak Inflow (cfs)	Max HGL Elevation Attained (ft)	Max Surcharge Depth Attained (ft)	Min Freeboard Attained (ft)	Time of Peak Flooding Occurrence (days hh:mm)	Total Flooded Volume (ac-in)	Total Time Flooded (min)
1	E_BdryBasinOut	Junction	3519.00	3522.00	3519.00	3522.00	0.00	107.67	3520.53	0.00	4.47	0 00:00	0.00	0.00
2	HwyClvt-Cen-Out	Junction	3520.00	3523.00	3520.00	3523.00	0.00	33.63	3520.44	0.00	4.06	0 00:00	0.00	0.00
3	HwyClvt-E-Out	Junction	3509.00	3512.00	3509.00	3512.00	0.00	164.41	3515.61	0.00	0.89	0 00:00	0.00	0.00
4	NWBasinIn	Junction	3513.00	3516.00	3513.00	3516.00	50000.00	338.96	3515.39	0.00	0.61	0 00:00	0.00	0.00
5	RR-Culv-N-Out	Junction	3555.50	3557.50	3555.50	3557.50	0.00	15.08	3555.91	0.00	2.59	0 00:00	0.00	0.00
6	RR-Culv-S-Out	Junction	3565.50	3567.50	2565.50	3567.50	0.00	7.54	3565.86	0.00	2.64	0 00:00	0.00	0.00
7	S_Drn-E-SCnr	Junction	3536.00	3539.00	3535.00	3539.00	0.00	86.78	3537.36	0.00	1.64	0 00:00	0.00	0.00
8	S_Drn-NE_CNR	Junction	3514.40	3517.40	3514.40	3517.40	0.00	107.63	3515.93	0.00	1.47	0 00:00	0.00	0.00
9	S_Drn-S-ECnr	Junction	3524.00	3527.00	3524.00	3527.00	0.00	86.73	3525.36	0.00	1.64	0 00:00	0.00	0.00
10	S_Drn-W-S_Start	Junction	3556.00	3559.00	3556.00	3559.00	0.00	65.42	3557.16	0.00	1.84	0 00:00	0.00	0.00
11	ContactWtrStrg	Storage Node	3526.00	3536.00	3524.00		152460.00	159.21	3526.00				0.00	0.00
12	E_Bdry-Mid	Storage Node	3517.00	3523.00	3517.00		500.00	107.82	3522.03				0.00	0.00
13	Hwy-CenEst	Storage Node	3513.00	3519.00	3513.00		6000.00	182.18	3518.87				0.00	0.00
14	HwyClvt-Cen	Storage Node	3519.00	3526.00	3519.00		2000.00	36.74	3523.06				0.00	0.00
15	HwyClvt-E	Storage Node	3510.00	3515.00	3510.00		1000.00	183.19	3514.90				0.00	0.00
16	NWBasin	Storage Node	3498.00	3508.00	3498.00		1800000.00	336.84	3498.00				0.00	0.00
17	ProcStrg	Storage Node	3526.00	3529.00	3526.00		1000.00	53.99	3526.00				0.00	0.00
18	RRClvt-N	Storage Node	3554.00	3557.00	3554.00		1000.00	77.84	3557.00				34.62	69.00
19	RRClvt-S	Storage Node	3564.00	3567.00	3564.00		2500.00	68.71	3567.00				50.55	114.00

Link Summary

SN	Element ID	Element Type	From (Inlet) Node	To (Outlet) Node	Length (ft)	Inlet Invert Elevation (ft)	Outlet Invert Elevation (ft)	Average Slope (%)	Diameter or Height (in)	Manning's Roughness	Peak Flow (cfs)	Design Flow Capacity (cfs)	Peak Flow/ Design Flow Ratio	Peak Flow Velocity (ft/sec)	Peak Flow Depth (ft)	Peak Flow Depth/ Total Depth Ratio	Total Time Surcharged (min)	Reported Condition
1	E-Prop-Nhalf	Channel	E_BdryBasinOut	S_Drn-NE_CNR	1499.64	3517.00	3514.40	0.1700	36.000	0.0360	107.63	384.13	0.28	6.38	1.53	0.51	0.00	
2	E-Prop-Shalf	Channel	S_Drn-S-ECnr	E_Bdry-Mid	1284.15	3524.00	3520.00	0.3100	36.000	0.0360	86.72	384.13	0.23	5.97	1.36	0.45	0.00	
3	HwyClvtC-Out_NWcollctn	Channel	HwyClvt-Cen-Out	NWBasinIn	2194.11	3520.00	3514.00	0.2700	24.000	0.0320	33.26	465.74	0.07	3.64	0.43	0.22	0.00	
4	HwyClvtE-Out_NWcollctn	Channel	HwyClvt-E-Out	NWBasinIn	3882.48	3514.50	3513.00	0.0400	24.000	0.0320	164.32	465.74	0.35	6.38	1.11	0.55	0.00	
5	HwyWtoCen	Channel	RR-Culv-N-Out	HwyClvt-Cen	2375.24	3555.50	3519.00	1.5400	12.000	0.0320	15.13	69.56	0.22	3.45	0.41	0.41	0.00	
6	NEcnr	Channel	S_Drn-NE_CNR	HwyClvt-E	905.56	3514.40	3513.00	0.1500	36.000	0.0320	107.61	432.15	0.25	6.92	1.43	0.48	0.00	
7	NWcollctn_NWbsn	Channel	NWBasinIn	NWBasin	5342.82	3514.00	3498.00	0.3000	24.000	0.0320	336.84	620.76	0.54	7.82	1.38	0.69	0.00	
8	SEcnr	Channel	S_Drn-E-SCnr	S_Drn-S-ECnr	1816.82	3536.00	3524.00	0.6600	36.000	0.0360	86.73	384.13	0.23	5.97	1.36	0.45	0.00	
9	S-Prop	Channel	S_Drn-W-S_Start	S_Drn-E-SCnr	1421.18	3556.00	3536.00	1.4100	36.000	0.0360	65.42	384.13	0.17	5.48	1.16	0.39	0.00	
10	W-Prop-LDAedge	Channel	RR-Culv-S-Out	RR-Culv-N-Out	863.84	3565.50	3555.50	1.1600	12.000	0.0320	7.54	43.90	0.17	3.09	0.36	0.36	0.00	
11	HwyClvtC-Ctrl_NWcollctn	Orifice	HwyClvt-Cen	HwyClvt-Cen-Out		3519.00	3520.00		36.000		33.63							
12	HwyClvtE-Ctrl_NWcollctn	Orifice	HwyClvt-E	HwyClvt-E-Out		3510.00	3509.00		48.000		164.41							
13	RRClvtC_HwyClvtC	Orifice	RRClvt-N	RR-Culv-N-Out		3554.00	3555.50		18.000		7.54							
14	RRClvtS_HwyClvtC	Orifice	RRClvt-S	RR-Culv-S-Out		3564.00	3565.50		18.000		7.54							
15	Link-17	Weir	E_Bdry-Mid	E_BdryBasinOut		3517.00	3519.00				107.67							
16	MidBasinToEast	Weir	Hwy-CenEst	HwyClvt-E		3513.00	3510.00				70.36							

Subbasin Hydrology

Subbasin : Processing

Input Data

Area (ac) 57.98
Weighted Curve Number 77.00
Rain Gage ID .... LL-PFtable

Composite Curve Number

Soil/Surface Description	Area (acres)	Soil Group	Curve Number
Desert shrub range, Poor	115.39	B	77.00
Composite Area & Weighted CN	115.39		77.00

Time of Concentration

TOC Method : SCS TR-55

Sheet Flow Equation :

Tc = (0.007 * ((n * Lf)^0.8)) / ((P^0.5) * (Sf^0.4))

Where :

Tc = Time of Concentration (hr)
n = Manning's roughness
Lf = Flow Length (ft)
P = 2 yr, 24 hr Rainfall (inches)
Sf = Slope (ft/ft)

Shallow Concentrated Flow Equation :

V = 16.1345 * (Sf^0.5) (unpaved surface)
V = 20.3282 * (Sf^0.5) (paved surface)
V = 15.0 * (Sf^0.5) (grassed waterway surface)
V = 10.0 * (Sf^0.5) (nearly bare & untilled surface)
V = 9.0 * (Sf^0.5) (cultivated straight rows surface)
V = 7.0 * (Sf^0.5) (short grass pasture surface)
V = 5.0 * (Sf^0.5) (woodland surface)
V = 2.5 * (Sf^0.5) (forest w/heavy litter surface)
Tc = (Lf / V) / (3600 sec/hr)

Where:

Tc = Time of Concentration (hr)
Lf = Flow Length (ft)
V = Velocity (ft/sec)
Sf = Slope (ft/ft)

Channel Flow Equation :

V = (1.49 * (R^(2/3)) * (Sf^0.5)) / n
R = Aq / Wp
Tc = (Lf / V) / (3600 sec/hr)

Where :

Tc = Time of Concentration (hr)
Lf = Flow Length (ft)
R = Hydraulic Radius (ft)
Aq = Flow Area (ft^2)
Wp = Wetted Perimeter (ft)
V = Velocity (ft/sec)
Sf = Slope (ft/ft)
n = Manning's roughness

Sheet Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.15	0.00	0.00
Flow Length (ft) :	100	0.00	0.00
Slope (%) :	.5	0.00	0.00
2 yr, 24 hr Rainfall (in) :	2.24	0.00	0.00
Velocity (ft/sec) :	0.08	0.00	0.00
Computed Flow Time (min) :	20.39	0.00	0.00

Shallow Concentrated Flow Computations	Subarea A	Subarea B	Subarea C
Flow Length (ft) :	1000	0.00	0.00
Slope (%) :	.5	0.00	0.00
Surface Type :	Paved	Unpaved	Unpaved
Velocity (ft/sec) :	1.44	0.00	0.00
Computed Flow Time (min) :	11.57	0.00	0.00

Channel Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.1	0.00	0.00
Flow Length (ft) :	2400	0.00	0.00
Channel Slope (%) :	.5	0.00	0.00
Cross Section Area (ft^2) :	2	0.00	0.00
Wetted Perimeter (ft) :	6	0.00	0.00
Velocity (ft/sec) :	0.51	0.00	0.00
Computed Flow Time (min) :	78.97	0.00	0.00

Total TOC (min) 110.94

Subbasin Runoff Results

Total Rainfall (in) 4.48
Total Runoff (in) 2.19
Peak Runoff (cfs) 54.06
Weighted Curve Number 77.00
Time of Concentration (days hh:mm:ss) 0 01:50:56

Subbasin : RoadsideEast

Input Data

Area (ac) 26.63
Weighted Curve Number 77.00
Rain Gage ID .... LL-PFtable

Composite Curve Number

Soil/Surface Description	Area (acres)	Soil Group	Curve Number
Desert shrub range, Poor	26.63	B	77.00
Composite Area & Weighted CN	26.63		77.00

Time of Concentration

Sheet Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.15	0.00	0.00
Flow Length (ft) :	100	0.00	0.00
Slope (%) :	1	0.00	0.00
2 yr, 24 hr Rainfall (in) :	2.24	0.00	0.00
Velocity (ft/sec) :	0.11	0.00	0.00
Computed Flow Time (min) :	15.45	0.00	0.00

Channel Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.1	0.00	0.00
Flow Length (ft) :	400	0.00	0.00
Channel Slope (%) :	1	0.00	0.00
Cross Section Area (ft²) :	3	0.00	0.00
Wetted Perimeter (ft) :	7	0.00	0.00
Velocity (ft/sec) :	0.85	0.00	0.00
Computed Flow Time (min) :	7.87	0.00	0.00

Total TOC (min) 23.32

Subbasin Runoff Results

Total Rainfall (in) 4.48
Total Runoff (in) 2.19
Peak Runoff (cfs) 90.89
Weighted Curve Number 77.00
Time of Concentration (days hh:mm:ss) 0 00:23:19

Subbasin : RoadsideW

Input Data

Area (ac) 12.87
Weighted Curve Number 77.00
Rain Gage ID .... LL-PFtable

Composite Curve Number

Soil/Surface Description	Area (acres)	Soil Group	Curve Number
Desert shrub range, Poor	12.87	B	77.00
Composite Area & Weighted CN	12.87		77.00

Time of Concentration

Sheet Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.15	0.00	0.00
Flow Length (ft) :	100	0.00	0.00
Slope (%) :	1	0.00	0.00
2 yr, 24 hr Rainfall (in) :	2.24	0.00	0.00
Velocity (ft/sec) :	0.11	0.00	0.00
Computed Flow Time (min) :	15.45	0.00	0.00

Shallow Concentrated Flow Computations	Subarea A	Subarea B	Subarea C
Flow Length (ft) :	500	0.00	0.00
Slope (%) :	1	0.00	0.00
Surface Type :	Unpaved	Unpaved	Unpaved
Velocity (ft/sec) :	1.61	0.00	0.00
Computed Flow Time (min) :	5.18	0.00	0.00

Channel Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.1	0.00	0.00
Flow Length (ft) :	1800	0.00	0.00
Channel Slope (%) :	1	0.00	0.00
Cross Section Area (ft²) :	3	0.00	0.00
Wetted Perimeter (ft) :	7	0.00	0.00
Velocity (ft/sec) :	0.85	0.00	0.00
Computed Flow Time (min) :	35.42	0.00	0.00

Total TOC (min) 56.05

Subbasin Runoff Results

Total Rainfall (in) 4.48
Total Runoff (in) 2.19
Peak Runoff (cfs) 21.99
Weighted Curve Number 77.00
Time of Concentration (days hh:mm:ss) 0 00:56:03

Subbasin : Runoff-Central

Input Data

Area (ac) 103.88
Weighted Curve Number 77.00
Rain Gage ID .... LL-PFtable

Composite Curve Number

Soil/Surface Description	Area (acres)	Soil Group	Curve Number
Desert shrub range, Poor	161.63	B	77.00
Composite Area & Weighted CN	161.63		77.00

Time of Concentration

Sheet Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.15	0.00	0.00
Flow Length (ft) :	100	0.00	0.00
Slope (%) :	1	0.00	0.00
2 yr, 24 hr Rainfall (in) :	2.24	0.00	0.00
Velocity (ft/sec) :	0.11	0.00	0.00
Computed Flow Time (min) :	15.45	0.00	0.00

Shallow Concentrated Flow Computations	Subarea A	Subarea B	Subarea C
Flow Length (ft) :	500	0.00	0.00
Slope (%) :	1	0.00	0.00
Surface Type :	Unpaved	Unpaved	Unpaved
Velocity (ft/sec) :	1.61	0.00	0.00
Computed Flow Time (min) :	5.18	0.00	0.00

Channel Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.1	0.00	0.00
Flow Length (ft) :	1700	0.00	0.00
Channel Slope (%) :	1	0.00	0.00
Cross Section Area (ft²) :	3	0.00	0.00
Wetted Perimeter (ft) :	7	0.00	0.00
Velocity (ft/sec) :	0.85	0.00	0.00
Computed Flow Time (min) :	33.45	0.00	0.00

Total TOC (min) 54.08

Subbasin Runoff Results

Total Rainfall (in) 4.48
Total Runoff (in) 2.19
Peak Runoff (cfs) 183.00
Weighted Curve Number 77.00
Time of Concentration (days hh:mm:ss) 0 00:54:05

Subbasin : Runoff-East

Input Data

Area (ac) 57.16
Weighted Curve Number 77.00
Rain Gage ID .... LL-PFtable

Composite Curve Number

Soil/Surface Description	Area (acres)	Soil Group	Curve Number
Desert shrub range, Poor	109.15	B	77.00
Composite Area & Weighted CN	109.15		77.00

Time of Concentration

Sheet Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.15	0.00	0.00
Flow Length (ft) :	100	0.00	0.00
Slope (%) :	.8	0.00	0.00
2 yr, 24 hr Rainfall (in) :	2.24	0.00	0.00
Velocity (ft/sec) :	0.10	0.00	0.00
Computed Flow Time (min) :	16.90	0.00	0.00

Shallow Concentrated Flow Computations	Subarea A	Subarea B	Subarea C
Flow Length (ft) :	1200	0.00	0.00
Slope (%) :	.8	0.00	0.00
Surface Type :	Unpaved	Unpaved	Unpaved
Velocity (ft/sec) :	1.44	0.00	0.00
Computed Flow Time (min) :	13.89	0.00	0.00

Channel Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.1	0.00	0.00
Flow Length (ft) :	1800	0.00	0.00
Channel Slope (%) :	1	0.00	0.00
Cross Section Area (ft²) :	2	0.00	0.00
Wetted Perimeter (ft) :	5	0.00	0.00
Velocity (ft/sec) :	0.81	0.00	0.00
Computed Flow Time (min) :	37.09	0.00	0.00

Total TOC (min) 67.87

Subbasin Runoff Results

Total Rainfall (in) 4.48
Total Runoff (in) 2.19
Peak Runoff (cfs) 82.47
Weighted Curve Number 77.00
Time of Concentration (days hh:mm:ss) 0 01:07:52

Subbasin : Runoff-North

Input Data

Area (ac) 158.86
Weighted Curve Number 77.00
Rain Gage ID .... LL-PFtable

Composite Curve Number

Soil/Surface Description	Area (acres)	Soil Group	Curve Number
Desert shrub range, Poor	158.86	B	77.00
Composite Area & Weighted CN	158.86		77.00

Time of Concentration

Sheet Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.15	0.00	0.00
Flow Length (ft) :	100	0.00	0.00
Slope (%) :	.5	0.00	0.00
2 yr, 24 hr Rainfall (in) :	2.24	0.00	0.00
Velocity (ft/sec) :	0.08	0.00	0.00
Computed Flow Time (min) :	20.39	0.00	0.00

Shallow Concentrated Flow Computations	Subarea A	Subarea B	Subarea C
Flow Length (ft) :	4000	0.00	0.00
Slope (%) :	.5	0.00	0.00
Surface Type :	Unpaved	Unpaved	Unpaved
Velocity (ft/sec) :	1.14	0.00	0.00
Computed Flow Time (min) :	58.48	0.00	0.00

Total TOC (min) 78.87

Subbasin Runoff Results

Total Rainfall (in) 4.48
Total Runoff (in) 2.19
Peak Runoff (cfs) 200.79
Weighted Curve Number 77.00
Time of Concentration (days hh:mm:ss) 0 01:18:52

Subbasin : Runoff-West

Input Data

Area (ac) 115.39
Weighted Curve Number 77.00
Rain Gage ID .... LL-PFtable

Composite Curve Number

Soil/Surface Description	Area (acres)	Soil Group	Curve Number
Desert shrub range, Poor	115.39	B	77.00
Composite Area & Weighted CN	115.39		77.00

Time of Concentration

Sheet Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.15	0.00	0.00
Flow Length (ft) :	100	0.00	0.00
Slope (%) :	1.4	0.00	0.00
2 yr, 24 hr Rainfall (in) :	2.24	0.00	0.00
Velocity (ft/sec) :	0.12	0.00	0.00
Computed Flow Time (min) :	13.51	0.00	0.00

Shallow Concentrated Flow Computations	Subarea A	Subarea B	Subarea C
Flow Length (ft) :	2000	0.00	0.00
Slope (%) :	1.38	0.00	0.00
Surface Type :	Unpaved	Unpaved	Unpaved
Velocity (ft/sec) :	1.90	0.00	0.00
Computed Flow Time (min) :	17.54	0.00	0.00

Channel Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.1	0.00	0.00
Flow Length (ft) :	2400	0.00	0.00
Channel Slope (%) :	1.38	0.00	0.00
Cross Section Area (ft²) :	3	0.00	0.00
Wetted Perimeter (ft) :	7	0.00	0.00
Velocity (ft/sec) :	0.99	0.00	0.00
Computed Flow Time (min) :	40.20	0.00	0.00

Total TOC (min) 71.25

Subbasin Runoff Results

Total Rainfall (in) 4.48
Total Runoff (in) 2.19
Peak Runoff (cfs) 159.59
Weighted Curve Number 77.00
Time of Concentration (days hh:mm:ss) 0 01:11:15

Subbasin : RunOn-South

Input Data

Area (ac) 94.60
Weighted Curve Number 77.00
Rain Gage ID .... LL-PFtable

Composite Curve Number

Soil/Surface Description	Area (acres)	Soil Group	Curve Number
Desert shrub range, Poor	100.32	B	77.00
Composite Area & Weighted CN	100.32		77.00

Time of Concentration

Sheet Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.15	0.00	0.00
Flow Length (ft) :	100	0.00	0.00
Slope (%) :	1.5	0.00	0.00
2 yr, 24 hr Rainfall (in) :	2.24	0.00	0.00
Velocity (ft/sec) :	0.13	0.00	0.00
Computed Flow Time (min) :	13.14	0.00	0.00

Shallow Concentrated Flow Computations	Subarea A	Subarea B	Subarea C
Flow Length (ft) :	3400	0.00	0.00
Slope (%) :	1.5	0.00	0.00
Surface Type :	Unpaved	Unpaved	Unpaved
Velocity (ft/sec) :	1.98	0.00	0.00
Computed Flow Time (min) :	28.62	0.00	0.00

Channel Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.1	0.00	0.00
Flow Length (ft) :	5200	0.00	0.00
Channel Slope (%) :	.77	0.00	0.00
Cross Section Area (ft²) :	5	0.00	0.00
Wetted Perimeter (ft) :	11	0.00	0.00
Velocity (ft/sec) :	0.77	0.00	0.00
Computed Flow Time (min) :	112.12	0.00	0.00

Total TOC (min) 153.88

Subbasin Runoff Results

Total Rainfall (in) 4.48
Total Runoff (in) 2.19
Peak Runoff (cfs) 65.68
Weighted Curve Number 77.00
Time of Concentration (days hh:mm:ss) 0 02:33:53

Subbasin : RunOn-SouthEast

Input Data

Area (ac) 75.01
Weighted Curve Number 77.00
Rain Gage ID .... LL-PFtable

Composite Curve Number

Soil/Surface Description	Area (acres)	Soil Group	Curve Number
Desert shrub range, Poor	75.01	B	77.00
Composite Area & Weighted CN	75.01		77.00

Time of Concentration

Sheet Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.15	0.00	0.00
Flow Length (ft) :	100	0.00	0.00
Slope (%) :	1.9	0.00	0.00
2 yr, 24 hr Rainfall (in) :	2.24	0.00	0.00
Velocity (ft/sec) :	0.14	0.00	0.00
Computed Flow Time (min) :	11.95	0.00	0.00

Shallow Concentrated Flow Computations	Subarea A	Subarea B	Subarea C
Flow Length (ft) :	2300	0.00	0.00
Slope (%) :	1.9	0.00	0.00
Surface Type :	Unpaved	Unpaved	Unpaved
Velocity (ft/sec) :	2.22	0.00	0.00
Computed Flow Time (min) :	17.27	0.00	0.00

Channel Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.2	0.00	0.00
Flow Length (ft) :	4600	0.00	0.00
Channel Slope (%) :	.5	0.00	0.00
Cross Section Area (ft²) :	4	0.00	0.00
Wetted Perimeter (ft) :	9	0.00	0.00
Velocity (ft/sec) :	0.31	0.00	0.00
Computed Flow Time (min) :	249.89	0.00	0.00

Total TOC (min) 279.11

Subbasin Runoff Results

Total Rainfall (in) 4.48
Total Runoff (in) 2.19
Peak Runoff (cfs) 30.52
Weighted Curve Number 77.00
Time of Concentration (days hh:mm:ss) 0 04:39:07

Subbasin : RunOn-SouthWest

Input Data

Area (ac) 43.95
Weighted Curve Number 77.00
Rain Gage ID .... LL-PFtable

Composite Curve Number

Soil/Surface Description	Area (acres)	Soil Group	Curve Number
Desert shrub range, Poor	54.50	B	77.00
Composite Area & Weighted CN	54.50		77.00

Time of Concentration

Sheet Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.15	0.00	0.00
Flow Length (ft) :	100	0.00	0.00
Slope (%) :	1.35	0.00	0.00
2 yr, 24 hr Rainfall (in) :	2.24	0.00	0.00
Velocity (ft/sec) :	0.12	0.00	0.00
Computed Flow Time (min) :	13.70	0.00	0.00

Shallow Concentrated Flow Computations	Subarea A	Subarea B	Subarea C
Flow Length (ft) :	1000	0.00	0.00
Slope (%) :	1.35	0.00	0.00
Surface Type :	Unpaved	Unpaved	Unpaved
Velocity (ft/sec) :	1.87	0.00	0.00
Computed Flow Time (min) :	8.91	0.00	0.00

Channel Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.1	0.00	0.00
Flow Length (ft) :	2400	0.00	0.00
Channel Slope (%) :	1.35	0.00	0.00
Cross Section Area (ft^2) :	5	0.00	0.00
Wetted Perimeter (ft) :	11	0.00	0.00
Velocity (ft/sec) :	1.02	0.00	0.00
Computed Flow Time (min) :	39.08	0.00	0.00

Total TOC (min) 61.70

Subbasin Runoff Results

Total Rainfall (in) 4.48
Total Runoff (in) 2.19
Peak Runoff (cfs) 69.02
Weighted Curve Number 77.00
Time of Concentration (days hh:mm:ss) 0 01:01:42

Subbasin : RunOn-West

Input Data

Area (ac) 31.10
Weighted Curve Number 77.00
Rain Gage ID .... LL-PFtable

Composite Curve Number

Soil/Surface Description	Area (acres)	Soil Group	Curve Number
Desert shrub range, Poor	34.80	B	77.00
Composite Area & Weighted CN	34.80		77.00

Time of Concentration

Sheet Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.15	0.00	0.00
Flow Length (ft) :	100	0.00	0.00
Slope (%) :	1.85	0.00	0.00
2 yr, 24 hr Rainfall (in) :	2.24	0.00	0.00
Velocity (ft/sec) :	0.14	0.00	0.00
Computed Flow Time (min) :	12.08	0.00	0.00

Shallow Concentrated Flow Computations	Subarea A	Subarea B	Subarea C
Flow Length (ft) :	3000	0.00	0.00
Slope (%) :	1.85	0.00	0.00
Surface Type :	Unpaved	Unpaved	Unpaved
Velocity (ft/sec) :	2.19	0.00	0.00
Computed Flow Time (min) :	22.83	0.00	0.00

Total TOC (min) 34.91

Subbasin Runoff Results

Total Rainfall (in) 4.48
Total Runoff (in) 2.19
Peak Runoff (cfs) 78.93
Weighted Curve Number 77.00
Time of Concentration (days hh:mm:ss) 0 00:34:55

Junction Input

SN	Element ID	Invert Elevation (ft)	Ground/Rim (Max) Elevation (ft)	Ground/Rim (Max) Offset (ft)	Initial Water Elevation (ft)	Initial Water Depth (ft)	Surcharge Elevation (ft)	Surcharge Depth (ft)	Ponded Area (ft²)	Minimum Pipe Cover (in)
1	E_BdryBasinOut	3519.00	3522.00	3.00	3519.00	0.00	3522.00	0.00	0.00	0.00
2	HwyClvt-Cen-Out	3520.00	3523.00	3.00	3520.00	0.00	3523.00	0.00	0.00	0.00
3	HwyClvt-E-Out	3509.00	3512.00	3.00	3509.00	0.00	3512.00	0.00	0.00	0.00
4	NWBasinIn	3513.00	3516.00	3.00	3513.00	0.00	3516.00	0.00	50000.00	0.00
5	RR-Culv-N-Out	3555.50	3557.50	2.00	3555.50	0.00	3557.50	0.00	0.00	0.00
6	RR-Culv-S-Out	3565.50	3567.50	2.00	2565.50	-1000.00	3567.50	0.00	0.00	0.00
7	S_Drn-E-SCnr	3536.00	3539.00	3.00	3535.00	-1.00	3539.00	0.00	0.00	0.00
8	S_Drn-NE_CNR	3514.40	3517.40	3.00	3514.40	0.00	3517.40	0.00	0.00	0.00
9	S_Drn-S-ECnr	3524.00	3527.00	3.00	3524.00	0.00	3527.00	0.00	0.00	0.00
10	S_Drn-W-S_Start	3556.00	3559.00	3.00	3556.00	0.00	3559.00	0.00	0.00	0.00

Junction Results

SN	Element ID	Peak Inflow (cfs)	Peak Lateral Inflow (cfs)	Max HGL Elevation Attained (ft)	Max HGL Depth Attained (ft)	Max Surcharge Depth Attained (ft)	Min Freeboard Attained (ft)	Average HGL Elevation Attained (ft)	Average HGL Depth Attained (ft)	Time of Max HGL Occurrence (days hh:mm)	Time of Peak Flooding Occurrence (days hh:mm)	Total Flooded Volume (ac-in)	Total Time Flooded (min)
1	E_BdryBasinOut	107.67	0.00	3520.53	1.53	0.00	4.47	3519.45	0.45	0 06:58	0 00:00	0.00	0.00
2	HwyClvt-Cen-Out	33.63	0.00	3520.44	0.44	0.00	4.06	3520.10	0.10	0 06:47	0 00:00	0.00	0.00
3	HwyClvt-E-Out	164.41	0.00	3515.61	6.61	0.00	0.89	3514.82	5.82	0 07:48	0 00:00	0.00	0.00
4	NWBasinIn	338.96	200.59	3515.39	2.39	0.00	0.61	3514.34	1.34	0 07:00	0 00:00	0.00	0.00
5	RR-Culv-N-Out	15.08	0.00	3555.91	0.41	0.00	2.59	3555.64	0.14	0 07:15	0 00:00	0.00	0.00
6	RR-Culv-S-Out	7.54	0.00	3565.86	0.36	0.00	2.64	3565.63	0.13	0 06:28	0 00:00	0.00	0.00
7	S_Drn-E-SCnr	86.78	30.46	3537.36	1.36	0.00	1.64	3536.38	0.38	0 07:52	0 00:00	0.00	0.00
8	S_Drn-NE_CNR	107.63	0.00	3515.93	1.53	0.00	1.47	3514.85	0.45	0 07:02	0 00:00	0.00	0.00
9	S_Drn-S-ECnr	86.73	0.00	3525.36	1.36	0.00	1.64	3524.38	0.38	0 07:56	0 00:00	0.00	0.00
10	S_Drn-W-S_Start	65.42	65.42	3557.16	1.16	0.00	1.84	3556.26	0.26	0 07:40	0 00:00	0.00	0.00

Channel Input

SN	Element ID	Length (ft)	Inlet Invert Elevation (ft)	Inlet Invert Offset (ft)	Outlet Invert Elevation (ft)	Outlet Invert Offset (ft)	Total Drop (ft)	Average Slope (%)	Shape	Height (ft)	Width (ft)	Manning's Roughness	Entrance Losses	Exit/Bend Losses	Additional Losses	Initial Flow (cfs)	Flap Gate
1	E-Prop-Nhalf	1499.64	3517.00	-2.00	3514.40	0.00	2.60	0.1700	Trapezoidal	3.000	20.000	0.0360	0.5000	0.5000	0.0000	0.00	No
2	E-Prop-Shalf	1284.15	3524.00	0.00	3520.00	3.00	4.00	0.3100	Trapezoidal	3.000	20.000	0.0360	0.5000	0.5000	0.0000	0.00	No
3	HwyClvtC-Out_NWcollctn	2194.11	3520.00	0.00	3514.00	1.00	6.00	0.2700	Trapezoidal	2.000	32.000	0.0320	0.5000	0.5000	0.0000	0.00	No
4	HwyClvtE-Out_NWcollctn	3882.48	3514.50	5.50	3513.00	0.00	1.50	0.0400	Trapezoidal	2.000	32.000	0.0320	0.5000	0.5000	0.0000	0.00	No
5	HwyWtoCen	2375.24	3555.50	0.00	3519.00	0.00	36.50	1.5400	Trapezoidal	1.000	14.000	0.0320	0.5000	0.5000	0.0000	0.00	No
6	NEcnr	905.56	3514.40	0.00	3513.00	3.00	1.40	0.1500	Trapezoidal	3.000	20.000	0.0320	0.5000	0.5000	0.0000	0.00	No
7	NWcollctn_NWbsn	5342.82	3514.00	1.00	3498.00	0.00	16.00	0.3000	Trapezoidal	2.000	34.000	0.0320	0.5000	0.5000	0.0000	0.00	No
8	SEcnr	1816.82	3536.00	0.00	3524.00	0.00	12.00	0.6600	Trapezoidal	3.000	20.000	0.0360	0.5000	0.5000	0.0000	0.00	No
9	S-Prop	1421.18	3556.00	0.00	3536.00	0.00	20.00	1.4100	Trapezoidal	3.000	20.000	0.0360	0.5000	0.5000	0.0000	0.00	No
10	W-Prop-LDAedge	863.84	3565.50	0.00	3555.50	0.00	10.00	1.1600	Trapezoidal	1.000	10.000	0.0320	0.5000	0.5000	0.0000	0.00	No

Channel Results

SN	Element ID	Peak Flow (cfs)	Time of Peak Flow Occurrence (days hh:mm)	Design Flow Capacity (cfs)	Peak Flow/ Design Flow Ratio	Peak Flow Velocity (ft/sec)	Travel Time (min)	Peak Flow Depth (ft)	Peak Flow Depth/ Total Depth Ratio	Total Time Surcharged (min)	Froude Number	Reported Condition
1	E-Prop-Nhalf	107.63	0 07:02	384.13	0.28	6.38	3.92	1.53	0.51	0.00		
2	E-Prop-Shalf	86.72	0 07:58	384.13	0.23	5.97	3.59	1.36	0.45	0.00		
3	HwyClvtC-Out_NWcollctn	33.26	0 06:54	465.74	0.07	3.64	10.05	0.43	0.22	0.00		
4	HwyClvtE-Out_NWcollctn	164.32	0 07:55	465.74	0.35	6.38	10.14	1.11	0.55	0.00		
5	HwyWtoCen	15.13	0 07:20	69.56	0.22	3.45	11.47	0.41	0.41	0.00		
6	NEcnr	107.61	0 07:03	432.15	0.25	6.92	2.18	1.43	0.48	0.00		
7	NWcollctn_NWbsn	336.84	0 07:07	620.76	0.54	7.82	11.39	1.38	0.69	0.00		
8	SEcnr	86.73	0 07:56	384.13	0.23	5.97	5.07	1.36	0.45	0.00		
9	S-Prop	65.42	0 07:41	384.13	0.17	5.48	4.32	1.16	0.39	0.00		
10	W-Prop-LDAedge	7.54	0 08:09	43.90	0.17	3.09	4.66	0.36	0.36	0.00		

Storage Nodes

Storage Node : ContactWtrStrg

Input Data

Invert Elevation (ft) 3526.00
Max (Rim) Elevation (ft) 3536.00
Max (Rim) Offset (ft) 10.00
Initial Water Elevation (ft) 3524.00
Initial Water Depth (ft) .... -2.00
Ponded Area (ft²) 152460.00
Evaporation Loss 0.00

Output Summary Results

Peak Inflow (cfs) 159.21
Peak Lateral Inflow (cfs) 159.21
Peak Outflow (cfs) 0.00
Peak Exfiltration Flow Rate (cfm) 0.00
Max HGL Elevation Attained (ft) 3526.00
Max HGL Depth Attained (ft) 0
Average HGL Elevation Attained (ft) 3526.00
Average HGL Depth Attained (ft) 0
Time of Max HGL Occurrence (days hh:mm) 0 00:00
Total Exfiltration Volume (1000-ft³) 0.000
Total Flooded Volume (ac-in) 0
Total Time Flooded (min) 0
Total Retention Time (sec) 0.00

Storage Node : E_Bdry-Mid

Input Data

Invert Elevation (ft) 3517.00
Max (Rim) Elevation (ft) 3523.00
Max (Rim) Offset (ft) 6.00
Initial Water Elevation (ft) 3517.00
Initial Water Depth (ft) 0.00
Ponded Area (ft²) 500.00
Evaporation Loss 0.00

Outflow Weirs

SN	Element ID	Weir Type	Flap Gate	Crest Elevation (ft)	Crest Offset (ft)	Length (ft)	Weir Total Height (ft)	Discharge Coefficient
1	Link-17	Trapezoidal	No	3520.00	3.00	8.00	3.00	3.33

Output Summary Results

Peak Inflow (cfs) 107.82
Peak Lateral Inflow (cfs) 82.24
Peak Outflow (cfs) 107.67
Peak Exfiltration Flow Rate (cfm) 0.00
Max HGL Elevation Attained (ft) 3522.03
Max HGL Depth Attained (ft) 5.03
Average HGL Elevation Attained (ft) 3519.82
Average HGL Depth Attained (ft) 2.82
Time of Max HGL Occurrence (days hh:mm) 0 06:58
Total Exfiltration Volume (1000-ft³) 0.000
Total Flooded Volume (ac-in) 0
Total Time Flooded (min) 0
Total Retention Time (sec) 0.00

Storage Node : Hwy-CenEst

Input Data

Invert Elevation (ft) 3513.00
Max (Rim) Elevation (ft) 3519.00
Max (Rim) Offset (ft) 6.00
Initial Water Elevation (ft) 3513.00
Initial Water Depth (ft) 0.00
Ponded Area (ft²) 6000.00
Evaporation Loss 0.00

Outflow Weirs

SN	Element ID	Weir Type	Flap Gate	Crest Elevation (ft)	Crest Offset (ft)	Length (ft)	Weir Total Height (ft)	Discharge Coefficient
1	MidBasinToEast	Trapezoidal	No	3517.00	4.00	12.00	1.00	3.33

Output Summary Results

Peak Inflow (cfs) 182.18
Peak Lateral Inflow (cfs) 182.18
Peak Outflow (cfs) 70.36
Peak Exfiltration Flow Rate (cfm) 0.00
Max HGL Elevation Attained (ft) 3518.87
Max HGL Depth Attained (ft) 5.87
Average HGL Elevation Attained (ft) 3516.24
Average HGL Depth Attained (ft) 3.24
Time of Max HGL Occurrence (days hh:mm) 0 07:08
Total Exfiltration Volume (1000-ft³) 0.000
Total Flooded Volume (ac-in) 0
Total Time Flooded (min) 0
Total Retention Time (sec) 0.00

Storage Node : HwyClvt-Cen

Input Data

Invert Elevation (ft)	3519.00
Max (Rim) Elevation (ft)	3526.00
Max (Rim) Offset (ft)	7.00
Initial Water Elevation (ft)	3519.00
Initial Water Depth (ft)	0.00
Ponded Area (ft²)	2000.00
Evaporation Loss	0.00

Output Summary Results

Peak Inflow (cfs)	36.74
Peak Lateral Inflow (cfs)	21.98
Peak Outflow (cfs)	33.63
Peak Exfiltration Flow Rate (cfm)	0.00
Max HGL Elevation Attained (ft)	3523.06
Max HGL Depth Attained (ft)	4.06
Average HGL Elevation Attained (ft)	3520.64
Average HGL Depth Attained (ft)	1.64
Time of Max HGL Occurrence (days hh:mm)	0 06:47
Total Exfiltration Volume (1000-ft³)	0.000
Total Flooded Volume (ac-in)	0
Total Time Flooded (min)	0
Total Retention Time (sec)	0.00

Storage Node : HwyClvt-E

Input Data

Invert Elevation (ft)	3510.00
Max (Rim) Elevation (ft)	3515.00
Max (Rim) Offset (ft)	5.00
Initial Water Elevation (ft)	3510.00
Initial Water Depth (ft)	0.00
Ponded Area (ft²)	1000.00
Evaporation Loss	0.00

Output Summary Results

Peak Inflow (cfs)	183.19
Peak Lateral Inflow (cfs)	90.80
Peak Outflow (cfs)	164.41
Peak Exfiltration Flow Rate (cfm)	0.00
Max HGL Elevation Attained (ft)	3514.90
Max HGL Depth Attained (ft)	4.9
Average HGL Elevation Attained (ft)	3511.77
Average HGL Depth Attained (ft)	1.77
Time of Max HGL Occurrence (days hh:mm)	0 07:48
Total Exfiltration Volume (1000-ft³)	0.000
Total Flooded Volume (ac-in)	0
Total Time Flooded (min)	0
Total Retention Time (sec)	0.00

Storage Node : NWBasin

Input Data

Invert Elevation (ft) 3498.00
Max (Rim) Elevation (ft) 3508.00
Max (Rim) Offset (ft) 10.00
Initial Water Elevation (ft) 3498.00
Initial Water Depth (ft) 0.00
Ponded Area (ft^2) 1800000.00
Evaporation Loss 0.00

Output Summary Results

Peak Inflow (cfs) 336.84
Peak Lateral Inflow (cfs) 0.00
Peak Outflow (cfs) 0.00
Peak Exfiltration Flow Rate (cfm) 0.00
Max HGL Elevation Attained (ft) 3498.00
Max HGL Depth Attained (ft) 0
Average HGL Elevation Attained (ft) 3498.00
Average HGL Depth Attained (ft) 0
Time of Max HGL Occurrence (days hh:mm) 0 00:00
Total Exfiltration Volume (1000-ft^3) 0.000
Total Flooded Volume (ac-in) 0
Total Time Flooded (min) 0
Total Retention Time (sec) 0.00

Storage Node : ProcStrg

Input Data

Invert Elevation (ft)	3526.00
Max (Rim) Elevation (ft)	3529.00
Max (Rim) Offset (ft)	3.00
Initial Water Elevation (ft)	3526.00
Initial Water Depth (ft)	0.00
Ponded Area (ft²)	1000.00
Evaporation Loss	0.00

Output Summary Results

Peak Inflow (cfs)	53.99
Peak Lateral Inflow (cfs)	53.99
Peak Outflow (cfs)	0.00
Peak Exfiltration Flow Rate (cfm)	0.00
Max HGL Elevation Attained (ft)	3526.00
Max HGL Depth Attained (ft)	0
Average HGL Elevation Attained (ft)	3526.00
Average HGL Depth Attained (ft)	0
Time of Max HGL Occurrence (days hh:mm)	0 00:00
Total Exfiltration Volume (1000-ft³)	0.000
Total Flooded Volume (ac-in)	0
Total Time Flooded (min)	0
Total Retention Time (sec)	0.00

Storage Node : RRClvt-N

Input Data

Invert Elevation (ft) 3554.00
Max (Rim) Elevation (ft) 3557.00
Max (Rim) Offset (ft) 3.00
Initial Water Elevation (ft) 3554.00
Initial Water Depth (ft) 0.00
Ponded Area (ft²) 1000.00
Evaporation Loss 0.00

Output Summary Results

Peak Inflow (cfs) 77.84
Peak Lateral Inflow (cfs) 77.84
Peak Outflow (cfs) 7.54
Peak Exfiltration Flow Rate (cfm) 0.00
Max HGL Elevation Attained (ft) 3557.00
Max HGL Depth Attained (ft) 3
Average HGL Elevation Attained (ft) 3555.52
Average HGL Depth Attained (ft) 1.52
Time of Max HGL Occurrence (days hh:mm) 0 06:26
Total Exfiltration Volume (1000-ft³) 0.000
Total Flooded Volume (ac-in) 34.62
Total Time Flooded (min) 69
Total Retention Time (sec) 0.00

Storage Node : RRClvt-S

Input Data

Invert Elevation (ft) 3564.00
Max (Rim) Elevation (ft) 3567.00
Max (Rim) Offset (ft) 3.00
Initial Water Elevation (ft) 3564.00
Initial Water Depth (ft) 0.00
Ponded Area (ft^2) 2500.00
Evaporation Loss 0.00

Output Summary Results

Peak Inflow (cfs) 68.71
Peak Lateral Inflow (cfs) 68.71
Peak Outflow (cfs) 7.54
Peak Exfiltration Flow Rate (cfm) 0.00
Max HGL Elevation Attained (ft) 3567.00
Max HGL Depth Attained (ft) 3
Average HGL Elevation Attained (ft) 3565.62
Average HGL Depth Attained (ft) 1.62
Time of Max HGL Occurrence (days hh:mm) 0 06:33
Total Exfiltration Volume (1000-ft^3) 0.000
Total Flooded Volume (ac-in) 50.55
Total Time Flooded (min) 114
Total Retention Time (sec) 0.00

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1.0 INTRODUCTION

Lea Land LLC (the Facility) is an existing Surface Waste Management Facility (SWMF) providing oil field waste solids (OFWS) disposal services. The existing Lea Land SWMF is subject to regulation under the New Mexico Oil and Gas Rules, specifically 19.15.9.711 and 19.15.36 NMAC, administered by the Oil Conservation Division (OCD) of the NM Energy, Minerals, and Natural Resources Department (NMEMNRD). This document is a component of the "Application for Permit Modification" that proposes continued operations of the existing approved waste disposal unit; lateral and vertical expansion of the landfill via the construction of new double-lined cells; and the addition of waste processing capabilities. The proposed Facility is designed in compliance with 19.15.36 NMAC and will be constructed and operated in compliance with a Surface Waste Management Facility Permit issued by the OCD. The Facility is owned by, and will be constructed and operated by, Lea Land LLC.

The Lea Land SWMF is one of the most recently designed facilities to meet the new more stringent standards that, for instance, mandate double liners and leak detection for land disposal. The new services that Lea Land will provide needed resources to fill an existing void in the market for technologies that exceed current OCD requirements.

1.1 Site Location

The Lea Land site is located approximately 27 miles northeast of Carlsbad, straddling US Highway 62-180 (Highway 62) in Lea County, NM. The Lea Land site is comprised of a 642-acre ± tract of land encompassing Section 32, Township 20 South, Range 32 East, Lea County, NM. Site access is currently provided on the south side of US Highway 62. The coordinates for the approximate center of the Lea Land site are Latitude 32°31'46.77" and Longitude -103°47'18.25".

1.2 Facility Description

The Lea Land SWMF comprises approximately 463 acres ± of the 642-acre ± site and will include two main components: an oil field waste Processing Area and an oil field waste solids Landfill, as well as related infrastructure (i.e., access, waste receiving, stormwater management, etc.). Oil field wastes are delivered to the Lea Land SWMF from oil and gas exploration and production operations in southeastern NM and west Texas. The Permit Plans (**Attachment III.1.A**) identify the locations of the Processing Area and Landfill Disposal facilities. The proposed facilities are detailed in **Table II.1.2** (**Volume II.1**) and are anticipated to be developed in four primary phases as described in **Table II.1.3** (**Volume II.1**).

2.0 DESIGN CRITERIA

An alternative design for the Lea Land SWMF landfill liner system that includes the use of on-site soils augmented by additional geosynthetics and geocomposites is proposed. In addition, an alternative design is proposed for its final cover system using on-site soils. The alternative liner and final cover are designed to meet the intent of the requirements of the New Mexico Oil Conservation Division (OCD) 19.15.36.14C NMAC, i.e., if an alternative liner design using geosynthetics or geocomposites and alternative final cover is proposed, 19.15.36.14 C(9) NMAC requires:

> *"Alternatively, the operator may propose a performance-based landfill design system using geosynthetics or geocomposites, including geogrids, geonets, geosynthetics clay liners, composite liner systems, etc., when supported by EPA's "hydrologic evaluation of landfill performance" (HELP) model or other division-approved model. The operator shall design the landfill to prevent the "bathtub effect". The bathtub effect occurs when a more permeable cover is placed over a less permeable bottom liner or natural subsoil."*

and further, 19.15.36.14F NMAC specifies that:

> *"The leachate collection and removal system protective layer and soil component of the leak detection system shall consist of soil materials that shall be free of organic matter, shall have a portion of material passing the no. 200 sieve no greater than five percent by weight and shall have a uniformity coefficient (Cu) less than 6, where Cu is defined as D60/D10. Geosynthetic materials or geocomposites including geonets and geotextiles, if used as components of the leachate collection and removal or leak detection system, shall have a hydraulic conductivity, transmissivity and chemical and physical qualities that oil field waste placement, equipment operation or leachate generation will not adversely affect. These geosynthetics or geocomposites, if used in conjunction with the soil protective cover for liners, shall have a hydraulic conductivity designed to ensure that the liner's hydraulic head never exceeds one foot."*

3.0 PURPOSE

Following several draft HELP model iterations and numerous discussions with OCD and its consultant regarding interpretation of 19.15.36 NMAC; it was determined that there are inherent conflicts between these Rules and the EPA HELP model User's Guide for Version 3 (**Attachment III.4.D**). As a result, in collaboration with OCD, a conventional HELP model approach addressing post-construction; active; closure; and post-closure operational stages was deployed. The basis of this multi-stage approach is the "Guidance Document for Performance Demonstration for an Alternate Cover Design under Section 502.A.2 of the New Mexico Solid Waste Management Regulations (20 NMAC 9.1) Using HELP Modeling; and Performance Demonstration for an Alternate Liner Design under Section 306.A.2 of the New Mexico Solid Waste Management Regulations (20 NMAC 9.1) Using HELP Modeling" (**Attachment III.4.E**, dated April 1, 1998), hereafter referred to as the "Guidance Document." The Guidance Document was used for this demonstration to the extent applicable and configured to apply to OCD Part 36 designs, i.e., substitution of Solid Waste Management prescriptive liner elements for OCD prescriptive liner elements. The Guidance Document used herein providing, for over 20 years, a subjective means to evaluate landfill liner and cover systems using very conservative assumptions.

This document presents the results of modeling conducted using the EPA's "Hydrologic Evaluation of Landfill Performance" (HELP) model; the HELP model User's Guide for Version 3 (**Attachment III.4.D**); and the Guidance Document to evaluate the proposed performance-based liner and final cover systems over the operational and post-closure life of the landfill. Also presented is a formal request for OCD approval to utilize the performance-based liner and final cover design and allow the use of alternate soil gradation specifications for soils used in construction of the protective soil layer (PSL). Laboratory analysis results of on-site soil demonstrate availability of an SC material, per the Unified Soil Classification System (USCS), i.e., a clayey sand, sand-clay mixture. The relevant engineering criteria for the SC material include a hydraulic conductivity of 3.8×10^{-3} cm/sec (80% of maximum dry density, i.e., uncompacted) or 2.0×10^{-4} cm/sec (90% of maximum dry density); with approximately 3.0 percent passing the No. 200 sieve; and having a Cu higher than the prescriptive standard. The high Cu value is attributed to the unconventional use of hydrometer fines evaluation rather than the traditional field-applied dry sieve analyses (**Attachments III.10.B** and **C**) (HELP Model Soil Texture 10 was set to 2.0×10^{-4} cm/sec).

Therefore, this document represents a formal request for OCD approval for use of alternate soil gradation specifications of no greater than 10 percent passing the No. 200 sieve and a Cu less than 30 (obtained by dry sieve) when identifying soils for construction of the Lea Land LLC protective soil layer (PSL) component of the composite liner system. In the unlikely event that on-site soils exceed these requirements, the design provides an option for a supplemental chimney drain system. Inclusion of the supplemental chimney drain for future cell development would be dependent on observed soils and analyses of available PSL materials by the design engineer at the time of cell excavation. The proposed supplemental chimney drain system is designed to be placed directly over the leachate collection system as shown on the **Permit Plans,** and to the same technical specifications utilized for leachate pipe aggregate and geotextile. The supplemental chimney drain system serves as an unencumbered leachate pathway in the event that flow through the PSL is insufficient. The proposed supplemental chimney drain system design was not included in the HELP Model in order to maintain a conservative evaluation approach.

The remainder of this document is organized as follows:

- Section 4.0 presents the methodology in this demonstration.
- Section 5.0 presents an overview of the demonstration modeling for performance-based composite liner and performance-based final cover designs.
- Section 6.0 presents a discussion of HELP model demonstration analyses for the:
  - Prescriptive liner system (Simulation #5)
  - Performance-based liner system (Simulation #6)
  - Initial start-up stage - open landfill with no waste (Simulation #7)
  - Operational stage – partially filled (Simulation #8)
  - Closure stage – closed with bare ground final cover (Simulation #9)
  - Post-Closure stage – closed with poorly vegetated final cover (Simulation #10)
- Section 7.0 presents the conclusions drawn from this demonstration modeling and the request for approval for the use of a;
  - Performance-based composite liner design;
  - Performance-based final cover design;
  - Alternative PSL soil specifications; and
  - Supplemental chimney drain system.

4.0 HELP MODEL METHODOLOGY

The methodology used to demonstrate that the performance of the alternative liner system and alternative final cover designs protect the uppermost aquifer rely on the USEPA's HELP modeling program and the Guidance Document. The demonstrations described below were performed using HELP Model, Version 3.07.

5.0 OVERVIEW OF DEMONSTRATION MODELING

Because the Lea Land LLC facility is planning to use an alternative design for its liner and final cover systems, the HELP model simulation analyses were organized to support two demonstrations. The demonstrations are referred to as "Tier 1" and "Tier 2" in the Guidance Document and are generally described as follows:

- Tier 1 – Proposed performance-based alternative liner system provides equivalent protection as the prescriptive liner system.
  - Evaluate the performance of the prescriptive liner system and the proposed alternative liner system. Demonstration is successful when the analyses shows equal or less percolation/leakage through the bottom layer of the proposed alternative liner system than the percolation rate through the prescriptive liner system. Gordon/PSC has performed the HELP model simulation analysis for Lea Land LLC facility that meets the requirements of the Guidance Document (**Attachment III.10.E**). Simulation #5 and Simulation #6 respectively are presented in Section 6.2.

- Tier 2 – Proposed liner system and proposed final cover system protect the uppermost aquifer during various operational stages.
    - o Evaluate:
        - open landfill at start-up when the landfill contains no waste (Simulation #7);
        - partially filled landfill (Simulation #8);
        - landfill in closed condition with bare ground (Simulation #9);
        - landfill in closed condition with poor vegetation (Simulation #10).
    - o Demonstration is successful when the analyses from Simulation #9 and #10 indicate no leakage. Simulations #7 through #10 are presented in Section 6.3.

6.0 HELP MODEL INPUT PARAMETERS

In each of the proposed performance-based alternative liner and final cover demonstrations, the input parameters for the HELP model have been selected utilizing guidance from the "Users Guide for Version 3" as provided by the USEPA (**Attachment III.4.D**); and the Guidance Document (**Attachment III.4.E**). Except for the prescriptive liner design provided in 19.15.36.14.C NMAC, the design parameters common to each demonstration are as follows:

6.1 Slope and Distance

Slope steepness and lateral drainage distance were derived from the design parameters for the units specific to the Lea Land Landfill. The liner system in Unit IV has the flattest floor slope (2.50% along the leachate collection header and 3.78% cross-slope leading to the leachate collection header), and the longest lateral drainage distance (1,550 feet), (**Figure III.4.1**). The top portion of the final cover system (i.e., crown), has a relatively uniform average slope of 5%; the longest lateral drainage distance (285 feet) occurs from the crown of the landfill to sideslope (**Figure III.4.2**). The landfill sideslopes are 4 horizontal to 1 vertical (4:1). Experience dictates that modeling the shallowest slopes produce the most conservative evaluation. Therefore, throughout these analyses, the following design parameters have been used:

- Liner system:
    - o lateral drainage distance = 1,550 ft
    - o slope = 2.50%
    - o liner footprint = 75.08 acres
- Final cover system:
    - o lateral drainage distance = 285 ft
    - o slope = 5%
    - o cover area = 75.08 acres

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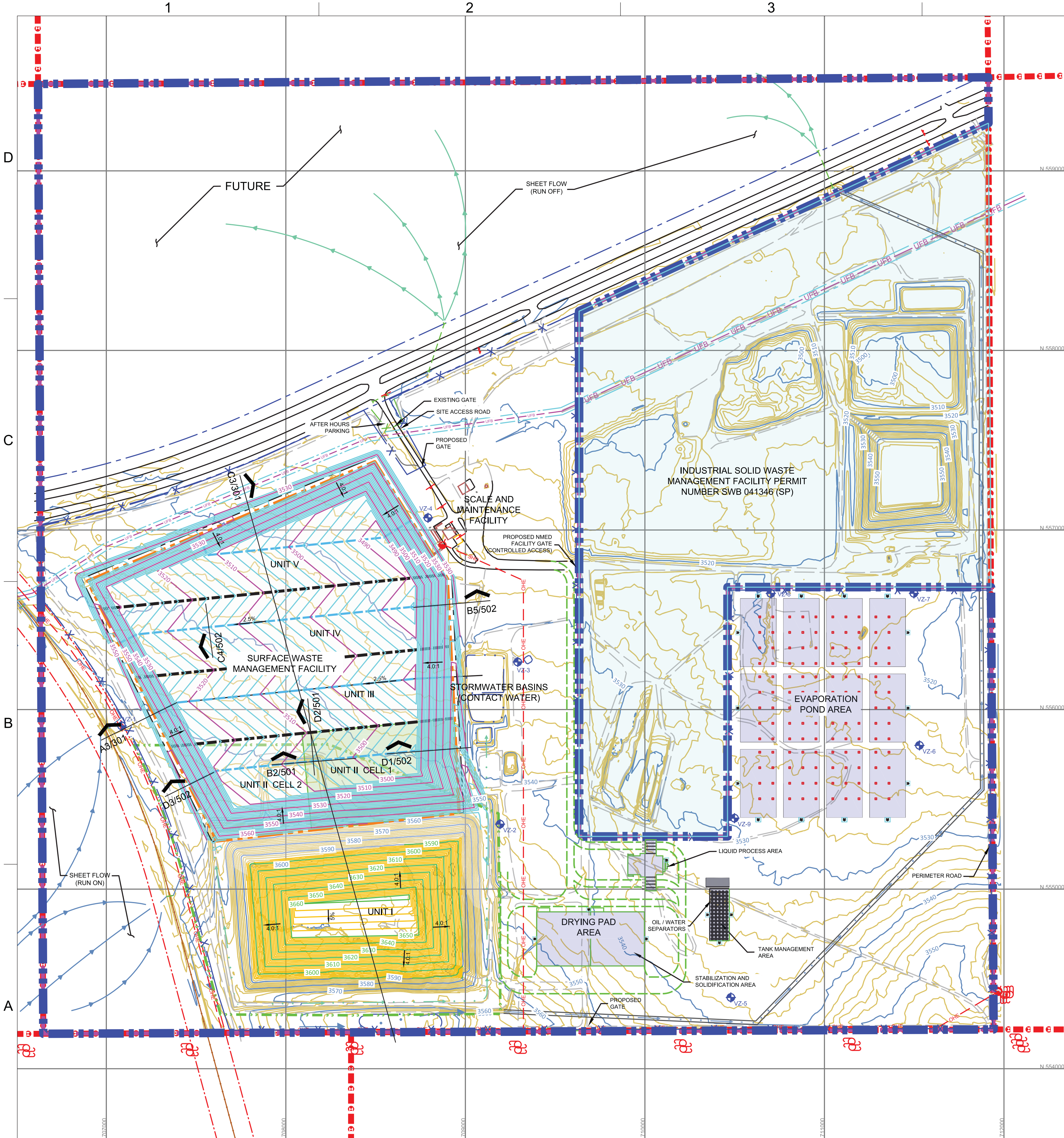

LEGEND

- SECTION BOUNDARY
- PROPERTY BOUNDARY
- EXISTING OCD PERMIT BOUNDARY
- EXISTING HIGHWAY RIGHT-OF-WAY
- EXISTING RAILWAY RIGHT-OF-WAY
- EXISTING TELECOM EASEMENT
- UFB — UFB: EXISTING UNDERGROUND FIBER-OPTIC LINE
- EXISTING EDGE OF PAVED ROADWAY
- EXISTING EDGE OF UNPAVED ROADWAY
- EXISTING CENTERLINE OF RAILWAY
- EXISTING DRAINAGE RUN-ON FLOW PATH
- EXISTING DRAINAGE RUNOFF FLOW PATH
- 3600: EXISTING GRADE ELEVATION CONTOUR - INDEX (10')
- EXISTING GRADE ELEVATION CONTOUR - INTERMEDIATE (2')
- 3600: PROPOSED BASEGRADE ELEVATION CONTOUR - INDEX (10')
- PROPOSED BASEGRADE ELEVATION CONTOUR - INTERMEDIATE (2')
- 3600: EXISTING PERMIT COMPLETION GRADE CONTOUR - INDEX (10')
- EXISTING PERMIT COMPLETION GRADE CONTOUR - INTERMEDIATE (2')
- PROPOSED NEW PERMIT LINED LANDFILL BOUNDARY
- PROPOSED NEW PERMIT UNIT BOUNDARY
- PROPOSED LEACHATE COLLECTION PIPE
- EXISTING FENCE
- PROPOSED FENCE
- NOT IN OCD PERMIT
- PROPOSED SURFACE WASTE MANAGEMENT BOUNDARY
- PROPOSED TELECOM EASEMENT
- UFB: PROPOSED UNDERGROUND FIBER-OPTIC LINE ROUTE
- OHE: PROPOSED OVERHEAD ELECTRICAL LINE
- PROPOSED EDGE OF UNPAVED ROADWAY
- PROPOSED DRAINAGE RUN-ON FLOW PATH
- PROPOSED DRAINAGE RUNOFF CHANNEL
- EXISTING CULVERT
- H2S MONITORING LOCATION
- VZ-4: PROPOSED VADOSE WELL LOCATION
- POWER POLE
- D3/502: CROSS-SECTION LOCATION
- N 5580000 / E 7080000: SITE GRID

GORDON PSC ENVIRONMENTAL

CHARLES WALTER FIEDLER NEW MEXICO 19731 06/26/2019 PROFESSIONAL ENGINEER

Charles W. Fiedler P.E.

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LEA LAND LLC SURFACE WASTE MANAGEMENT FACILITY

PERMIT APPLICATION

LEA COUNTY, NEW MEXICO

KEY PLAN

N

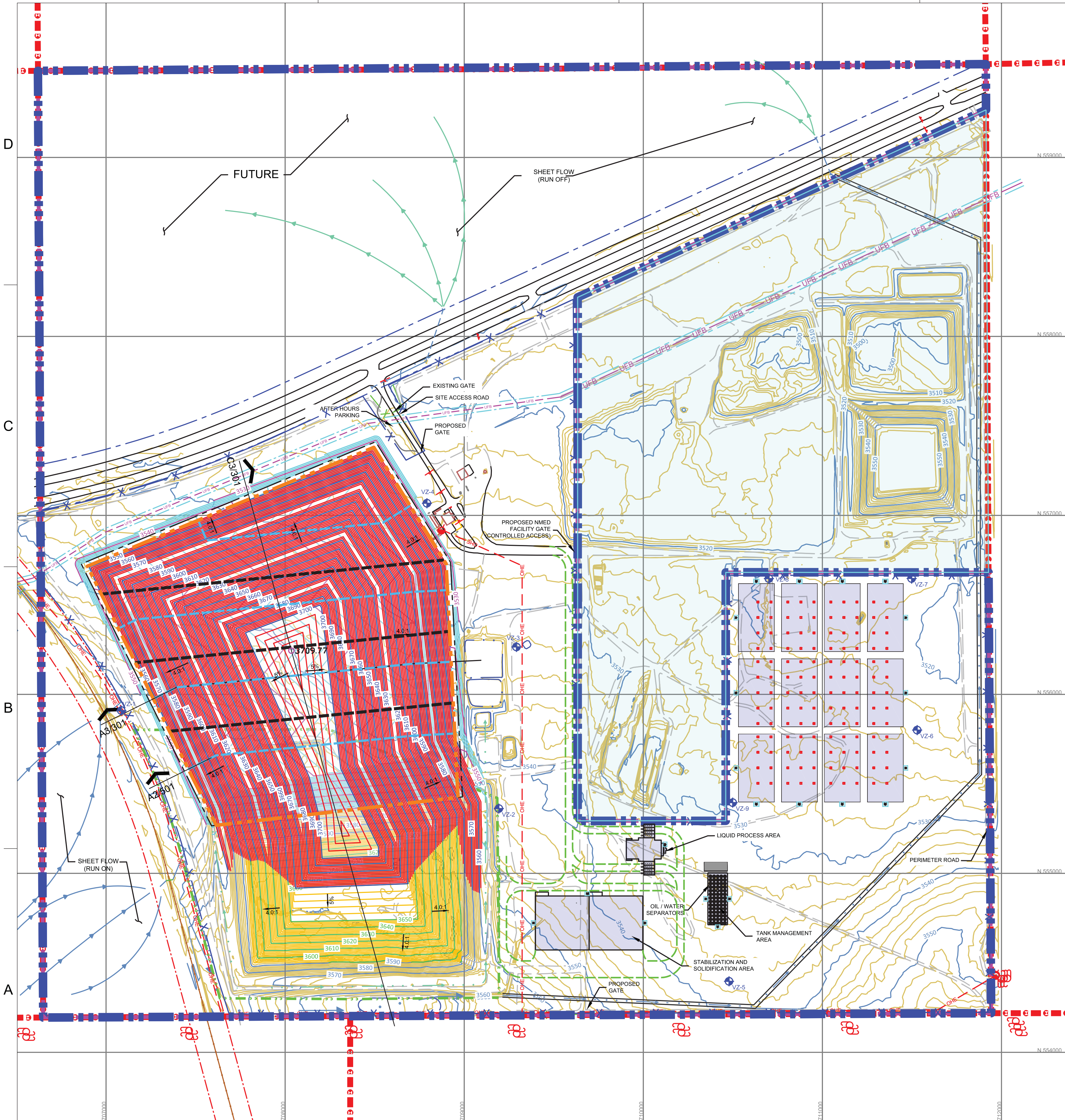
0 300' 600'

NO	DATE	DESCRIPTION
1	06/03/2019	REVISIONS PER OCD REVIEW
-	04/10/19	OCD PERMIT APPLICATION REVIEW

ISSUING OFFICE: RIO RANCHO PROJECT NO: 0416.18

BASE GRADING

FIGURE III.4.1



LEGEND

- SECTION BOUNDARY
- PROPERTY BOUNDARY
- EXISTING OCD PERMIT BOUNDARY
- EXISTING HIGHWAY RIGHT-OF-WAY
- EXISTING RAILWAY RIGHT-OF-WAY
- EXISTING TELECOM EASEMENT
- UFB — UFB — EXISTING UNDERGROUND FIBER-OPTIC LINE
- EXISTING EDGE OF PAVED ROADWAY
- EXISTING EDGE OF UNPAVED ROADWAY
- EXISTING CENTERLINE OF RAILWAY
- EXISTING DRAINAGE RUN-ON FLOW PATH
- EXISTING DRAINAGE RUNOFF FLOW PATH
- 3600 — PROPOSED FINAL GRADE ELEVATION CONTOUR - INDEX (10')
- PROPOSED FINAL GRADE ELEVATION - INTERMEDIATE (2')
- 3600 — EXISTING GRADE ELEVATION CONTOUR - INDEX (10')
- EXISTING GRADE ELEVATION CONTOUR - INTERMEDIATE (2')
- 3600 — EXISTING PERMIT COMPLETION GRADE CONTOUR - INDEX (10')
- EXISTING PERMIT COMPLETION GRADE CONTOUR - INTERMEDIATE (2')
- X — EXISTING FENCE
- X — PROPOSED FENCE
- NOT IN OCD PERMIT
- PROPOSED SURFACE WASTE MANAGEMENT BOUNDARY
- PROPOSED TELECOM EASEMENT
- UFB — PROPOSED UNDERGROUND FIBER-OPTIC LINE ROUTE
- OHE — PROPOSED OVERHEAD ELECTRICAL LINE
- PROPOSED EDGE OF UNPAVED ROADWAY
- PROPOSED DRAINAGE RUN-ON FLOW PATH
- PROPOSED DRAINAGE RUNOFF CHANNEL
- EXISTING CULVERT
- H2S MONITORING LOCATION
- VZ-4 PROPOSED VADOSE WELL LOCATION
- POWER POLE
- D3/502 CROSS-SECTION LOCATION
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LEA LAND LLC SURFACE WASTE MANAGEMENT FACILITY

PERMIT APPLICATION

LEA COUNTY, NEW MEXICO

KEY PLAN

N

0 300' 600'

NO	DATE	DESCRIPTION
1	06/03/2019	REVISIONS PER OCD REVIEW
-	04/10/19	OCD PERMIT APPLICATION REVIEW

ISSUING OFFICE: RIO RANCHO PROJECT NO: 0416.18

FINAL GRADING

FIGURE III.4.2

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6.2 Environmental

All of the simulation analyses for HELP modeling demonstrations were performed using identical environmental loading conditions. Precipitation and temperature input data were derived from the Western Regional Climatic Center's database. The nearest location with sufficient data is Carlsbad COOP, Carlsbad, New Mexico - 29149 utilizing the 5 wettest years from a minimum 40 years of available weather data (1912 – 1916). Solar radiation data was synthetically generated by the HELP model based on coefficients for Roswell, New Mexico. Roswell was used as its latitude was the closest to the site's latitude as recommended by the User's Guide for Version 3 (**Attachment III.4.D**). Evapotranspiration data (e.g., average wind speed and seasonal relative humidity) was obtained from Roswell, New Mexico as well. The evaporative zone depth was set to 14 inches except for Simulation #10, which was set to 24 inches as recommended by the Guidance Document (**Attachment III.4.E**). Maximum leaf area index was conservatively set to 0.0, i.e., bare ground. The surface layer was modeled as having no vegetation. The initial SCS Curve Number was set to the value obtained from drainage calculations (**Volume III,** Section 3). The HELP model corrected the input CN based on slope and length.

6.3 Soils

Geotechnical analysis of on-site soils indicates that the soils available at the Lea Land LLC Surface Waste Management Facility site consist primarily of clayey sand sand-clay mixture (i.e., USCS Category SC). **Attachments III.4.B and C** provides a summary of geotechnical test results. The type of soil used to represent the alternative liner protective soil layer and the alternative final cover vegetative and barrier layers in the simulations are listed below:

Soil Description	HELP Model Soil Type	USCS Soil Type
clayey sands, sand-clay mixture	10 (set to 2.0 x 10^{-4} cm/sec)	SC

The primary parameters that differentiate soils from one another are the saturated hydraulic conductivity, K_{sat}, and the moisture-retention characteristics that are related to the field capacity and the wilting point. As the HELP model soil type number increases, the saturated hydraulic conductivity decreases, and the soils tend to retain more water. Default values from the HELP model were assigned to the porosity, field capacity and wilting point for each soil type. Moisture content was initialized to be the value of the wilting point plus 25% of the difference between the wilting point and the field capacity [i.e., (field capacity – wilting point) x 0.25 + wilting point] for Simulations

5, 6, 7 and when a layer first appears in subsequent simulations. Simulations 8-10 utilize the previous simulation's Final Water Storage value by layer as the Initial Soil Water Content value for each subsequent simulation as outlined in the Guidance Document.

19.15.36.14F NMAC requires that the protective drainage layer be constructed using granular soils that contain no more than 5% fines by weight (i.e., material passing a No. 200 sieve) and that have a uniformity coefficient less than 6.0 where Cu is defined as D60/D10. As part of its design for the alternative liner system, Lea Land LLC proposes to use on-site soils for the protective soil layer that contain no more than 10% fines by weight and a uniformity coefficient less than 30 (obtained by dry sieve evaluation) in combination with an optional supplemental chimney drain system placed directly over the leachate collection system as shown on the **Permit Plans** and described in Section 3.0.

6.3 Waste Soils

The majority of the waste stream to be landfilled are oil exploration byproducts primarily comprised of contaminated soils developed from drilling operations. Classifying such soils for technical evaluation is difficult as waste soil classifications vary by regional geologic formations and drilling depths. Additionally, well developers often enhance the drilling fluids with oil or polymer-based proprietary products that lubricate and carry cuttings to the surface for separation and disposal. Nevertheless, HELP modeling relies on establishing a realistic soil texture that among other parameters, establishes a saturated hydraulic conductivity for the waste stream.

Literature searches consistently verify that the presence of crude oil in a soil matrix decreases the permeability of silty clay to clayey soil samples, i.e., the soil sample becomes more impermeable. It is reasoned that when soil particles are coated with oil, the soil particles tend to stick together, literally blocking the passage of water. The extent of permeability change is proportional to the percent of oil present, but generally is projected to account for an order of magnitude impact, i.e., for example, an uncontaminated silty clay soil sample that has a saturated hydraulic conductivity of $k=1.3x10^{-7}$ cm/sec when contaminated with less than 10% crude oil has a conservative $k=1.3x10^{-8}$ cm/sec (**Attachments III.4.F** and **III.4.G**). Assuming that the uncontaminated drilling soils have an equivalent saturated hydraulic conductivity as the alluvial soils found on-site, i.e., $k=2.0x10^{-4}$ cm/sec, a conservative assumption, the corresponding waste stream could reasonably be expected to have a hydraulic conductivity of $k=2.0x10^{-5}$ cm/sec. The corresponding HELP Model Soil Texture 22 is used throughout the applicable simulations with

Material Characteristics listed below.

The outputs from the HELP model runs, which include a listing of the input parameters, are provided as attachments to this document in both hard copy (**Attachment III.4.A**) and electronic format (**Attachment III.4.H**).

6.4 Initial Conditions

The following performance-based alternative landfill liner and final cover system component default values for HELP Model Soil Texture Classes and Material Characteristics are used in the simulations. The Simulation numbers correspond to the Initial Soil Water Content values for that Simulation.

- Vegetative Layer (Simulations 9 & 10)
  - Soil Texture 10 w/modified Saturated Hydraulic Conductivity – 0
  - Thickness – 24 inches
  - Total Porosity (vol/vol) - 0.3980
  - Field Capacity (vol/vol) - 0.2440
  - Wilting Point (vol/vol) - 0.1360
  - Initial Soil Water Content – (9) 0.1630 / (10) 0.2381
  - Saturated Hydraulic Conductivity (cm/sec) – 2.0×10^{-4}

- Barrier Layer (Simulations 9 & 10)
  - Soil Texture 10 w/modified Saturated Hydraulic Conductivity – 0
  - Thickness – 6 inches
  - Total Porosity (vol/vol) - 0.3980
  - Field Capacity (vol/vol) - 0.2440
  - Wilting Point (vol/vol) - 0.1360
  - Initial Soil Water Content – (9) 0.3980 / (10) 0.3980
  - Saturated Hydraulic Conductivity (cm/sec) – 2.0×10^{-4}

- Intermediate Cover (Simulations 8, 9 & 10)
  - Soil Texture 10 w/modified Saturated Hydraulic Conductivity – 0
  - Thickness –12 inches
  - Total Porosity (vol/vol) - 0.3980
  - Field Capacity (vol/vol) - 0.2440
  - Wilting Point (vol/vol) - 0.1360
  - Initial Soil Water Content – (8) 0.1630 / (9) 0.1871 / (10) 0.2754
  - Saturated Hydraulic Conductivity (cm/sec) – 2.0×10^{-4}

- Oil Field Waste (Simulations 8, 9 &10)
  - Soil Texture – 22
  - USCS Soil Classification - ML (compacted) - silty or clayey fine sands
  - Thickness - varies
  - Total Porosity (vol/vol) - 0.4190
  - Field Capacity (vol/vol) - 0.3070
  - Wilting Point (vol/vol) - 0.1800
  - Initial Soil Water Content – (8) 0.2117 / (9) 0.2759 / (10) 0.2780
  - Saturated Hydraulic Conductivity (cm/sec) - 1.9×10^{-5}

- Protective Soil Layer (Simulations 6-10)
  - Soil Texture – 10 w/modified Saturated Hydraulic Conductivity – 0
  - Thickness – 24 inches
  - Total Porosity (vol/vol) - 0.3980
  - Field Capacity (vol/vol) - 0.2440
  - Wilting Point (vol/vol) – 0.1360
  - Initial Soil Water Content (vol/vol) – (6) 0.1630 / (7) 0.1630 / (8) 0.2499 / (9) 0.2440 / (10) 0.2440
  - Saturated Hydraulic Conductivity (cm/sec) – 2.0×10^{-4}

- Geocomposite Drainage Layer (Simulations 6-10)
  - Modeled as 200-mil Geonet
  - Material Characteristic – 20
  - Total Porosity (vol/vol) - 0.8500
  - Field Capacity (vol/vol) - 0.0100
  - Wilting Point (vol/vol) - 0.0050
  - Initial Soil Water Content – (6) 0.0062 / (7) 0.0062 / (8) 0.1321 / (9) 0.0100 / (10) 0.0100
  - Saturated Hydraulic Conductivity (cm/sec) – 10

- Upper Liner (Simulations 6-10)
  - 60-mil HDPE
  - Material Characteristic – 35
  - Saturated Hydraulic Conductivity (cm/sec) – 2.0×10^{-13}

- Leak Detection System (Simulations 6-10)
  - 200-mil Geonet
  - Material Characteristic – 20
  - Total Porosity (vol/vol) - 0.8500

  - Field Capacity (vol/vol) - 0.0100
  - Wilting Point (vol/vol) - 0.0050
  - Initial Soil Water Content – (6) 0.0062 / (7) 0.0062 / (8) 0.1226 / (9) 0.0100 / (10) 0.0100
  - Saturated Hydraulic Conductivity (cm/sec) – 10

- Lower Liner (Simulations 6-10)
  - 60-mil HDPE
  - Material Characteristic – 35
  - Saturated Hydraulic Conductivity (cm/sec) – $2.0x10^{-13}$

- GCL (Geosynthetic Clay Liner) (Simulations 6-10)
  - Material Characteristic – 17
  - Total Porosity (vol/vol) - 0.7500
  - Field Capacity (vol/vol) - 0.7470
  - Wilting Point (vol/vol) - 0.4000
  - Initial Soil Water Content (vol/vol) – (6) 0.7500 / (7) 0.7500 / (8) 0.7500 / (9) 0.7500 / (10) 0.7500
  - Saturated Hydraulic Conductivity (cm/sec) – $3.0x10^{-9}$

7.0 HELP MODEL DEMONSTRATION ANALYSES

In the Tier I liner simulation analysis, the landfill has conservatively been assumed to be in an open condition with no waste present; and with 100% of the precipitation retained within the landfill with no runoff. The default parameters for the proposed performance-based alternative liner system are outlined in Section 6.0 with HELP model Simulation 6. Simulation 5 represents the prescriptive design outlined in 19.15.36.14C NMAC. The default parameters for the prescriptive design listed from top to bottom are as follows:

- Leachate Collection and Removal System Protection Layer
  - Soil Texture – 1
  - Thickness – 12 inches
  - Total Porosity (vol/vol) - 0.4170
  - Field Capacity (vol/vol) - 0.0450
  - Wilting Point (vol/vol) – 0.0180
  - Initial Soil Water Content (vol/vol) – 0.0247
  - Saturated Hydraulic Conductivity (cm/sec) – $1.0x10^{-2}$

- Leachate Collection and Removal System
  - Soil Texture – 1

  - Thickness – 24 inches
  - Total Porosity (vol/vol) - 0.4170
  - Field Capacity (vol/vol) - 0.0450
  - Wilting Point (vol/vol) – 0.0180
  - Initial Soil Water Content (vol/vol) – 0.0247
  - Saturated Hydraulic Conductivity (cm/sec) – 1.0×10^{-2}

Note that the Leachate Collection and Removal System Protection Layer (12 inches) and the Leachate Collection and Removal System (24 inches) were modeled as a single 36-inch-thick layer in Simulation 5.

- Upper Geomembrane Liner
  - 60-mil HDPE
  - Material Characteristic – 35 w/modified Saturated Hydraulic Conductivity – 0
  - Saturated Hydraulic Conductivity (cm/sec) – 1.0×10^{-9}

- Leak Detection System
  - Soil Texture – 6
  - Thickness – 24 inches
  - Total Porosity (vol/vol) - 0.4530
  - Field Capacity (vol/vol) - 0.1900
  - Wilting Point (vol/vol) - 0.0850
  - Initial Soil Water Content – 0.1112
  - Saturated Hydraulic Conductivity (cm/sec) – 7.2×10^{-4}

- Lower Geomembrane Liner
  - 60-mil HDPE
  - Material Characteristic – 35 w/modified Saturated Hydraulic Conductivity – 0
  - Saturated Hydraulic Conductivity (cm/sec) – 1.0×10^{-9}

- Base Layer
  - Soil Texture – 16
  - Thickness – 24 inches
  - Total Porosity (vol/vol) - 0.4270
  - Field Capacity (vol/vol) - 0.4180
  - Wilting Point (vol/vol) - 0.3670
  - Initial Soil Water Content – 0.4270
  - Saturated Hydraulic Conductivity (cm/sec) – 1.02×10^{-7}

The outputs from the HELP model runs, which include a listing of the input parameters, are provided as attachments to this document in both hard copy (**Attachment III.4.A**) and electronic format (**Attachment III.4.H**).

The Tier 1 simulation analysis (Simulation 5) is based on the Guidance Document and evaluates the performance of the prescriptive liner system prescribed in 19.15.36 NMAC. The input parameters used to represent the prescriptive liner system are provided above. The landfill was modeled as "active" with 0% of the surface area available for stormwater runoff. The performance of the prescriptive liner system is demonstrated by the 5-year average annual percolation/leakage rate through the bottom liner layer; and the head on the Upper Geomembrane Liner as calculated by the HELP model.

The Tier 1 simulation analysis (Simulation 6) is based on the Guidance Document and evaluates the performance of the proposed performance-based alternative liner system under the same conditions and calculation methods as described for the prescriptive liner above.

TABLE III.4.1 - Tier I Performance Results for Prescriptive and Alternative Liner Systems

Liner System	Simulation	Soil Type for Protective Soil Layer	Average Annual Percolation Rate Through Bottom Liner (in/yr)	Average Annual Head on HDPE Liner (in)
Prescriptive	5	1	1.00526	7.437
Proposed Alternative Liner	6	10 modified=0	0.0000	0.062

According to the Guidance Document, a successful demonstration of equivalent protection has been made when the analyses shows equal or less percolation/leakage through the bottom layer of the proposed alternative liner than the percolation/leakage through the bottom layer of the prescriptive liner. **Table III.4.1** clearly demonstrate that the proposed performance-based alternative liner system has significantly less leakage through the bottom layer of the liner system and less head on the top uppermost HDPE liner component than comparable values for the prescriptive liner system. Therefore, a successful demonstration has been made for the first Tier.

The Tier 2 simulation analysis is equally based on the Guidance Document and evaluates the proposed alternative liner and final cover design over the entire operational development of the landfill. The Guidance Document provides an automatic aid to simulating the entire operational development by requiring that each successive HELP model simulation use the previous simulation's moisture content output as the input for the following simulation. Gordon Environmental/PSC adjusted the initial soil moisture content and utilized the output of each layer as input for subsequent layers as noted in the "Initial Conditions" section.

The Guidance Document requires that four simulations encompassing the entire life cycle of the landfill to model actual design conditions and operational development as closely as possible must be performed (Simulations 7, 8, 9 & 10). This is accomplished through a succession of four model simulations: one simulation of the open landfill, a second with the landfill partially filled with oil field wastes, a third with the landfill in a closed condition with bare ground, and a fourth with the landfill in the closed condition with "poor" vegetation.

Simulations:

- Simulation 7 -The initial simulation must model the open landfill at start-up when the landfill contains no waste. The time period should extend for the anticipated duration of this condition (conservatively a minimum of two years).
- Simulation 8 – Perform a succeeding simulation to model conditions of the partially filled landfill for a five-year period including intermediate covers.
- Simulation 9 – Model the landfill in the closed condition with bare ground (a minimum of two years).
- Simulation 10 – Finally, perform a simulation to model the landfill in the closed condition with poor vegetation for the remainder of the post-closure care period (a minimum of 28 years).

The outputs from the individual HELP model runs, which include a listing of the input parameters, are provided as attachments to this document in both hard copy (**Attachment III.4.A**) and electronic format (**Attachment III.4.H**).

TABLE III.4.2 - Tier 2, Performance Results for Alterative Liner and Alternate Final Cover Systems

Simulation	HELP Model Soil Type			Average Annual Head on Top FML Layer (in/yr)	Average Annual Percolation Rate (in/yr)
	Protective Soil Layer	Vegetative (Erosion) and Intermediate Cover Layers	Barrier (Infiltration) Layer		
7	10 w/modified hydraulic conductivity=0	10 w/modified hydraulic conductivity=0	10 w/modified hydraulic conductivity=0	0.047	0.00000
8				0.000	0.00000
9				0.000	0.00000
10				0.000	0.00000

According to the Guidance Document, if simulations indicate no leakage after the third simulation (Simulation 9) and the subsequent simulation (Simulation 10), then the simulations have served to demonstrate the concentration values will not be exceeded in the uppermost aquifer at the relative point of compliance. Therefore, a successful demonstration has been made for the second tier.

8.0 CONCLUSIONS AND REQUEST FOR APPROVAL

Gordon/PSC has prepared performance demonstrations for the Lea Land LLC performance-based liner system design and for the performance-based final cover system design. These analyses were based on 19.15.36.14C(9) NMAC when supported by the HELP model and OCD division-approved use of the Guidance Document (**Attachment III.4.E**). The analyses demonstrate the following:

- In the Tier 1 performance-based alternative liner system simulation (Simulation 6) analysis, when the leachate collection and removal system protection layer (PSL) is modeled using HELP model soil type 10 with a modified hydraulic conductivity of 2.0×10^{-4} cm/sec, the average annual percolation/leakage rate calculated for the performance-based alternative liner system through the bottom layer is less than the percolation/leakage rate calculated for the prescriptive liner system (Simulation 5). Similarly, in the Tier 1 performance-based alternative liner system simulation (Simulation 6) analysis, when the leachate collection and removal system protection layer (PSL) is modeled using HELP model soil type 10 with a modified hydraulic conductivity of 2.0×10^{-4} cm/sec, the average annual head calculated for the performance-based alternative liner system on the upper FML layer is less than the average annual head calculated for the prescriptive liner system (Simulation 5). For the performance-based alternative liner simulation analysis, the average annual percolation rate calculated through the performance-based alternative liner system design is 0.0000 inches versus 1.00526 inches for the prescriptive liner design. For the performance-based alternative liner simulation analysis, the average annual head on the upper FML is 0.062 inches versus 7.437 inches for the prescriptive liner design. Therefore, for this soil type, the performance of the proposed alternative liner system design meets the Tier 1 demonstration requirements.

- In the Tier 2 simulation analyses, the complete landfill, including both the performance-based alternative liner system and the performance-based alternative final cover system designs, have been modeled. The vegetative (erosion), barrier (infiltration), intermediate cover, and leachate collection and removal system protection layer were modeled using soil type 10 with a modified hydraulic conductivity of 2.0 x 10^{-4}. In Simulation 7 – 2-year start-up with no waste, the average annual head on the upper FML layer is modeled to be 0.047 inches. The average annual head on the upper FML for Simulation 8 – 5-year partially filled; Simulation 9 – 2-year closed condition with bare soil; and Simulation 10 – 28-year closed condition with poor vegetation; is modeled to be 0.000 inches. All Tier 2 simulations predicted the average annual percolation/leakage rate for their respective durations to be 0.00000 inches. Therefore, for the soil types modeled for the vegetative (erosion), barrier (infiltration), intermediate cover, and leachate collection and removal system protection layer, the performance of the proposed alternative liner system and proposed alternative final cover system designs meets the Tier 2 demonstration requirements.

The HELP modeling for the analyses presented in this document demonstrates that the efficiency of the performance-based alternative liner system and performance-based alternative cover system designs meets the requirements of 19.15.36.14C NMAC and the Guidance Document. For the purposes of this demonstration, the performance-based alternative liner system and the performance-based alternative cover system designs have been shown to be effective using sustainable soils available on the Lea Land LLC site.

To allow Lea Land LLC flexibility in using on-site soils to construct the protective soil layer, this document serves as a request to OCD for approval to use the performance-based liner system design and to construct the protective soil layer using soils that contain less than 10% fines and has a uniformity coefficient (Cu) less than 30 (by dry sieve analyses). Additionally, Lea Land LLC is also requesting approval for a supplemental chimney drain system to be used in conjunction with the proposed alternative liner design. Inclusion of a supplemental chimney drain for future cell construction would be dependent on observed soils and analyses of available PSL materials by the design engineer.

ATTACHMENT III.4.A

HELP MODEL OUTPUT FILES

ATTACHMENT III.4.A-1

TIER 1, SIMULATION 5

PRESCRIPTIVE LINER

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******************************************************************************
******************************************************************************
**                                                                          **
**                                                                          **
**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE               **
**                HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)                **
**                  DEVELOPED BY ENVIRONMENTAL LABORATORY                   **
**                    USAE WATERWAYS EXPERIMENT STATION                     **
**             FOR USEPA RISK REDUCTION ENGINEERING LABORATORY              **
**                                                                          **
**                                                                          **
******************************************************************************
******************************************************************************


PRECIPITATION DATA FILE:     d:\lllf\DATA4.D4
TEMPERATURE DATA FILE:       D:\lllf\DATA7.D7
SOLAR RADIATION DATA FILE:   d:\lllf\DATA13.D13
EVAPOTRANSPIRATION DATA:     d:\lllf\DATA11.D11
SOIL AND DESIGN DATA FILE:   d:\lllf\SIM5.D10
OUTPUT DATA FILE:            d:\lllf\SIM5OUT.OUT


TIME:   9: 7     DATE:   7/ 5/2019


******************************************************************************

     TITLE:  Lea Land Landfill Simulation #5
          PRESCRIPTIVE LINER
******************************************************************************


     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER
              WERE SPECIFIED BY THE USER.

                                   LAYER  1
                                   --------

                       TYPE 2 - LATERAL DRAINAGE LAYER
          – LEACHATE COLLECTION AND REMOVAL SYSTEM PROTECTION LAYER
                                      &
                     LEACHATE COLLECTION AND REMOVAL SYSTEM
                         MATERIAL TEXTURE NUMBER   1
          THICKNESS                     =     36.00   INCHES
          POROSITY                      =      0.4170 VOL/VOL
          FIELD CAPACITY                =      0.0450 VOL/VOL
          WILTING POINT                 =      0.0180 VOL/VOL
          INITIAL SOIL WATER CONTENT    =      0.0247 VOL/VOL
          EFFECTIVE SAT. HYD. COND.     =  0.999999978000E-02 CM/SEC
          SLOPE                         =      2.50   PERCENT
          DRAINAGE LENGTH               =   1550.0    FEET
```

```
                        LAYER  2
                        --------

            TYPE 4 - FLEXIBLE MEMBRANE LINER – TOP HDPE
              MATERIAL TEXTURE NUMBER   0
THICKNESS                   =      0.06   INCHES
POROSITY                    =      0.0000 VOL/VOL
FIELD CAPACITY              =      0.0000 VOL/VOL
WILTING POINT               =      0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT  =      0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =  0.999999972000E-09 CM/SEC
FML PINHOLE DENSITY         =      1.00   HOLES/ACRE
FML INSTALLATION DEFECTS    =      1.00   HOLES/ACRE
FML PLACEMENT QUALITY       =  3 - GOOD

                        LAYER  3
                        --------

            TYPE 2 - LATERAL DRAINAGE LAYER – LEAK DETECTION SYSTEM
              MATERIAL TEXTURE NUMBER   6
THICKNESS                   =     24.00   INCHES
POROSITY                    =      0.4530 VOL/VOL
FIELD CAPACITY              =      0.1900 VOL/VOL
WILTING POINT               =      0.0850 VOL/VOL
INITIAL SOIL WATER CONTENT  =      0.1112 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =  0.720000011000E-03 CM/SEC
SLOPE                       =      2.50   PERCENT
DRAINAGE LENGTH             =   1550.0    FEET

                        LAYER  4
                        --------

            TYPE 4 - FLEXIBLE MEMBRANE LINER – BOTTOM HDPE
              MATERIAL TEXTURE NUMBER   0
THICKNESS                   =      0.06   INCHES
POROSITY                    =      0.0000 VOL/VOL
FIELD CAPACITY              =      0.0000 VOL/VOL
WILTING POINT               =      0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT  =      0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =  0.999999972000E-09 CM/SEC
FML PINHOLE DENSITY         =      1.00   HOLES/ACRE
FML INSTALLATION DEFECTS    =      1.00   HOLES/ACRE
FML PLACEMENT QUALITY       =  3 - GOOD
```

LAYER 5

TYPE 3 - BARRIER SOIL LINER **BASE LAYER**
MATERIAL TEXTURE NUMBER 16

THICKNESS	=	24.00	INCHES
POROSITY	=	0.4270	VOL/VOL
FIELD CAPACITY	=	0.4180	VOL/VOL
WILTING POINT	=	0.3670	VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.4270	VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.100000001000E-06	CM/SEC

GENERAL DESIGN AND EVAPORATIVE ZONE DATA

NOTE: SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM DEFAULT SOIL DATA BASE USING SOIL TEXTURE # 1 WITH BARE GROUND CONDITIONS, A SURFACE SLOPE OF 2.% AND A SLOPE LENGTH OF 1550. FEET.

SCS RUNOFF CURVE NUMBER	=	70.30	
FRACTION OF AREA ALLOWING RUNOFF	=	0.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	75.080	ACRES
EVAPORATIVE ZONE DEPTH	=	14.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	0.346	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	5.838	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	0.252	INCHES
INITIAL SNOW WATER	=	0.000	INCHES
INITIAL WATER IN LAYER MATERIALS	=	13.806	INCHES
TOTAL INITIAL WATER	=	13.806	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
ROSWELL NEW MEXICO

STATION LATITUDE	=	32.52	DEGREES
MAXIMUM LEAF AREA INDEX	=	0.00	
START OF GROWING SEASON (JULIAN DATE)	=	76	
END OF GROWING SEASON (JULIAN DATE)	=	310	
EVAPORATIVE ZONE DEPTH	=	14.0	INCHES
AVERAGE ANNUAL WIND SPEED	=	8.70	MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	49.00	%
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	40.00	%
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	53.00	%
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	52.00	%

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR ROSWELL NEW MEXICO

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
0.20	0.37	0.76	1.78	0.35	2.35
2.62	2.26	2.83	1.67	0.77	1.10

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR ROSWELL NEW MEXICO

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
45.80	48.40	53.50	62.00	71.50	78.30
80.50	78.80	71.90	62.60	54.00	43.10

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR ROSWELL NEW MEXICO
AND STATION LATITUDE = 33.24 DEGREES

****

ANNUAL TOTALS FOR YEAR 1

	INCHES	CU. FEET	PERCENT
PRECIPITATION	15.63	4259806.500	100.00
RUNOFF	0.000	0.000	0.00
EVAPOTRANSPIRATION	9.891	2695568.000	63.28
DRAINAGE COLLECTED FROM LAYER 1	0.8938	243597.578	5.72
PERC./LEAKAGE THROUGH LAYER 2	0.817694	222854.672	5.23
AVG. HEAD ON TOP OF LAYER 2	2.6665		
DRAINAGE COLLECTED FROM LAYER 3	0.0000	0.000	0.00
PERC./LEAKAGE THROUGH LAYER 5	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 4	0.0000		
CHANGE IN WATER STORAGE	4.846	1320640.120	31.00
SOIL WATER AT START OF YEAR	15.140	4126369.250	
SOIL WATER AT END OF YEAR	19.986	5447009.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.796	0.00

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ANNUAL TOTALS FOR YEAR 2

	INCHES	CU. FEET	PERCENT
PRECIPITATION	19.70	5369045.500	100.00
RUNOFF	0.000	0.000	0.00
EVAPOTRANSPIRATION	12.483	3402121.500	63.37
DRAINAGE COLLECTED FROM LAYER 1	2.5355	691037.750	12.87
PERC./LEAKAGE THROUGH LAYER 2	2.272322	619299.500	11.53
AVG. HEAD ON TOP OF LAYER 2	7.5930		
DRAINAGE COLLECTED FROM LAYER 3	0.0135	3685.597	0.07
PERC./LEAKAGE THROUGH LAYER 5	0.116741	31816.541	0.59
AVG. HEAD ON TOP OF LAYER 4	0.5597		
CHANGE IN WATER STORAGE	4.551	1240385.120	23.10
SOIL WATER AT START OF YEAR	19.986	5447009.000	
SOIL WATER AT END OF YEAR	24.537	6687394.500	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.638	0.00

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ANNUAL TOTALS FOR YEAR 3

	INCHES	CU. FEET	PERCENT
PRECIPITATION	19.61	5344517.000	100.00
RUNOFF	0.000	0.000	0.00
EVAPOTRANSPIRATION	13.583	3701910.500	69.27
DRAINAGE COLLECTED FROM LAYER 1	3.2879	896080.937	16.77
PERC./LEAKAGE THROUGH LAYER 2	2.908300	792629.375	14.83
AVG. HEAD ON TOP OF LAYER 2	9.8587		
DRAINAGE COLLECTED FROM LAYER 3	0.1430	38963.262	0.73
PERC./LEAKAGE THROUGH LAYER 5	1.232961	336031.594	6.29
AVG. HEAD ON TOP OF LAYER 4	5.9427		
CHANGE IN WATER STORAGE	1.363	371531.687	6.95
SOIL WATER AT START OF YEAR	24.537	6687394.500	
SOIL WATER AT END OF YEAR	25.459	6938495.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.442	120431.141	2.25
ANNUAL WATER BUDGET BALANCE	0.0000	-0.845	0.00

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ANNUAL TOTALS FOR YEAR 4

	INCHES	CU. FEET	PERCENT
PRECIPITATION	15.87	4325216.000	100.00
RUNOFF	0.000	0.000	0.00
EVAPOTRANSPIRATION	10.209	2782426.000	64.33
DRAINAGE COLLECTED FROM LAYER 1	3.1038	845917.375	19.56
PERC./LEAKAGE THROUGH LAYER 2	2.753954	750563.750	17.35
AVG. HEAD ON TOP OF LAYER 2	9.2865		
DRAINAGE COLLECTED FROM LAYER 3	0.2469	67301.023	1.56
PERC./LEAKAGE THROUGH LAYER 5	1.776936	484286.906	11.20
AVG. HEAD ON TOP OF LAYER 4	10.2522		
CHANGE IN WATER STORAGE	0.533	145283.922	3.36
SOIL WATER AT START OF YEAR	25.459	6938495.000	
SOIL WATER AT END OF YEAR	26.434	7204210.000	
SNOW WATER AT START OF YEAR	0.442	120431.141	2.78
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	1.040	0.00

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ANNUAL TOTALS FOR YEAR 5

	INCHES	CU. FEET	PERCENT
PRECIPITATION	17.20	4687695.000	100.00
RUNOFF	0.000	0.000	0.00
EVAPOTRANSPIRATION	13.694	3732216.500	79.62
DRAINAGE COLLECTED FROM LAYER 1	2.5923	706515.375	15.07
PERC./LEAKAGE THROUGH LAYER 2	2.321594	632728.187	13.50
AVG. HEAD ON TOP OF LAYER 2	7.7780		
DRAINAGE COLLECTED FROM LAYER 3	0.3055	83259.437	1.78
PERC./LEAKAGE THROUGH LAYER 5	1.899671	517737.031	11.04
AVG. HEAD ON TOP OF LAYER 4	12.7176		
CHANGE IN WATER STORAGE	-1.292	-352035.344	-7.51
SOIL WATER AT START OF YEAR	26.434	7204210.000	
SOIL WATER AT END OF YEAR	25.142	6852174.500	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	2.209	0.00

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 5

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	0.14	0.22	0.83	1.27	0.35	1.63
	3.15	1.99	2.71	1.34	1.95	2.01
STD. DEVIATIONS	0.16	0.06	0.73	1.02	0.33	1.98
	1.47	1.38	1.72	1.54	2.39	1.39
RUNOFF						
TOTALS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
STD. DEVIATIONS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
EVAPOTRANSPIRATION						
TOTALS	0.498	0.159	0.647	0.769	0.403	0.921
	1.866	2.019	1.837	0.982	1.081	0.789
STD. DEVIATIONS	0.410	0.042	0.635	0.536	0.318	0.992
	1.110	1.319	0.973	0.873	1.005	0.636
LATERAL DRAINAGE COLLECTED FROM LAYER 1						
TOTALS	0.2508	0.1988	0.1866	0.1614	0.1566	0.1570
	0.1842	0.2059	0.2136	0.2348	0.2557	0.2773
STD. DEVIATIONS	0.1584	0.1222	0.1120	0.0965	0.0849	0.0798
	0.0972	0.1001	0.0380	0.0170	0.0668	0.0952
PERCOLATION/LEAKAGE THROUGH LAYER 2						
TOTALS	0.2207	0.1761	0.1666	0.1449	0.1418	0.1421
	0.1655	0.1841	0.1911	0.2096	0.2268	0.2453
STD. DEVIATIONS	0.1382	0.1075	0.0993	0.0860	0.0749	0.0700
	0.0848	0.0867	0.0325	0.0143	0.0563	0.0802
LATERAL DRAINAGE COLLECTED FROM LAYER 3						
TOTALS	0.0095	0.0094	0.0111	0.0113	0.0119	0.0117
	0.0118	0.0122	0.0123	0.0132	0.0132	0.0142
STD. DEVIATIONS	0.0106	0.0100	0.0116	0.0117	0.0122	0.0117
	0.0116	0.0120	0.0116	0.0116	0.0110	0.0113

PERCOLATION/LEAKAGE THROUGH LAYER 5

TOTALS	0.0711	0.0686	0.0787	0.0789	0.0831	0.0816
	0.0842	0.0854	0.0862	0.0933	0.0936	0.1005
STD. DEVIATIONS	0.0755	0.0698	0.0776	0.0763	0.0793	0.0771
	0.0790	0.0799	0.0758	0.0757	0.0713	0.0728

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 2

AVERAGES	8.8540	7.6895	6.5876	5.8863	5.5273	5.7262
	6.5009	7.2673	7.7924	8.2902	9.3285	9.7881
STD. DEVIATIONS	5.5920	4.6994	3.9532	3.5184	2.9975	2.9093
	3.4317	3.5327	1.3868	0.6007	2.4366	3.3601

DAILY AVERAGE HEAD ON TOP OF LAYER 4

AVERAGES	4.6762	5.0298	5.4438	5.7365	5.8515	5.9203
	5.7693	5.9619	6.2218	6.4691	6.6832	6.9696
STD. DEVIATIONS	5.2182	5.3793	5.7079	5.9420	5.9587	5.9502
	5.6657	5.8942	5.8543	5.6990	5.5829	5.5567

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 5

	INCHES	CU. FEET	PERCENT
PRECIPITATION	17.60 (1.967)	4797256.0	100.00
RUNOFF	0.000 (0.0000)	0.00	0.000
EVAPOTRANSPIRATION	11.972 (1.8209)	3262848.25	68.015
LATERAL DRAINAGE COLLECTED FROM LAYER 1	2.48268 (0.94519)	676629.812	14.10452
PERCOLATION/LEAKAGE THROUGH LAYER 2	2.21477 (0.82738)	603615.062	12.58251
AVERAGE HEAD ON TOP OF LAYER 2 **TOP HDPE**	7.437 (2.837)		
LATERAL DRAINAGE COLLECTED FROM LAYER 3	0.14178 (0.13639)	38641.863	0.80550
PERCOLATION/LEAKAGE THROUGH LAYER 5 **BASE LAYER (CLAY)**	1.00526 (0.90102)	273974.406	5.71107
AVERAGE HEAD ON TOP OF LAYER 4	5.894 (5.673)		
CHANGE IN WATER STORAGE	2.000 (2.6457)	545161.06	11.364

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              PEAK DAILY VALUES FOR YEARS    1 THROUGH    5
   ------------------------------------------------------------------------
                                                 (INCHES)      (CU. FT.)
                                                ----------   -------------
      PRECIPITATION                               2.17        591412.687

      RUNOFF                                      0.000            0.0000

      DRAINAGE COLLECTED FROM LAYER  1            0.01504       4097.85791

      PERCOLATION/LEAKAGE THROUGH LAYER  2        0.013039      3553.75781

      AVERAGE HEAD ON TOP OF LAYER  2            16.454

      MAXIMUM HEAD ON TOP OF LAYER  2            28.451

      LOCATION OF MAXIMUM HEAD IN LAYER  1
            (DISTANCE FROM DRAIN)               209.1 FEET

      DRAINAGE COLLECTED FROM LAYER  3            0.00087        237.24052

      PERCOLATION/LEAKAGE THROUGH LAYER  5        0.005277      1438.13074

      AVERAGE HEAD ON TOP OF LAYER  4            13.230

      MAXIMUM HEAD ON TOP OF LAYER  4            23.318

      LOCATION OF MAXIMUM HEAD IN LAYER  3
            (DISTANCE FROM DRAIN)               183.2 FEET

      SNOW WATER                                  3.22        878556.3120


      MAXIMUM VEG. SOIL WATER (VOL/VOL)                   0.2113

      MINIMUM VEG. SOIL WATER (VOL/VOL)                   0.0180

       ***  Maximum heads are computed using McEnroe's equations.  ***

            Reference:  Maximum Saturated Depth over Landfill Liner
                        by Bruce M. McEnroe, University of Kansas
                        ASCE Journal of Environmental Engineering
                        Vol. 119, No. 2, March 1993, pp. 262-270.

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FINAL WATER STORAGE AT END OF YEAR 5

LAYER	(INCHES)	(VOL/VOL)
1	5.5520	0.1542
2	0.0000	0.0000
3	8.0074	0.3336
4	0.0000	0.0000
5	10.2480	0.4270
SNOW WATER	0.000	

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ATTACHMENT III.4.A-2

TIER 1, SIMULATION 6

ALTERNATIVE LINER, SOIL TYPE 10 MODIFIED

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******************************************************************************
******************************************************************************
**                                                                          **
**                                                                          **
**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE               **
**                HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)                **
**                  DEVELOPED BY ENVIRONMENTAL LABORATORY                   **
**                    USAE WATERWAYS EXPERIMENT STATION                     **
**             FOR USEPA RISK REDUCTION ENGINEERING LABORATORY              **
**                                                                          **
**                                                                          **
******************************************************************************
******************************************************************************


PRECIPITATION DATA FILE:    d:\lllf\DATA4.D4
TEMPERATURE DATA FILE:      D:\lllf\DATA7.D7
SOLAR RADIATION DATA FILE:  d:\lllf\DATA13.D13
EVAPOTRANSPIRATION DATA:    d:\lllf\DATA11.D11
SOIL AND DESIGN DATA FILE:  d:\lllf\SIM6A.D10
OUTPUT DATA FILE:           d:\lllf\SIM6OUT.OUT


TIME:   9:10     DATE:   7/ 5/2019


******************************************************************************

     TITLE:  Lea Land Landfill Simulation #6
          PERFORMANCE-BASED ALTERNATE LINER SYSTEM
******************************************************************************


     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER
              WERE SPECIFIED BY THE USER.

                                   LAYER  1
                                   --------

                      TYPE 2 - LATERAL DRAINAGE LAYER - PSL
                        MATERIAL TEXTURE NUMBER   0
           THICKNESS                    =     24.00   INCHES
           POROSITY                     =      0.3980 VOL/VOL
           FIELD CAPACITY               =      0.2440 VOL/VOL
           WILTING POINT                =      0.1360 VOL/VOL
           INITIAL SOIL WATER CONTENT   =      0.1630 VOL/VOL
           EFFECTIVE SAT. HYD. COND.    =  0.199999995000E-03 CM/SEC
```

```
                        LAYER  2
                        --------

            TYPE 2 - LATERAL DRAINAGE LAYER
           – GEOCOMPOSITE (MODELED AS GEONET)
               MATERIAL TEXTURE NUMBER  20
THICKNESS                  =      0.20   INCHES
POROSITY                   =      0.8500 VOL/VOL
FIELD CAPACITY             =      0.0100 VOL/VOL
WILTING POINT              =      0.0050 VOL/VOL
INITIAL SOIL WATER CONTENT =      0.0062 VOL/VOL
EFFECTIVE SAT. HYD. COND.  =   10.0000000000     CM/SEC
SLOPE                      =      2.50   PERCENT
DRAINAGE LENGTH            =   1550.0    FEET


                         LAYER  3
                        --------

            TYPE 4 - FLEXIBLE MEMBRANE LINER – TOP HDPE
              MATERIAL TEXTURE NUMBER  35
THICKNESS                  =      0.06   INCHES
POROSITY                   =      0.0000 VOL/VOL
FIELD CAPACITY             =      0.0000 VOL/VOL
WILTING POINT              =      0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT =      0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.  =  0.199999996000E-12 CM/SEC
FML PINHOLE DENSITY        =      1.00   HOLES/ACRE
FML INSTALLATION DEFECTS   =      1.00   HOLES/ACRE
FML PLACEMENT QUALITY      =  3 - GOOD


                        LAYER  4
                        --------

            TYPE 2 - LATERAL DRAINAGE LAYER - GEONET
              MATERIAL TEXTURE NUMBER  20
THICKNESS                  =      0.20   INCHES
POROSITY                   =      0.8500 VOL/VOL
FIELD CAPACITY             =      0.0100 VOL/VOL
WILTING POINT              =      0.0050 VOL/VOL
INITIAL SOIL WATER CONTENT =      0.0062 VOL/VOL
EFFECTIVE SAT. HYD. COND.  =   10.0000000000     CM/SEC
SLOPE                      =      2.50   PERCENT
DRAINAGE LENGTH            =   1550.0    FEET
```

LAYER 5

TYPE 4 - FLEXIBLE MEMBRANE LINER **– BOTTOM HDPE**
MATERIAL TEXTURE NUMBER 35

```
THICKNESS                    =      0.06   INCHES
POROSITY                     =      0.0000 VOL/VOL
FIELD CAPACITY               =      0.0000 VOL/VOL
WILTING POINT                =      0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.199999996000E-12 CM/SEC
FML PINHOLE DENSITY          =      1.00   HOLES/ACRE
FML INSTALLATION DEFECTS     =      1.00   HOLES/ACRE
FML PLACEMENT QUALITY        =  3 - GOOD
```

LAYER 6

TYPE 3 - BARRIER SOIL LINER **- GCL**
MATERIAL TEXTURE NUMBER 17

```
THICKNESS                    =      0.25   INCHES
POROSITY                     =      0.7500 VOL/VOL
FIELD CAPACITY               =      0.7470 VOL/VOL
WILTING POINT                =      0.4000 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.7500 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.300000003000E-08 CM/SEC
```

GENERAL DESIGN AND EVAPORATIVE ZONE DATA

```
NOTE:  SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM A USER-
         SPECIFIED CURVE NUMBER OF 77.0, A SURFACE SLOPE
         OF  2.% AND A SLOPE LENGTH OF 1550. FEET.

SCS RUNOFF CURVE NUMBER              =     75.10
FRACTION OF AREA ALLOWING RUNOFF     =      0.0    PERCENT
AREA PROJECTED ON HORIZONTAL PLANE   =     75.080  ACRES
EVAPORATIVE ZONE DEPTH               =     14.0    INCHES
INITIAL WATER IN EVAPORATIVE ZONE    =      2.282  INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE   =      5.572  INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE   =      1.904  INCHES
INITIAL SNOW WATER                   =      0.000  INCHES
INITIAL WATER IN LAYER MATERIALS     =      4.102  INCHES
TOTAL INITIAL WATER                  =      4.102  INCHES
TOTAL SUBSURFACE INFLOW              =      0.00   INCHES/YEAR
```

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
ROSWELL NEW MEXICO

STATION LATITUDE	=	32.52 DEGREES
MAXIMUM LEAF AREA INDEX	=	0.00
START OF GROWING SEASON (JULIAN DATE)	=	76
END OF GROWING SEASON (JULIAN DATE)	=	310
EVAPORATIVE ZONE DEPTH	=	14.0 INCHES
AVERAGE ANNUAL WIND SPEED	=	8.70 MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	49.00 %
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	40.00 %
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	53.00 %
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	52.00 %

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR ROSWELL NEW MEXICO

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
0.20	0.37	0.76	1.78	0.35	2.35
2.62	2.26	2.83	1.67	0.77	1.10

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR ROSWELL NEW MEXICO

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
45.80	48.40	53.50	62.00	71.50	78.30
80.50	78.80	71.90	62.60	54.00	43.10

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR ROSWELL NEW MEXICO
AND STATION LATITUDE = 33.24 DEGREES

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ANNUAL TOTALS FOR YEAR 1

	INCHES	CU. FEET	PERCENT
PRECIPITATION	15.63	4259806.500	100.00
RUNOFF	0.000	0.000	0.00
EVAPOTRANSPIRATION	11.755	3203824.500	75.21
DRAINAGE COLLECTED FROM LAYER 2	2.1388	582909.250	13.68
PERC./LEAKAGE THROUGH LAYER 3	0.681252	185668.609	4.36
AVG. HEAD ON TOP OF LAYER 3	0.0642		
DRAINAGE COLLECTED FROM LAYER 4	0.6805	185461.406	4.35
PERC./LEAKAGE THROUGH LAYER 6	0.000001	0.181	0.00
AVG. HEAD ON TOP OF LAYER 5	0.0021		
CHANGE IN WATER STORAGE	1.055	287610.781	6.75
SOIL WATER AT START OF YEAR	4.103	1118284.370	
SOIL WATER AT END OF YEAR	5.158	1405895.250	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.420	0.00

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ANNUAL TOTALS FOR YEAR 2

	INCHES	CU. FEET	PERCENT
PRECIPITATION	19.70	5369045.500	100.00
RUNOFF	0.000	0.000	0.00
EVAPOTRANSPIRATION	15.054	4102907.250	76.42
DRAINAGE COLLECTED FROM LAYER 2	2.5296	689412.000	12.84
PERC./LEAKAGE THROUGH LAYER 3	1.013664	276264.375	5.15
AVG. HEAD ON TOP OF LAYER 3	0.1355		
DRAINAGE COLLECTED FROM LAYER 4	1.0040	273636.375	5.10
PERC./LEAKAGE THROUGH LAYER 6	0.000001	0.286	0.00
AVG. HEAD ON TOP OF LAYER 5	0.0030		
CHANGE IN WATER STORAGE	1.112	303088.844	5.65
SOIL WATER AT START OF YEAR	5.158	1405895.250	
SOIL WATER AT END OF YEAR	6.271	1708984.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.981	0.00

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ANNUAL TOTALS FOR YEAR 3

	INCHES	CU. FEET	PERCENT
PRECIPITATION	19.61	5344517.000	100.00
RUNOFF	0.000	0.000	0.00
EVAPOTRANSPIRATION	16.622	4530109.000	84.76
DRAINAGE COLLECTED FROM LAYER 2	3.3898	923867.250	17.29
PERC./LEAKAGE THROUGH LAYER 3	0.821673	223939.219	4.19
AVG. HEAD ON TOP OF LAYER 3	0.0535		
DRAINAGE COLLECTED FROM LAYER 4	0.8313	226566.562	4.24
PERC./LEAKAGE THROUGH LAYER 6	0.000001	0.297	0.00
AVG. HEAD ON TOP OF LAYER 5	0.0025		
CHANGE IN WATER STORAGE	-1.233	-336026.437	-6.29
SOIL WATER AT START OF YEAR	6.271	1708984.000	
SOIL WATER AT END OF YEAR	4.596	1252526.500	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.442	120431.141	2.25
ANNUAL WATER BUDGET BALANCE	0.0000	0.458	0.00

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ANNUAL TOTALS FOR YEAR 4

	INCHES	CU. FEET	PERCENT
PRECIPITATION	15.87	4325216.000	100.00
RUNOFF	0.000	0.000	0.00
EVAPOTRANSPIRATION	11.947	3255924.750	75.28
DRAINAGE COLLECTED FROM LAYER 2	2.0676	563492.125	13.03
PERC./LEAKAGE THROUGH LAYER 3	0.594630	162060.828	3.75
AVG. HEAD ON TOP OF LAYER 3	0.0535		
DRAINAGE COLLECTED FROM LAYER 4	0.5941	161904.625	3.74
PERC./LEAKAGE THROUGH LAYER 6	0.000001	0.303	0.00
AVG. HEAD ON TOP OF LAYER 5	0.0018		
CHANGE IN WATER STORAGE	1.262	343893.906	7.95
SOIL WATER AT START OF YEAR	4.596	1252526.500	
SOIL WATER AT END OF YEAR	6.299	1716851.500	
SNOW WATER AT START OF YEAR	0.442	120431.141	2.78
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.550	0.00

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ANNUAL TOTALS FOR YEAR 5

	INCHES	CU. FEET	PERCENT
PRECIPITATION	17.20	4687695.000	100.00
RUNOFF	0.000	0.000	0.00
EVAPOTRANSPIRATION	16.740	4562452.000	97.33
DRAINAGE COLLECTED FROM LAYER 2	0.9864	268829.500	5.73
PERC./LEAKAGE THROUGH LAYER 3	0.224259	61119.523	1.30
AVG. HEAD ON TOP OF LAYER 3	0.0029		
DRAINAGE COLLECTED FROM LAYER 4	0.2191	59710.387	1.27
PERC./LEAKAGE THROUGH LAYER 6	0.000001	0.339	0.00
AVG. HEAD ON TOP OF LAYER 5	0.0006		
CHANGE IN WATER STORAGE	-0.746	-203298.078	-4.34
SOIL WATER AT START OF YEAR	6.299	1716851.500	
SOIL WATER AT END OF YEAR	5.554	1513553.500	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	1.431	0.00

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        AVERAGE MONTHLY VALUES IN INCHES FOR YEARS    1 THROUGH    5
-------------------------------------------------------------------------------

                        JAN/JUL  FEB/AUG  MAR/SEP  APR/OCT  MAY/NOV  JUN/DEC
                        -------  -------  -------  -------  -------  -------
 PRECIPITATION
 -------------
   TOTALS                 0.14     0.22     0.83     1.27     0.35     1.63
                          3.15     1.99     2.71     1.34     1.95     2.01

   STD. DEVIATIONS        0.16     0.06     0.73     1.02     0.33     1.98
                          1.47     1.38     1.72     1.54     2.39     1.39

 RUNOFF
 ------
   TOTALS                 0.000    0.000    0.000    0.000    0.000    0.000
                          0.000    0.000    0.000    0.000    0.000    0.000

   STD. DEVIATIONS        0.000    0.000    0.000    0.000    0.000    0.000
                          0.000    0.000    0.000    0.000    0.000    0.000

 EVAPOTRANSPIRATION
 ------------------
   TOTALS                 0.851    0.273    0.623    0.746    0.315    1.222
                          2.539    2.071    2.092    1.324    1.300    1.067

   STD. DEVIATIONS        0.600    0.094    0.659    0.667    0.236    1.578
                          1.579    1.414    1.257    0.953    1.070    0.654

 LATERAL DRAINAGE COLLECTED FROM LAYER  2
 ----------------------------------------
   TOTALS                 0.0831   0.0017   0.0001   0.0000   0.0000   0.3349
                          0.4463   0.1265   0.3079   0.2253   0.3943   0.3024

   STD. DEVIATIONS        0.1410   0.0032   0.0001   0.0000   0.0000   0.5748
                          0.4680   0.2051   0.6863   0.4470   0.7515   0.4516

 PERCOLATION/LEAKAGE THROUGH LAYER  3
 ------------------------------------
   TOTALS                 0.0251   0.0014   0.0001   0.0000   0.0000   0.1211
                          0.1004   0.0319   0.1056   0.0662   0.1092   0.1061

   STD. DEVIATIONS        0.0387   0.0023   0.0002   0.0001   0.0000   0.2505
                          0.1096   0.0477   0.2335   0.1257   0.1976   0.1670

 LATERAL DRAINAGE COLLECTED FROM LAYER  4
 ----------------------------------------
   TOTALS                 0.0272   0.0014   0.0001   0.0000   0.0000   0.1188
                          0.1013   0.0319   0.1066   0.0655   0.1081   0.1051

   STD. DEVIATIONS        0.0427   0.0023   0.0002   0.0000   0.0000   0.2512
                          0.1082   0.0456   0.2363   0.1232   0.1941   0.1624
```

```
 PERCOLATION/LEAKAGE THROUGH LAYER  6
 ------------------------------------
   TOTALS                 0.0000   0.0000   0.0000   0.0000   0.0000   0.0000
                          0.0000   0.0000   0.0000   0.0000   0.0000   0.0000

   STD. DEVIATIONS        0.0000   0.0000   0.0000   0.0000   0.0000   0.0000
                          0.0000   0.0000   0.0000   0.0000   0.0000   0.0000


-------------------------------------------------------------------------------
               AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)
-------------------------------------------------------------------------------


 DAILY AVERAGE HEAD ON TOP OF LAYER  3
 -------------------------------------
   AVERAGES               0.0029   0.0001   0.0000   0.0000   0.0000   0.1958
                          0.0697   0.0045   0.1498   0.0744   0.1117   0.1340

   STD. DEVIATIONS        0.0050   0.0001   0.0000   0.0000   0.0000   0.4307
                          0.1144   0.0072   0.3349   0.1644   0.2449   0.2915

 DAILY AVERAGE HEAD ON TOP OF LAYER  5
 -------------------------------------
   AVERAGES               0.0010   0.0001   0.0000   0.0000   0.0000   0.0043
                          0.0036   0.0011   0.0039   0.0023   0.0039   0.0037

   STD. DEVIATIONS        0.0015   0.0001   0.0000   0.0000   0.0000   0.0092
                          0.0038   0.0016   0.0086   0.0043   0.0071   0.0057

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 5

	INCHES	CU. FEET	PERCENT
PRECIPITATION	17.60 (1.967)	4797256.0	100.00
RUNOFF	0.000 (0.0000)	0.00	0.000
EVAPOTRANSPIRATION	14.424 (2.4420)	3931043.50	81.944
LATERAL DRAINAGE COLLECTED FROM LAYER 2	2.22243 (0.86833)	605702.062	12.62601
PERCOLATION/LEAKAGE THROUGH LAYER 3	0.66710 (0.29392)	181810.500	3.78989
AVERAGE HEAD ON TOP OF LAYER 3 **TOP HDPE**	0.062 (0.048)		
LATERAL DRAINAGE COLLECTED FROM LAYER 4	0.66579 (0.29441)	181455.875	3.78249
PERCOLATION/LEAKAGE THROUGH LAYER 6 **GCL**	0.00000 (0.00000)	0.281	0.00001
AVERAGE HEAD ON TOP OF LAYER 5	0.002 (0.001)		
CHANGE IN WATER STORAGE	0.290 (1.1831)	79053.81	1.648

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             PEAK DAILY VALUES FOR YEARS    1 THROUGH    5
------------------------------------------------------------------------
                                                (INCHES)      (CU. FT.)
                                               ----------   -------------
  PRECIPITATION                                  2.17        591412.687

  RUNOFF                                         0.000            0.0000

  DRAINAGE COLLECTED FROM LAYER  2               0.18292      49851.97270

  PERCOLATION/LEAKAGE THROUGH LAYER  3           0.129328     35247.13670

  AVERAGE HEAD ON TOP OF LAYER  3                8.555

  MAXIMUM HEAD ON TOP OF LAYER  3               15.566

  LOCATION OF MAXIMUM HEAD IN LAYER  2
        (DISTANCE FROM DRAIN)                  139.0 FEET

  DRAINAGE COLLECTED FROM LAYER  4               0.11326      30868.17770

  PERCOLATION/LEAKAGE THROUGH LAYER  6           0.000000         0.00685

  AVERAGE HEAD ON TOP OF LAYER  5                0.124

  MAXIMUM HEAD ON TOP OF LAYER  5                0.247

  LOCATION OF MAXIMUM HEAD IN LAYER  4
        (DISTANCE FROM DRAIN)                    3.6 FEET

  SNOW WATER                                     3.22        878556.3120


  MAXIMUM VEG. SOIL WATER (VOL/VOL)                      0.3266

  MINIMUM VEG. SOIL WATER (VOL/VOL)                      0.1360


   ***  Maximum heads are computed using McEnroe's equations.  ***

        Reference:  Maximum Saturated Depth over Landfill Liner
                    by Bruce M. McEnroe, University of Kansas
                    ASCE Journal of Environmental Engineering
                    Vol. 119, No. 2, March 1993, pp. 262-270.

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FINAL WATER STORAGE AT END OF YEAR 5

LAYER	(INCHES)	(VOL/VOL)
1	5.3306	0.2221
2	0.0265	0.1324
3	0.0000	0.0000
4	0.0077	0.0387
5	0.0000	0.0000
6	0.1875	0.7500
SNOW WATER	0.000	

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ATTACHMENT III.4.A-3

TIER 2, SIMULATION 7

ALTERNATIVE LINER WITH SOIL TYPE 10 MODIFIED

2-YEAR START-UP WITH NO WASTE

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******************************************************************************
******************************************************************************
**                                                                          **
**                                                                          **
**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE               **
**                HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)                **
**                DEVELOPED BY ENVIRONMENTAL LABORATORY                     **
**                  USAE WATERWAYS EXPERIMENT STATION                       **
**              FOR USEPA RISK REDUCTION ENGINEERING LABORATORY             **
**                                                                          **
**                                                                          **
******************************************************************************
******************************************************************************


PRECIPITATION DATA FILE:    d:\lllf\DATA4.D4
TEMPERATURE DATA FILE:      D:\lllf\DATA7.D7
SOLAR RADIATION DATA FILE:  d:\lllf\DATA13.D13
EVAPOTRANSPIRATION DATA:    d:\lllf\DATA11.D11
SOIL AND DESIGN DATA FILE:  d:\lllf\SIM7.D10
OUTPUT DATA FILE:           d:\lllf\SIM7OUT.OUT


TIME:   9:12     DATE:   7/ 5/2019


******************************************************************************

     TITLE:  Lea Land Landfill Simulation #7
          START-UP WITH NO WASTE
******************************************************************************


     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER
              WERE SPECIFIED BY THE USER.

                              LAYER  1
                              --------

                  TYPE 2 - LATERAL DRAINAGE LAYER - PSL
                    MATERIAL TEXTURE NUMBER   0
           THICKNESS                   =     24.00   INCHES
           POROSITY                    =      0.3980 VOL/VOL
           FIELD CAPACITY              =      0.2440 VOL/VOL
           WILTING POINT               =      0.1360 VOL/VOL
           INITIAL SOIL WATER CONTENT  =      0.1630 VOL/VOL
           EFFECTIVE SAT. HYD. COND.   =  0.199999995000E-03 CM/SEC
```

LAYER 2

TYPE 2 - LATERAL DRAINAGE LAYER
– GEOCOMPOSITE (MODELED AS GEONET)
MATERIAL TEXTURE NUMBER 20

```
THICKNESS                    =      0.20   INCHES
POROSITY                     =      0.8500 VOL/VOL
FIELD CAPACITY               =      0.0100 VOL/VOL
WILTING POINT                =      0.0050 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.0062 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =   10.0000000000     CM/SEC
SLOPE                        =      2.50   PERCENT
DRAINAGE LENGTH              =   1550.0    FEET
```

LAYER 3

TYPE 4 - FLEXIBLE MEMBRANE LINER **– TOP HDPE**
MATERIAL TEXTURE NUMBER 35

```
THICKNESS                    =      0.06   INCHES
POROSITY                     =      0.0000 VOL/VOL
FIELD CAPACITY               =      0.0000 VOL/VOL
WILTING POINT                =      0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.199999996000E-12 CM/SEC
FML PINHOLE DENSITY          =      1.00   HOLES/ACRE
FML INSTALLATION DEFECTS     =      4.00   HOLES/ACRE
FML PLACEMENT QUALITY        =  3 - GOOD
```

LAYER 4

TYPE 2 - LATERAL DRAINAGE LAYER **- GEONET**
MATERIAL TEXTURE NUMBER 20

```
THICKNESS                    =      0.20   INCHES
POROSITY                     =      0.8500 VOL/VOL
FIELD CAPACITY               =      0.0100 VOL/VOL
WILTING POINT                =      0.0050 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.0062 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =   10.0000000000     CM/SEC
SLOPE                        =      2.50   PERCENT
DRAINAGE LENGTH              =   1550.0    FEET
```

LAYER 5

TYPE 4 - FLEXIBLE MEMBRANE LINER **– BOTTOM HDPE**

MATERIAL TEXTURE NUMBER 35

THICKNESS	=	0.06 INCHES
POROSITY	=	0.0000 VOL/VOL
FIELD CAPACITY	=	0.0000 VOL/VOL
WILTING POINT	=	0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.199999996000E-12 CM/SEC
FML PINHOLE DENSITY	=	1.00 HOLES/ACRE
FML INSTALLATION DEFECTS	=	4.00 HOLES/ACRE
FML PLACEMENT QUALITY	=	3 - GOOD

LAYER 6

TYPE 3 - BARRIER SOIL LINER **- GCL**

MATERIAL TEXTURE NUMBER 17

THICKNESS	=	0.25 INCHES
POROSITY	=	0.7500 VOL/VOL
FIELD CAPACITY	=	0.7470 VOL/VOL
WILTING POINT	=	0.4000 VOL/VOL
INITIAL SOIL WATER CONTENT	=	0.7500 VOL/VOL
EFFECTIVE SAT. HYD. COND.	=	0.300000003000E-08 CM/SEC

GENERAL DESIGN AND EVAPORATIVE ZONE DATA

NOTE: SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM A USER-SPECIFIED CURVE NUMBER OF 77.0, A SURFACE SLOPE OF 2.% AND A SLOPE LENGTH OF 1550. FEET.

SCS RUNOFF CURVE NUMBER	=	75.10	
FRACTION OF AREA ALLOWING RUNOFF	=	0.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	75.080	ACRES
EVAPORATIVE ZONE DEPTH	=	14.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	2.282	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	5.572	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	1.904	INCHES
INITIAL SNOW WATER	=	0.000	INCHES
INITIAL WATER IN LAYER MATERIALS	=	4.102	INCHES
TOTAL INITIAL WATER	=	4.102	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM
ROSWELL NEW MEXICO

STATION LATITUDE	=	32.52 DEGREES
MAXIMUM LEAF AREA INDEX	=	0.00
START OF GROWING SEASON (JULIAN DATE)	=	76
END OF GROWING SEASON (JULIAN DATE)	=	310
EVAPORATIVE ZONE DEPTH	=	14.0 INCHES
AVERAGE ANNUAL WIND SPEED	=	8.70 MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	49.00 %
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	40.00 %
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	53.00 %
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	52.00 %

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR ROSWELL NEW MEXICO

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
0.20	0.37	0.76	1.78	0.35	2.35
2.62	2.26	2.83	1.67	0.77	1.10

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR ROSWELL NEW MEXICO

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
45.80	48.40	53.50	62.00	71.50	78.30
80.50	78.80	71.90	62.60	54.00	43.10

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING
COEFFICIENTS FOR ROSWELL NEW MEXICO
AND STATION LATITUDE = 33.24 DEGREES

****

ANNUAL TOTALS FOR YEAR 1

	INCHES	CU. FEET	PERCENT
PRECIPITATION	15.63	4259806.500	100.00
RUNOFF	0.000	0.000	0.00
EVAPOTRANSPIRATION	11.755	3203824.500	75.21
DRAINAGE COLLECTED FROM LAYER 2	1.2809	349106.531	8.20
PERC./LEAKAGE THROUGH LAYER 3	1.539117	419471.469	9.85
AVG. HEAD ON TOP OF LAYER 3	0.0310		
DRAINAGE COLLECTED FROM LAYER 4	1.5384	419263.937	9.84
PERC./LEAKAGE THROUGH LAYER 6	0.000001	0.297	0.00
AVG. HEAD ON TOP OF LAYER 5	0.0046		
CHANGE IN WATER STORAGE	1.055	287610.781	6.75
SOIL WATER AT START OF YEAR	4.103	1118284.370	
SOIL WATER AT END OF YEAR	5.158	1405895.250	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.515	0.00

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ANNUAL TOTALS FOR YEAR 2

	INCHES	CU. FEET	PERCENT
PRECIPITATION	19.70	5369045.500	100.00
RUNOFF	0.000	0.000	0.00
EVAPOTRANSPIRATION	15.054	4102907.250	76.42
DRAINAGE COLLECTED FROM LAYER 2	1.5839	431676.312	8.04
PERC./LEAKAGE THROUGH LAYER 3	2.006316	546802.250	10.18
AVG. HEAD ON TOP OF LAYER 3	0.0638		
DRAINAGE COLLECTED FROM LAYER 4	1.9838	540662.625	10.07
PERC./LEAKAGE THROUGH LAYER 6	0.000002	0.436	0.00
AVG. HEAD ON TOP OF LAYER 5	0.0059		
CHANGE IN WATER STORAGE	1.078	293798.312	5.47
SOIL WATER AT START OF YEAR	5.158	1405895.250	
SOIL WATER AT END OF YEAR	6.236	1699693.500	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.799	0.00

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 2

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	0.18	0.25	0.64	2.13	0.34	2.36
	2.26	2.20	3.56	0.36	1.42	1.97
STD. DEVIATIONS	0.25	0.01	0.82	0.79	0.47	3.32
	0.41	0.62	1.70	0.50	0.91	2.14
RUNOFF						
TOTALS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
STD. DEVIATIONS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
EVAPOTRANSPIRATION						
TOTALS	0.409	0.260	0.776	1.220	0.335	1.936
	1.376	2.228	2.366	0.792	0.964	0.743
STD. DEVIATIONS	0.044	0.081	0.932	0.993	0.459	2.733
	1.589	0.909	1.706	1.046	0.979	0.037
LATERAL DRAINAGE COLLECTED FROM LAYER 2						
TOTALS	0.0000	0.0002	0.0001	0.0000	0.0000	0.3892
	0.0001	0.1374	0.4748	0.0225	0.0232	0.3850
STD. DEVIATIONS	0.0000	0.0003	0.0001	0.0000	0.0000	0.5504
	0.0001	0.1943	0.6702	0.0318	0.0328	0.5289
PERCOLATION/LEAKAGE THROUGH LAYER 3						
TOTALS	0.0000	0.0012	0.0003	0.0001	0.0001	0.5586
	0.0004	0.1592	0.5495	0.0494	0.0472	0.4067
STD. DEVIATIONS	0.0000	0.0017	0.0004	0.0001	0.0001	0.7899
	0.0006	0.2252	0.7729	0.0699	0.0668	0.5327
LATERAL DRAINAGE COLLECTED FROM LAYER 4						
TOTALS	0.0000	0.0012	0.0003	0.0000	0.0001	0.5585
	0.0005	0.1487	0.5587	0.0505	0.0428	0.3999
STD. DEVIATIONS	0.0000	0.0017	0.0004	0.0000	0.0001	0.7899
	0.0007	0.2102	0.7887	0.0711	0.0605	0.5105

PERCOLATION/LEAKAGE THROUGH LAYER 6

TOTALS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)

DAILY AVERAGE HEAD ON TOP OF LAYER 3

AVERAGES	0.0000	0.0000	0.0000	0.0000	0.0000	0.2380
	0.0000	0.0049	0.1802	0.0008	0.0008	0.1442
STD. DEVIATIONS	0.0000	0.0000	0.0000	0.0000	0.0000	0.3366
	0.0000	0.0069	0.2548	0.0011	0.0012	0.2033

DAILY AVERAGE HEAD ON TOP OF LAYER 5

AVERAGES	0.0000	0.0000	0.0000	0.0000	0.0000	0.0204
	0.0000	0.0052	0.0204	0.0018	0.0016	0.0141
STD. DEVIATIONS	0.0000	0.0001	0.0000	0.0000	0.0000	0.0288
	0.0000	0.0074	0.0288	0.0025	0.0022	0.0180

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 2

	INCHES		CU. FEET	PERCENT
PRECIPITATION	17.66	(2.878)	4814426.0	100.00
RUNOFF	0.000	(0.0000)	0.00	0.000
EVAPOTRANSPIRATION	13.405	(2.3327)	3653365.75	75.884
LATERAL DRAINAGE COLLECTED FROM LAYER 2	1.43242	(0.21423)	390391.437	8.10878
PERCOLATION/LEAKAGE THROUGH LAYER 3	1.77272	(0.33036)	483136.844	10.03519
AVERAGE HEAD ON TOP OF LAYER 3 **TOP HDPE**	0.047	(0.023)		
LATERAL DRAINAGE COLLECTED FROM LAYER 4	1.76107	(0.31497)	479963.250	9.96927
PERCOLATION/LEAKAGE THROUGH LAYER 6 **GCL**	0.00000	(0.00000)	0.366	0.00001
AVERAGE HEAD ON TOP OF LAYER 5	0.005	(0.001)		
CHANGE IN WATER STORAGE	1.067	(0.0161)	290704.56	6.038

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             PEAK DAILY VALUES FOR YEARS    1 THROUGH    2
------------------------------------------------------------------------
                                                (INCHES)     (CU. FT.)
                                               ----------   -------------
  PRECIPITATION                                  2.17        591412.687

  RUNOFF                                         0.000            0.0000

  DRAINAGE COLLECTED FROM LAYER  2               0.18289      49844.10160

  PERCOLATION/LEAKAGE THROUGH LAYER  3           0.395567    107807.91400

  AVERAGE HEAD ON TOP OF LAYER  3                6.974

  MAXIMUM HEAD ON TOP OF LAYER  3               12.847

  LOCATION OF MAXIMUM HEAD IN LAYER  2
        (DISTANCE FROM DRAIN)                  121.4 FEET

  DRAINAGE COLLECTED FROM LAYER  4               0.18276      49810.35940

  PERCOLATION/LEAKAGE THROUGH LAYER  6           0.000000          0.01993

  AVERAGE HEAD ON TOP OF LAYER  5                0.200

  MAXIMUM HEAD ON TOP OF LAYER  5                0.399

  LOCATION OF MAXIMUM HEAD IN LAYER  4
        (DISTANCE FROM DRAIN)                    4.1 FEET

  SNOW WATER                                     3.22        878556.3120


  MAXIMUM VEG. SOIL WATER (VOL/VOL)                      0.3266

  MINIMUM VEG. SOIL WATER (VOL/VOL)                      0.1360


   ***  Maximum heads are computed using McEnroe's equations.  ***

        Reference:  Maximum Saturated Depth over Landfill Liner
                    by Bruce M. McEnroe, University of Kansas
                    ASCE Journal of Environmental Engineering
                    Vol. 119, No. 2, March 1993, pp. 262-270.

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FINAL WATER STORAGE AT END OF YEAR 2

LAYER	(INCHES)	(VOL/VOL)
1	5.9968	0.2499
2	0.0264	0.1321
3	0.0000	0.0000
4	0.0245	0.1226
5	0.0000	0.0000
6	0.1875	0.7500
SNOW WATER	0.000	

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ATTACHMENT III.4.A-4

TIER 2, SIMULATION 8

ALTERNATIVE LINER WITH SOIL TYPE 10 MODIFIED

INTERMEDIATE COVER WITH SOIL TYPE 10 MODIFIED

5-YEAR PARTIALLY FILLED

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**                                                                          **
**                                                                          **
**              HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE               **
**                HELP MODEL VERSION 3.07  (1 NOVEMBER 1997)                **
**                  DEVELOPED BY ENVIRONMENTAL LABORATORY                   **
**                    USAE WATERWAYS EXPERIMENT STATION                     **
**             FOR USEPA RISK REDUCTION ENGINEERING LABORATORY              **
**                                                                          **
**                                                                          **
******************************************************************************
******************************************************************************


PRECIPITATION DATA FILE:    d:\lllf\DATA4.D4
TEMPERATURE DATA FILE:      D:\lllf\DATA7.D7
SOLAR RADIATION DATA FILE:  d:\lllf\DATA13.D13
EVAPOTRANSPIRATION DATA:    d:\lllf\DATA11.D11
SOIL AND DESIGN DATA FILE:  d:\lllf\SIM8MC.D10
OUTPUT DATA FILE:           d:\lllf\sim8mc.OUT


TIME:   9: 4     DATE:   7/ 9/2019


******************************************************************************

     TITLE:  Lea Land Landfill Simulation #8
          PARTIAL FILL WITH INTERMEDIATE COVER
******************************************************************************


     NOTE:  INITIAL MOISTURE CONTENT OF THE LAYERS AND SNOW WATER
              WERE SPECIFIED BY THE USER.


                                   LAYER  1
                                   --------

                     TYPE 1 - VERTICAL PERCOLATION LAYER – INTERMEDIATE COVER
                         MATERIAL TEXTURE NUMBER   0
           THICKNESS                    =     12.00   INCHES
           POROSITY                     =      0.3980 VOL/VOL
           FIELD CAPACITY               =      0.2440 VOL/VOL
           WILTING POINT                =      0.1360 VOL/VOL
           INITIAL SOIL WATER CONTENT   =      0.1630 VOL/VOL
           EFFECTIVE SAT. HYD. COND.    =  0.199999995000E-03 CM/SEC
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                         LAYER  2
                         --------

             TYPE 1 - VERTICAL PERCOLATION LAYER - WASTE
                 MATERIAL TEXTURE NUMBER  22
THICKNESS                    =    240.00   INCHES
POROSITY                     =      0.4190 VOL/VOL
FIELD CAPACITY               =      0.3070 VOL/VOL
WILTING POINT                =      0.1800 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.2117 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.189999992000E-04 CM/SEC

                         LAYER  3
                         --------

               TYPE 2 - LATERAL DRAINAGE LAYER - PSL
                 MATERIAL TEXTURE NUMBER   0
THICKNESS                    =     24.00   INCHES
POROSITY                     =      0.3980 VOL/VOL
FIELD CAPACITY               =      0.2440 VOL/VOL
WILTING POINT                =      0.1360 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.2499 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.199999995000E-03 CM/SEC

                         LAYER  4
                         --------

                TYPE 2 - LATERAL DRAINAGE LAYER
              – GEOCOMPOSITE (MODELED AS GEONET)
                  MATERIAL TEXTURE NUMBER  20
THICKNESS                    =      0.20   INCHES
POROSITY                     =      0.8500 VOL/VOL
FIELD CAPACITY               =      0.0100 VOL/VOL
WILTING POINT                =      0.0050 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.1321 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =   10.0000000000     CM/SEC
SLOPE                        =      2.50   PERCENT
DRAINAGE LENGTH              =   1550.0    FEET

                         LAYER  5
                         --------

               TYPE 4 - FLEXIBLE MEMBRANE LINER – TOP HDPE
                 MATERIAL TEXTURE NUMBER  35
THICKNESS                    =      0.06   INCHES
POROSITY                     =      0.0000 VOL/VOL
FIELD CAPACITY               =      0.0000 VOL/VOL
WILTING POINT                =      0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT   =      0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.    =  0.199999996000E-12 CM/SEC
FML PINHOLE DENSITY          =      1.00   HOLES/ACRE
FML INSTALLATION DEFECTS     =      4.00   HOLES/ACRE
FML PLACEMENT QUALITY        =  3 - GOOD
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                         LAYER  6
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             TYPE 2 - LATERAL DRAINAGE LAYER - GEONET
               MATERIAL TEXTURE NUMBER  20
THICKNESS                   =      0.20   INCHES
POROSITY                    =      0.8500 VOL/VOL
FIELD CAPACITY              =      0.0100 VOL/VOL
WILTING POINT               =      0.0050 VOL/VOL
INITIAL SOIL WATER CONTENT  =      0.1226 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =   10.0000000000     CM/SEC
SLOPE                       =      2.50   PERCENT
DRAINAGE LENGTH             =   1550.0    FEET

                          LAYER  7
                         --------

             TYPE 4 - FLEXIBLE MEMBRANE LINER – BOTTOM HDPE
               MATERIAL TEXTURE NUMBER  35
THICKNESS                   =      0.06   INCHES
POROSITY                    =      0.0000 VOL/VOL
FIELD CAPACITY              =      0.0000 VOL/VOL
WILTING POINT               =      0.0000 VOL/VOL
INITIAL SOIL WATER CONTENT  =      0.0000 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =  0.199999996000E-12 CM/SEC
FML PINHOLE DENSITY         =      1.00   HOLES/ACRE
FML INSTALLATION DEFECTS    =      4.00   HOLES/ACRE
FML PLACEMENT QUALITY       =  3 - GOOD


                         LAYER  8
                         --------

               TYPE 3 - BARRIER SOIL LINER - GCL
               MATERIAL TEXTURE NUMBER  17
THICKNESS                   =      0.25   INCHES
POROSITY                    =      0.7500 VOL/VOL
FIELD CAPACITY              =      0.7470 VOL/VOL
WILTING POINT               =      0.4000 VOL/VOL
INITIAL SOIL WATER CONTENT  =      0.7500 VOL/VOL
EFFECTIVE SAT. HYD. COND.   =  0.300000003000E-08 CM/SEC
```

GENERAL DESIGN AND EVAPORATIVE ZONE DATA

NOTE: SCS RUNOFF CURVE NUMBER WAS COMPUTED FROM A USER-SPECIFIED CURVE NUMBER OF 77.0, A SURFACE SLOPE OF 2.% AND A SLOPE LENGTH OF 1550. FEET.

SCS RUNOFF CURVE NUMBER	=	75.10	
FRACTION OF AREA ALLOWING RUNOFF	=	0.0	PERCENT
AREA PROJECTED ON HORIZONTAL PLANE	=	75.080	ACRES
EVAPORATIVE ZONE DEPTH	=	14.0	INCHES
INITIAL WATER IN EVAPORATIVE ZONE	=	2.379	INCHES
UPPER LIMIT OF EVAPORATIVE STORAGE	=	5.614	INCHES
LOWER LIMIT OF EVAPORATIVE STORAGE	=	1.992	INCHES
INITIAL SNOW WATER	=	0.000	INCHES
INITIAL WATER IN LAYER MATERIALS	=	59.000	INCHES
TOTAL INITIAL WATER	=	59.000	INCHES
TOTAL SUBSURFACE INFLOW	=	0.00	INCHES/YEAR

EVAPOTRANSPIRATION AND WEATHER DATA

NOTE: EVAPOTRANSPIRATION DATA WAS OBTAINED FROM ROSWELL NEW MEXICO

STATION LATITUDE	=	32.52	DEGREES
MAXIMUM LEAF AREA INDEX	=	0.00	
START OF GROWING SEASON (JULIAN DATE)	=	76	
END OF GROWING SEASON (JULIAN DATE)	=	310	
EVAPORATIVE ZONE DEPTH	=	14.0	INCHES
AVERAGE ANNUAL WIND SPEED	=	8.70	MPH
AVERAGE 1ST QUARTER RELATIVE HUMIDITY	=	49.00	%
AVERAGE 2ND QUARTER RELATIVE HUMIDITY	=	40.00	%
AVERAGE 3RD QUARTER RELATIVE HUMIDITY	=	53.00	%
AVERAGE 4TH QUARTER RELATIVE HUMIDITY	=	52.00	%

NOTE: PRECIPITATION DATA WAS SYNTHETICALLY GENERATED USING COEFFICIENTS FOR ROSWELL NEW MEXICO

NORMAL MEAN MONTHLY PRECIPITATION (INCHES)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
0.20	0.37	0.76	1.78	0.35	2.35
2.62	2.26	2.83	1.67	0.77	1.10

NOTE: TEMPERATURE DATA WAS SYNTHETICALLY GENERATED USING COEFFICIENTS FOR ROSWELL NEW MEXICO

NORMAL MEAN MONTHLY TEMPERATURE (DEGREES FAHRENHEIT)

JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
45.80	48.40	53.50	62.00	71.50	78.30
80.50	78.80	71.90	62.60	54.00	43.10

NOTE: SOLAR RADIATION DATA WAS SYNTHETICALLY GENERATED USING COEFFICIENTS FOR ROSWELL NEW MEXICO AND STATION LATITUDE = 33.24 DEGREES

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ANNUAL TOTALS FOR YEAR 1

	INCHES	CU. FEET	PERCENT
PRECIPITATION	15.63	4259806.500	100.00
RUNOFF	0.000	0.000	0.00
EVAPOTRANSPIRATION	11.807	3217754.750	75.54
DRAINAGE COLLECTED FROM LAYER 4	0.0493	13437.409	0.32
PERC./LEAKAGE THROUGH LAYER 5	0.116716	31809.850	0.75
AVG. HEAD ON TOP OF LAYER 5	0.0001		
DRAINAGE COLLECTED FROM LAYER 6	0.1392	37947.406	0.89
PERC./LEAKAGE THROUGH LAYER 8	0.000000	0.062	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0004		
CHANGE IN WATER STORAGE	3.635	990667.312	23.26
SOIL WATER AT START OF YEAR	59.026	16087085.000	
SOIL WATER AT END OF YEAR	62.661	17077752.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	-0.502	0.00

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ANNUAL TOTALS FOR YEAR 2

	INCHES	CU. FEET	PERCENT
PRECIPITATION	19.70	5369045.500	100.00
RUNOFF	0.000	0.000	0.00
EVAPOTRANSPIRATION	15.173	4135138.250	77.02
DRAINAGE COLLECTED FROM LAYER 4	0.0000	0.000	0.00
PERC./LEAKAGE THROUGH LAYER 5	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 5	0.0000		
DRAINAGE COLLECTED FROM LAYER 6	0.0000	0.000	0.00
PERC./LEAKAGE THROUGH LAYER 8	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0000		
CHANGE IN WATER STORAGE	4.527	1233906.000	22.98
SOIL WATER AT START OF YEAR	62.661	17077752.000	
SOIL WATER AT END OF YEAR	67.189	18311658.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	1.559	0.00

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ANNUAL TOTALS FOR YEAR 3

	INCHES	CU. FEET	PERCENT
PRECIPITATION	19.61	5344517.000	100.00
RUNOFF	0.000	0.000	0.00
EVAPOTRANSPIRATION	16.520	4502324.000	84.24
DRAINAGE COLLECTED FROM LAYER 4	0.0000	0.000	0.00
PERC./LEAKAGE THROUGH LAYER 5	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 5	0.0000		
DRAINAGE COLLECTED FROM LAYER 6	0.0000	0.000	0.00
PERC./LEAKAGE THROUGH LAYER 8	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0000		
CHANGE IN WATER STORAGE	3.090	842191.625	15.76
SOIL WATER AT START OF YEAR	67.189	18311658.000	
SOIL WATER AT END OF YEAR	69.837	19033418.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.442	120431.141	2.25
ANNUAL WATER BUDGET BALANCE	0.0000	1.421	0.00

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ANNUAL TOTALS FOR YEAR 4

	INCHES	CU. FEET	PERCENT
PRECIPITATION	15.87	4325216.000	100.00
RUNOFF	0.000	0.000	0.00
EVAPOTRANSPIRATION	12.030	3278670.750	75.80
DRAINAGE COLLECTED FROM LAYER 4	0.0000	0.000	0.00
PERC./LEAKAGE THROUGH LAYER 5	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 5	0.0000		
DRAINAGE COLLECTED FROM LAYER 6	0.0000	0.000	0.00
PERC./LEAKAGE THROUGH LAYER 8	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0000		
CHANGE IN WATER STORAGE	3.840	1046544.690	24.20
SOIL WATER AT START OF YEAR	69.837	19033418.000	
SOIL WATER AT END OF YEAR	74.119	20200394.000	
SNOW WATER AT START OF YEAR	0.442	120431.141	2.78
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	0.918	0.00

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ANNUAL TOTALS FOR YEAR 5

	INCHES	CU. FEET	PERCENT
PRECIPITATION	17.20	4687695.000	100.00
RUNOFF	0.000	0.000	0.00
EVAPOTRANSPIRATION	16.790	4575959.500	97.62
DRAINAGE COLLECTED FROM LAYER 4	0.0000	0.000	0.00
PERC./LEAKAGE THROUGH LAYER 5	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 5	0.0000		
DRAINAGE COLLECTED FROM LAYER 6	0.0000	0.000	0.00
PERC./LEAKAGE THROUGH LAYER 8	0.000000	0.000	0.00
AVG. HEAD ON TOP OF LAYER 7	0.0000		
CHANGE IN WATER STORAGE	0.410	111732.172	2.38
SOIL WATER AT START OF YEAR	74.119	20200394.000	
SOIL WATER AT END OF YEAR	74.529	20312126.000	
SNOW WATER AT START OF YEAR	0.000	0.000	0.00
SNOW WATER AT END OF YEAR	0.000	0.000	0.00
ANNUAL WATER BUDGET BALANCE	0.0000	3.639	0.00

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AVERAGE MONTHLY VALUES IN INCHES FOR YEARS 1 THROUGH 5

	JAN/JUL	FEB/AUG	MAR/SEP	APR/OCT	MAY/NOV	JUN/DEC
PRECIPITATION						
TOTALS	0.14	0.22	0.83	1.27	0.35	1.63
	3.15	1.99	2.71	1.34	1.95	2.01
STD. DEVIATIONS	0.16	0.06	0.73	1.02	0.33	1.98
	1.47	1.38	1.72	1.54	2.39	1.39
RUNOFF						
TOTALS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
STD. DEVIATIONS	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	0.000	0.000	0.000	0.000	0.000
EVAPOTRANSPIRATION						
TOTALS	0.861	0.276	0.627	0.748	0.315	1.190
	2.588	2.051	2.100	1.326	1.309	1.075
STD. DEVIATIONS	0.610	0.099	0.663	0.668	0.236	1.643
	1.643	1.438	1.268	0.954	1.056	0.651
LATERAL DRAINAGE COLLECTED FROM LAYER 4						
TOTALS	0.0099	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0220	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
PERCOLATION/LEAKAGE THROUGH LAYER 5						
TOTALS	0.0233	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0522	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
LATERAL DRAINAGE COLLECTED FROM LAYER 6						
TOTALS	0.0278	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
STD. DEVIATIONS	0.0623	0.0000	0.0000	0.0000	0.0000	0.0000
	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

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  PERCOLATION/LEAKAGE THROUGH LAYER  8
  ------------------------------------
    TOTALS                 0.0000   0.0000   0.0000   0.0000   0.0000   0.0000
                           0.0000   0.0000   0.0000   0.0000   0.0000   0.0000

    STD. DEVIATIONS        0.0000   0.0000   0.0000   0.0000   0.0000   0.0000
                           0.0000   0.0000   0.0000   0.0000   0.0000   0.0000


-------------------------------------------------------------------------------
               AVERAGES OF MONTHLY AVERAGED DAILY HEADS (INCHES)
-------------------------------------------------------------------------------


  DAILY AVERAGE HEAD ON TOP OF LAYER  5
  -------------------------------------
    AVERAGES               0.0003   0.0000   0.0000   0.0000   0.0000   0.0000
                           0.0000   0.0000   0.0000   0.0000   0.0000   0.0000

    STD. DEVIATIONS        0.0008   0.0000   0.0000   0.0000   0.0000   0.0000
                           0.0000   0.0000   0.0000   0.0000   0.0000   0.0000

  DAILY AVERAGE HEAD ON TOP OF LAYER  7
  -------------------------------------
    AVERAGES               0.0010   0.0000   0.0000   0.0000   0.0000   0.0000
                           0.0000   0.0000   0.0000   0.0000   0.0000   0.0000

    STD. DEVIATIONS        0.0022   0.0000   0.0000   0.0000   0.0000   0.0000
                           0.0000   0.0000   0.0000   0.0000   0.0000   0.0000

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     AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS    1 THROUGH    5
-------------------------------------------------------------------------------
                                    INCHES            CU. FEET       PERCENT
                               -------------------  -------------   ---------
 PRECIPITATION                 17.60    (   1.967)   4797256.0     100.00

 RUNOFF                         0.000   (  0.0000)         0.00      0.000

 EVAPOTRANSPIRATION            14.464   (  2.4044)   3941969.50     82.171

 LATERAL DRAINAGE COLLECTED     0.00986 (  0.02205)     2687.482    0.05602
   FROM LAYER  4

 PERCOLATION/LEAKAGE THROUGH    0.02334 (  0.05220)     6361.970     0.13262
   LAYER  5

 AVERAGE HEAD ON TOP            0.000 (     0.000)
   OF LAYER  5 TOP HDPE

 LATERAL DRAINAGE COLLECTED     0.02785 (  0.06227)     7589.481    0.15820
   FROM LAYER  6

 PERCOLATION/LEAKAGE THROUGH    0.00000 (  0.00000)        0.012     0.00000
   LAYER  8 GCL

 AVERAGE HEAD ON TOP            0.000 (     0.000)
   OF LAYER  7

 CHANGE IN WATER STORAGE        3.100   (  1.5896)    845008.31     17.614
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PEAK DAILY VALUES FOR YEARS 1 THROUGH 5

	(INCHES)	(CU. FT.)
PRECIPITATION	2.17	591412.687
RUNOFF	0.000	0.0000
DRAINAGE COLLECTED FROM LAYER 4	0.01132	3085.25317
PERCOLATION/LEAKAGE THROUGH LAYER 5	0.014082	3837.95776
AVERAGE HEAD ON TOP OF LAYER 5	0.012	
MAXIMUM HEAD ON TOP OF LAYER 5	0.032	
LOCATION OF MAXIMUM HEAD IN LAYER 4 (DISTANCE FROM DRAIN)	0.0 FEET	
DRAINAGE COLLECTED FROM LAYER 6	0.02097	5714.49268
PERCOLATION/LEAKAGE THROUGH LAYER 8	0.000000	0.00367
AVERAGE HEAD ON TOP OF LAYER 7	0.023	
MAXIMUM HEAD ON TOP OF LAYER 7	0.061	
LOCATION OF MAXIMUM HEAD IN LAYER 6 (DISTANCE FROM DRAIN)	0.0 FEET	
SNOW WATER	3.22	878556.3120

MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.3730
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.1423

*** Maximum heads are computed using McEnroe's equations. ***

Reference: Maximum Saturated Depth over Landfill Liner
by Bruce M. McEnroe, University of Kansas
ASCE Journal of Environmental Engineering
Vol. 119, No. 2, March 1993, pp. 262-270.

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FINAL WATER STORAGE AT END OF YEAR 5

LAYER	(INCHES)	(VOL/VOL)
1	2.2447	0.1871
2	66.2103	0.2759
3	5.8560	0.2440
4	0.0020	0.0100
5	0.0000	0.0000
6	0.0020	0.0100
7	0.0000	0.0000
8	0.1875	0.7500
SNOW WATER	0.000	

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