NM1-63

Permit Application Vol 3

Part 4 of 8

10/12/16

APPLICATION FOR PERMIT
OWL LANDFILL SERVICES, LLC

VOLUME III: LANDFILL ENGINEERING CALCULATIONS
SECTION 8: DRAINAGE CALCULATIONS

ATTACHMENT III.3.A

DRAINAGE MANUAL, VOLUME I: HYDROLOGY.

NEW MEXICO STATE HIGHWAY AND TRANSPORTATION DEPARTMENT.

PHILIPS, CHRISTOPHER S.; EASTERLING, CHARLES M.; HEGGEN, RICHARD J.; AND SCHALL, JAMES D. 1995.

DRAINAGE MANUAL

VOLUME 1, HYDROLOGY

DECEMBER, 1995

NEW MEXICO STATE HIGHWAY
AND TRANSPORTATION DEPARTMENT
PRELIMINARY DESIGN BUREAU/DRAINAGE SECTION
P. O. BOX 1149
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FOREWORD

The New Mexico State Highway and Transportation Department Drainage Section is pleased to present a comprehensive update to its Drainage Manual. Volume 1 focuses on Hydrology and the prediction of flood flows at highway crossings. A companion document is presently under development which will address drainage structure hydraulics as well as sediment and erosion at highway structures. Together these documents will summarize and standardize methods by which drainage structures are designed for NMSHTD Projects. Comments regarding the content of this document are welcomed, and should be addressed to: Section Head, Drainage Section, NMSHTD, P.O. Box 1149, Santa Fe, NM 87504-1149.

Pete K. Rahn — 7-18-96

Pete K. Rahn, Secretary — Date
New Mexico State Highway and Transportation Department

NEW MEXICO STATE HIGHWAY AND TRANSPORTATION DEPARTMENT

DRAINAGE MANUAL

VOLUME 1, HYDROLOGY

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NMSHTD DRAINAGE MANUAL

VOLUME 1, HYDROLOGY

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1 INTRODUCTION

1.1 DRAINAGE MANUAL PURPOSE AND USE

The New Mexico State Highway and Transportation Department (NMSHTD) is responsible for the maintenance and construction of a vast network of roads throughout the State of New Mexico. Public safety and prudent investment of public funds in our road network requires that each facility be reasonably protected from a damaging flood. Standard methods of analysis and design have evolved over the past fifty years. Certain methods commonly used by the NMSHTD Drainage Section have proven their validity for use in New Mexico. This Manual summarizes those common methods which have a proven record for use in this state.

The standard methods of Hydrologic analysis presented in this Drainage Manual should be used for all NMSHTD 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 in this Drainage Manual, followed by a step by step procedure to apply the method. Example problems are included to assist the drainage designer. Limitations on the use of each analysis method are also included. This Drainage Manual does not include descriptions of the development or derivation of analysis methods. References are provided for the reader who wishes to review the source documents for each method.

This Drainage Manual specifies which hydrologic analysis method may be used for a particular drainage structure, based on drainage area size and location. By limiting the choice of hydrologic analysis method, a consistent and appropriate level of analysis is assured for every drainage structure, large and small. Despite these efforts to standardize methods, proper drainage analysis and design is not complete without the inclusion of competent engineering judgement. Drainage designers working on NMSHTD projects are expected to apply engineering judgement throughout the design development process. "Does this make sense? Will it work? What are the consequences of a failure? What is the risk associated with keeping the present structure?" These are the kinds of questions which complete the drainage design process once the analytic methods described in this Manual have been performed.

1.2 DRAINAGE DESIGN CRITERIA GUIDELINES

Drainage structures within the NMSHTD facilities network must be designed to meet certain minimum standards. Design frequency flood events are selected for each element of the highway drainage system. The magnitude of the design event is consistent with the highway classification, average daily traffic, user safety, risk, and consideration of economic impacts. Each drainage structure is designed to safely pass the appropriate design frequency flood without compromising the entire traveled way. The "appropriate" flood magnitude is a matter of public policy, balancing limited economic resources with the need to provide benefits to the greatest number of facility users. **The NMSHTD Policy on Drainage Design Criteria may be found in a separate document of the same title.** As a separate document from this Drainage Manual, it may periodically be revised to accommodate changes in public policy. Users of the NMSHTD Drainage Manual should obtain a current copy of the NMSHTD Policy on Drainage Design Criteria, so that drainage structures are designed for the appropriate flood magnitudes.

1.3 USE OF METRIC STANDARDS IN THE DESIGN OF NMSHTD PROJECTS

The NMSHTD endorses the use of metric or International System of Units (SI) for the analysis and design of NMSHTD projects. All of the drainage design procedures identified in this Drainage Manual were developed in the context of US Standard units of measurement. For this reason the discussion of different methods is provided using US Standard units where required. However, all hydrologic equations are provided in both SI and US Standard formats.

The Drainage Section of the NMSHTD has developed this edition of their Drainage Manual in recognition of the transition period which will occur in the near future. Designers will be increasingly required to perform hydrologic and hydraulic calculations in SI units. Under current guidelines, all projects must be analyzed and designed in SI units by the end of September, 1996. Many designers have developed personal rules of thumb and error checking procedures which are in US Standard units (CFS per acre, ft. per second, etc.). These important design procedures must be carried on in SI units, not abandoned. By providing a Drainage Manual which promotes the use of SI without discarding US Standard, the NMSHTD Drainage Section hopes to promote an orderly transition to SI.

In this transition period, all drainage engineers and designers working on NMSHTD projects are strongly encouraged to use SI units whenever possible in their analyses. Additional SI design aides will be disseminated by the Drainage Section as they become available. We welcome your suggestions for promoting a smooth transition to SI based design. Please send your written comments to: Chief, Drainage Section, NMSHTD, P. O. Box 1149, Santa Fe, NM 87504–1149.

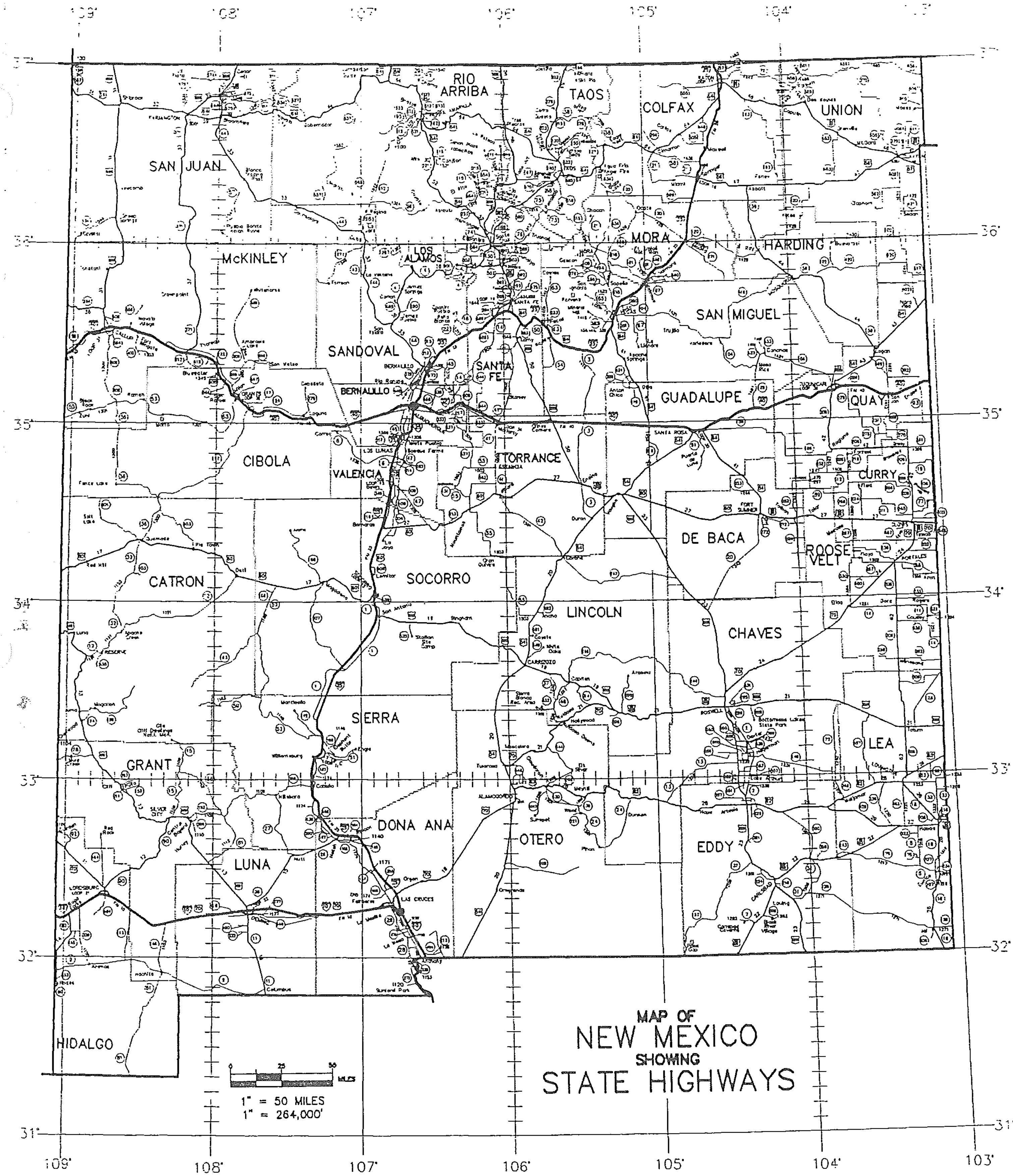


Figure 1-1
NMSHTD Facilities Map

2 *BASIC REQUIREMENTS FOR DRAINAGE STUDIES*

Drainage studies for NMSHTD projects must identify the hydrologic demands and hydraulic requirements of each drainage structure within the project limits. Each study will result in one or more drainage reports, summarizing the drainage improvements associated with the project. The drainage engineer's responsibility usually does not end with the drainage report. Staff engineers within the NMSHTD Drainage Section who prepare drainage reports will usually be responsible for drainage related permits (EPA, COE, FEMA), for development of a Sediment and Erosion Control Plan, and ongoing coordination with other NMSHTD Sections. Similar responsibilities may be required of consultants under contract with the NMSHTD. No matter what the total scope of services include, a drainage study and associated report(s) will be required. This section of the NMSHTD Drainage Manual describes the basic requirements of a drainage study for a NMSHTD project.

Most NMSHTD Projects include a standard set of project development milestones. These standard milestones are shown below. Drainage study elements are shown in bold text, identifying their location in the project development schedule. Specific requirements for these drainage study elements are described in the following sections.

<u>Typical Project Schedule</u>

- Preliminary Scoping Report
- Preliminary Field Review
    - **Drainage Field Inspection***
    - **Preliminary Drainage Report**
- Field Design Inspection
    - **Final Drainage Report**
- Grade and Drain Inspection
    - **Temporary Erosion and Sediment Control Plans**
- Plan in Hand
- Plans, Specifications & Estimate

*The drainage field inspection is sometimes combined with the Preliminary Field Review.

2.1 *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 the 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 existing structures and sizes, and evaluate the potential impacts of proposed drainage improvements. This is an opportunity for the drainage designer to field verify his or her preliminary design.

The basic elements of the drainage field inspection are listed below, with suggestions on things to look for and quantify in the field. The designer will probably develop a list of questions during the preliminary hydrologic analysis which need field verification. **Figure 2–1** is a field inspection form for drainage structures. This form should be copied and 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.

Field Inspection Suggestions

Watershed Conditions

- verify assumptions used in hydrologic analysis, including:
  - soil types, Hydrologic Soil Group (HSG)
  - land usage
  - vegetation and ground cover density
  - percent impervious
- evidence of flow diversions, stock ponds, etc. not accounted for in analysis

Existing Structures

- measure actual structure sizes, wall thickness, etc.
- identify actual locations: use mileposts, stations from as-built plans, distance meters, etc.
- structural condition: look for rust, spalling, cracks, deformed cross section
- structure subsidence: is the vertical alignment okay?
- evidence of outlet erosion and/or inlet sedimentation
- upstream high water marks: (when estimating the magnitude of flow events, an
  - approximate discharge can be calculated using the Slope – Area method)
- evidence of debris accumulation
- channel geometry upstream and downstream
- effectiveness of structure skew, inlet/outlet geometry
- does the existing structure appear capable of passing the design flow?
  - if not, what will happen? roadway overtopping? backwater onto adjacent properties?

Onsite Drainage Facilities (within the Right–of–Way) – evaluate how they are functioning

- roadside ditches: vegetation, ditch erosion, cut slopes erosion
- median swales working
- rundowns still working properly
- area inlets and catch basins working
- curbs, gutters diverting flows to desired locations
- is the pavement section being drained adequately?
- erosion of an embankment by pavement runoff

Interview NMSHTD Patrol Foreman

- identify inadequate drainage facility locations
- describe location and magnitude of major flow events
- discuss maintenance procedures including
  - standard practices
  - specific problem spots
  - frequency and timing of maintenance work
- list improvements suggested by Patrol Foreman

Interview Other Individuals as Required – State Police, local property owners, etc.

- be sure to get names, and date of interview

Evaluate Proposed Drainage Improvements

- does the proposed structure seem reasonable?
- does the upstream conveyance reflect the design flow?
- will a backwater condition adversely impact adjoining landowners?
- can the inlet condition be improved with trainer dikes?
- consider the proposed road section and profile for impacts to
  - structure extensions and resulting inlet/outlet locations
  - special designs for high fills or minimal cover conditions
- how will future maintenance operations be affected?
- would a different type of structure improve passage of sediment or debris?
- are debris control measures required?
- are additional drainage improvements needed?
- effectiveness of proposed skew angle

Evaluate Effectiveness of Maintenance Work

- is the pavement surface able to drain effectively?
- does water pond next to the pavement?
- are structure inlets obstructed with debris?
- do grading operations increase ditch or shoulder erosion?

Individual designers will undoubtedly come up with other questions to be answered in the drainage field inspection. However, these suggestions provide a basic list of items which should be evaluated in the field on each NMSHTD project.

Drainage Structure
Field Inspection Form

Verify Watershed Conditions

Land Use ______________________
Vegetation Type ______________________
Verify – Effective Drainage Area ________
Stock Ponds or Detention Facilities ________
Hydraulic Improvements ______________
Percent Cover ______________
Upstream Diversions ______________
Percent Impervious ______________
Other Comments ____

Structure Type

Size or Span ______________________
Clear Height ______________________
# of Piers or Barrells ____ ________
Invert to Pavement Height ________
Structure Skew ____________ Structure Slope ____________ Pier Type ________
Evidence of, Bridge Scour ______ Bed Lowering ____________ Bed Material ______
General Condition of Structure ______________________________________
Erosion ________ Spalling ________ Cracking ________ Barrell Deformation ______
Other Comments: ____

Structure Inlet Conditions

Wingwalls ____________ Headwalls ________ Training Dikes ______ Height ________
Upstream Channel ______ Bottom Width ______ Sideslopes ______ Longitudinal Slope ______
Evidence of, Debris ______ Sediment Deposition ______ Bank Caving ______ Headcutting ______
Evidence of, Ponding ________ Highwater Marks ________ Maintenance ____________
Channel Bed Material ____________ Channel Capacity Similar to Structure Capacity ______

Structure Outlet Conditions

Wingwalls ____________ Headwalls ________ Training Dikes ______ Height ________
Outlet Apron ______ Length ______ Erosion Control Measures ______ Length ________
Evidence of, Erosion at Outlet ________ Downstream Channel Instability ____________

General Conditions

Calculated Peak Design Flow ________ Is This Reasonable? ______________
Evidence of Flood Damage to Adjacent Properties ______________
Evidence of Stream Instability Effecting Adjacent Properties ______________
Irrigation Facilities Affected ______________________________
Environmental Hazards Present ______________________________
Photos Taken of: ______________________________
Survey Required: ______________________________
Items to Research Back at the Office: ______________________
Other Comments: ______________________________
____

Project Location:______________________________
CN#:______________________________
Date:______________________________
Inspected by:______________________________
Structure Location:______________________________
Project Station:______________________________

Figure 2-1
Drainage Structure Field Inspection Form

2.2 *PRELIMINARY DRAINAGE REPORT*

The preliminary drainage report should summarize the results of the preliminary drainage analysis. Structure Size recommendations will be reviewed by the NMSHTD Drainage Section, and will be used for field design plans by the Highway Design Section. Basic elements which should be included in the preliminary drainage report are listed below.

- Project Name, locaton, Project Control Number, etc.
- Drainage area topographic map with structure locations identified
- Identify soil types, vegetation and land use distribution
- Curve Number or Rational Formula "C" calculations
- Time of Concentration calculations
- Summarize the drainage field inspection results, including patrol foreman interview
- Drainage Structure Field Inspection forms
- Summary Table of existing and recommended drainage structure sizes and types
- Identify data sources used in the analysis

The preliminary drainage report should not include detailed print outs from hydrologic or hydraulic analyses. However, data generated in the analysis process should be kept on file and made available to the NMSHTD Drainage Section when requested.

2.3 *FINAL DRAINAGE REPORT*

The Final Drainage Report is basically a refinement of the Preliminary Drainage report. The Final Drainage Report is not begun until receipt of the preliminary design from the Highway Design Section. The preliminary highway design data must include: preliminary plan and profile sheets, with preliminary grade, typical roadway sections, toe of slope lines, and drainage structure survey data. Modifications to the preliminary hydrologic analysis 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. Permanent erosion protection design is completed, including riprap design, drainage structure outlet design and analysis of scour depths at critical locations. For watersheds producing high sediment loads, an estimate of upstream sediment transport and sediment continuity at the highway crossing structure may be required.

2.4 *TEMPORARY EROSION AND SEDIMENT CONTROL PLAN*

Design of temporary erosion and sediment control measures is not included in the preliminary or final drainage report. The drainage designer should refer to the document "National Pollutant Discharge Elimination System Implementation Package," prepared by the NMSHTD. Contact the NMSHTD Drainage Section in Santa Fe for further information.

3 *Hydrology*

3.1 *NMSHTD Approach to Hydrologic Analysis*

The New Mexico State Highway and Transportation Department must provide transportation facilities which are reasonably safe for the public. A safe roadway environment includes properly designed drainage structures. The NMSHTD must design drainage structures to meet minimum design standards, and must do so within certain budgetary constraints. Current minimum design standards for drainage facilities can be found in the document "**Drainage Design Criteria for NMSHTD Projects**." This document is available from the NMSHTD Drainage Section, in Santa Fe.

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

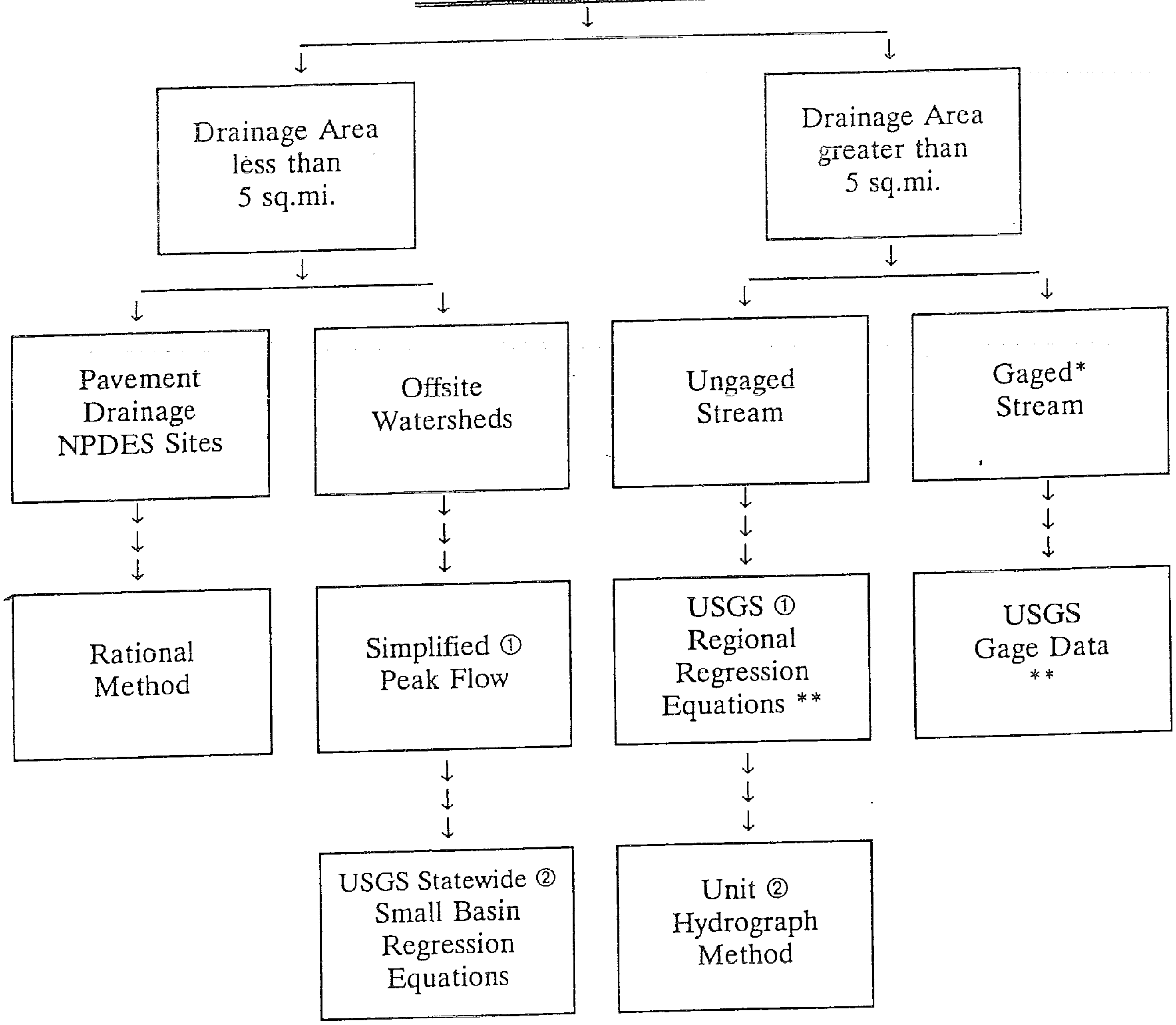
It is the goal of the NMSHTD Drainage Section to standardize the hydrologic analysis methods used on NMSHTD projects, requiring the use of standard methods which have a demonstrated performance record in this state. Many hydrologic analysis methods have been used in New Mexico with widely varying results. 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. By standardizing hydrologic analysis methods, a significant amount of confusion and debate will be removed from drainage analyses performed on NMSHTD projects. Guidelines for the use of NMSHTD approved hydrologic analysis methods are provided in this manual, along with visual aides to promote consistency in the selection of curve numbers.

3.2 *Selection of a Hydrologic Method*

The NMSHTD Drainage Section has established certain hydrologic analysis methods to be used on NMSHTD projects. Methods are selected based on drainage area size, and whether or not the highway facility is located in an Urban or Rural area. In general, NMSHTD personnel and consultants to the NMSHTD are required to use the hydrologic methods specified below. The NMSHTD Drainage Section may allow other hydrologic analysis methods to be used, depending on project specific circumstances. **Contact the Drainage Section and obtain approval before using a method other than those specified below.**

Figures 3–1 and **3–2** are used to select the appropriate hydrologic method for a particular drainage structure. When two or three methods are applicable, the order of preference is shown by a small symbol, ①. In areas where a local government agency has a drainage policy which mandates a specific hydrologic analysis method, that hydrologic analysis method shall be used on NMSHTD projects. For example, the AHYMO model using the COMPUTE NMHYD routine is approved for use in Albuquerque, but not in Roswell. When a particular drainage basin is borderline between two size categories, the more detailed analysis method shall be used. At the discretion of the designer, the Unit Hydrograph Method can be substituted for the Simplified Peak Flow method.

RURAL CONDITIONS



* Only gage data from USGS gages will be allowed for use on NMSHTD Projects.

** The NMSHTD may require designers to provide a supplementary Unit Hydrograph calculation for comparison purposes.

Figure 3-1
Methodology Selection
Flow Chart
Rural Conditions

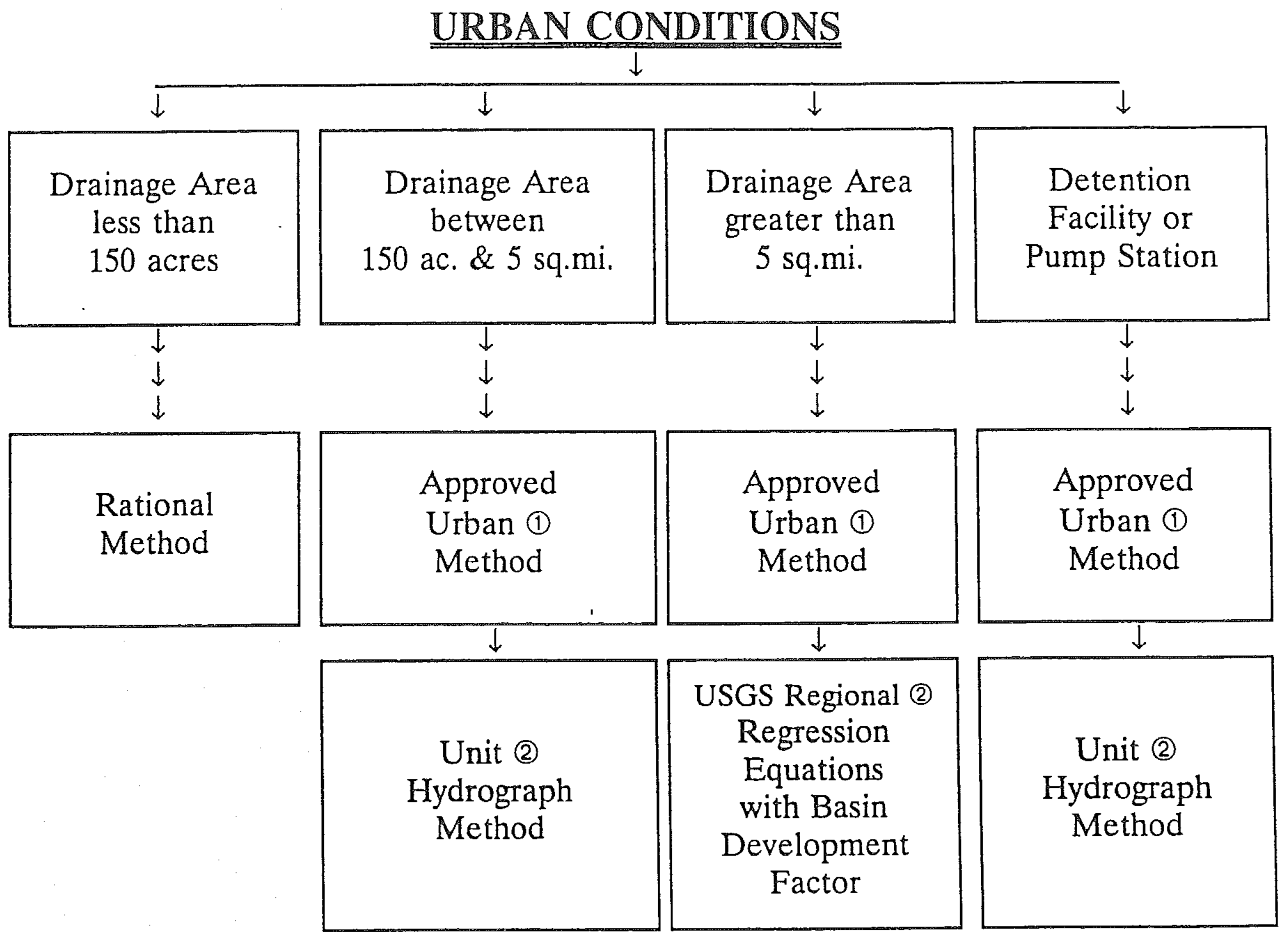


Figure 3–2
Methodology Selection
Flow Chart
Urban Conditions

3.3 DRAINAGE BASINS WITHOUT GAGE DATA

The vast majority of drainage structures on New Mexico highways pass flows from watersheds for which there is no measured data on rainfall or runoff. Peak rates of runoff and runoff volumes must therefore be estimated using analytical or parametric (regression) methods. Designers using this manual need not be proficient in statistical analysis procedures. The regression methods specified herein have been developed by the United States Geological Survey (USGS) and can be quickly applied. The analytic methods adopted for use by the NMSHTD are commonly accepted methods which have been used successfully in New Mexico. The Rational Formula is used for very small watersheds. SCS methods including the Unit Hydrograph procedure and the Peak Rate of Discharge for Small Watersheds are used for larger watersheds. In urban areas where established drainage policy dictates a particular hydrologic analysis method, analysis of drainage structures within that jurisdiction will follow the local established method.

Use of specific rain gage data will generally not be allowed on NMSHTD projects. Instead, rainfall data from the National Oceanic and Atmospheric Administration (NOAA) Precipitation – Frequency Atlas (Miller et al, 1973) will be used*. The purpose of this exclusion is to promote the use of regionally adjusted rainfall data, in lieu of reliance on data from a single location. Regional regression analysis techniques were used by NOAA to smooth the delineation of equal precipitation areas, removing some of the uncertainty associated with a single gage location. Use of regionalized rainfall data is particularly important in New Mexico where rainfall can vary dramatically from one location to another nearby location.

3.3.1 GENERAL DATA FOR HYDROLOGIC ANALYSIS

Certain characteristics of each drainage basin must be quantified to estimate peak rates of runoff and runoff volumes. Size of a drainage basin is always important. The quantity of rainfall is also important. The time distribution and intensity of rainfall has a direct effect on the rate of runoff. Rainfall lost to ground infiltration, localized depression ponding or plant absorbsion means less water available for runoff. The slope of the watershed and development of stream channels affects how fast runoff can reach the drainage structure. The following sections of this manual describe these factors in greater detail, and how to quantify them for use in each hydrologic analysis method.

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*The NOAA Rainfall Atlas is currently being revised (1995). Updated NOAA rainfall data will be used on NMSHTD projects once the revised Atlas is publicly available.

3.3.1.1 Drainage Basin Delineation

Drainage basins are usually defined graphically using topographic maps. USGS topographic maps at 1:24,000 scale provide adequate detail for NMSHTD projects and are available for most areas of New Mexico. Drainage structures crossing highways are usually located at low spots in the terrain, and are always provided where a stream channel exists. From the drainage structure location, drainage basin boundaries are drawn on the topographic map 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 runs downhill. Drainage basin boundary lines are generally drawn perpendicular to the topographic lines, following the ridgetops.

Once the overall drainage basin has been defined, the total drainage area should be measured. A planimeter is commonly used to measure areas from topographic maps. Drainage basin areas may also be measured electronically by digitizing map areas. Some USGS maps are now available in digital format. The historical grid method may also be used, where the basin map is overlaid with a transparent grid and grid rectangles are counted within the basin boundary lines.

Each drainage basin should be qualitatively assessed as follows:

- What hydrologic analysis method is required based on drainage basin size?

- Is one drainage basin okay for analysis purposes, or should we create sub–basins? Considerations might include: drastic changes in land slope, land use and development.

- Is the overall drainage basin shape somewhat consistent with implicit assumptions built into the analytical design methods? **Figure 3–3** shows the effects on hydrograph shape from different drainage basin shapes. The designer should consider subdividing drainage basins which are particularly elongated or short and wide.

- Will roads, diversions, ponds or other features within the drainage basin prevent it from behaving as a uniform, homogeneous watershed?

- In flat terrain, are there roads or other development features which act as drainage divides?

When these factors are accounted for, parameters such as Time of Concentration and Runoff Curve Number will more accurately portray the runoff response of the watershed.

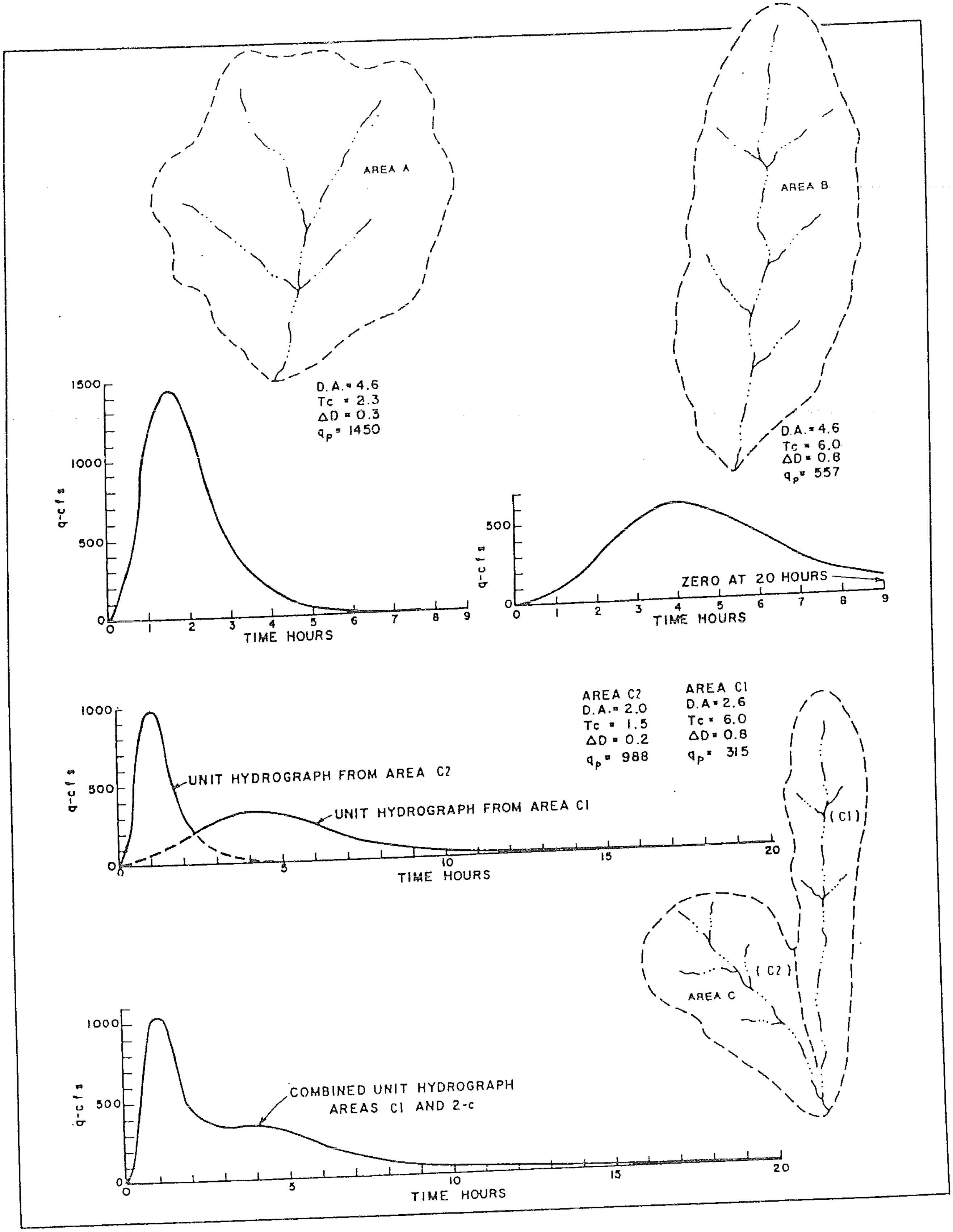


Figure 3-3
Effect of Drainage Basin Shape on Hydrograph Shape

Adapted from SCS, NEH-4, 1972

3.3.1.2 Rainfall

Rainfall data is a necessary input parameter for nearly all runoff computations performed on NMSHTD projects. The quantity of rainfall and the time distribution of the rainfall will both affect the resulting peak rate of runoff. Rainfall data is taken from the NOAA Precipitation – Frequency Atlas (Miller et al, 1973) or from updated NOAA maps when they become available. **Figures E–1 through E–12** in *Appendix E* of this manual provide the same NOAA data (1973) with a current (1995) State Highway map. Point precipitation values may be read from these **Figures** for the design rainfall event.

The designer must first determine the return frequency of the design flood to be used on a particular project or drainage structure. Design frequency floods are listed in a separate document, "**Drainage Design Criteria for NMSHTD Projects**," which may be obtained from the NMSHTD Drainage Section. Design frequencies are not included in this manual because the design criteria may change over time. **Designers should verify that they have the latest Drainage Design Criteria before proceeding with design on NMSHTD projects.**

For NMSHTD 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 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, the designer should evaluate both a rainfall event and a snowmelt/rainfall event as predicted by the USGS rural peak discharge regression equations.

3.3.1.2.1 Rainfall in the Rational Formula

Rainfall data must be transformed into an Intensity–Duration–Frequency (IDF) relationship for use in the Rational Formula. Rainfall intensity, i, has units of inches/hour, and changes with the Time of Concentration and design frequency. Specific IDF curves must be prepared for each NMSHTD project location. Generalized IDF curves should not be used. A manual procedure for preparing IDF curves is described below. A computer spreadsheet is used by the NMSHTD Drainage Section to expedite these calculations.

Manual IDF Procedure:

Step 1

Obtain the 6–hour and 24–hour point precipitation depths from **Figures E–1 through E–12**, or from the current NOAA Atlas. 2–year and 100–year depths are required, along with other return periods needed for the drainage analysis. Enter the values in the Depth–Duration–Frequency (DDF) Worksheet (**Figure 3–4**). Designers should make blank copies of the DDF/IDF Worksheet and the IDF Graph for use on different projects.

	2–yr	5–yr	10–yr	25–yr	50–yr	100-yr
5–min						
10–min						
15–min						
30–min						
1–hr						
2–hr						
3–hr						
6–hr						
12–hr						
24–hr						

Note: Cross hatching denotes which values are being entered in the DDF matrix of **Figure 3–4**.

3.3.1.3 *Rainfall Losses and Runoff Curve Numbers*

Runoff curve numbers are used to quantify rainfall losses such as infiltration, interception and depression storage. Curve numbers are required input for the SCS rainfall runoff models used in this manual: Simplified Peak Flow and SCS Unit Hydrograph methods. In practice, curve numbers range from about 40 to 100, with larger curve numbers representing more runoff. Factors such as land use, ground cover type, hydrologic condition and hydrologic soil group are used to select a curve number.

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 watershed for the assumed watershed conditions. It is very important that the designer consider what changes will occur in the watershed during the year. The NMSHTD cannot design for anticipated changes in development. However, the designer should account for seasonal variations in vegetation and ground cover. The condition of the watershed may vary dramatically from the date of field reconnaissance to the annual season of largest historic runoff. This problem is most evident in cultivated agricultural areas 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 3) the crop type may be changed from year to year. **The designer must exercise engineering judgement to determine the appropriate runoff curve number for a particular drainage basin or sub–basin.**

3.3.1.3.1 *Curve Number Selection*

Primary factors used in the selection of a curve number are described below. The designer must evaluate the watershed in terms of these factors to select an appropriate curve number. Tabulated curve number values are provided in this manual and may also be found in several SCS publications (SCS, 1986). A graphic method for selecting curve numbers in rural areas is provided in **Figure 3–8**. As an additional resource, photographs of different land uses and ground cover types are provided in *Appendix A*.

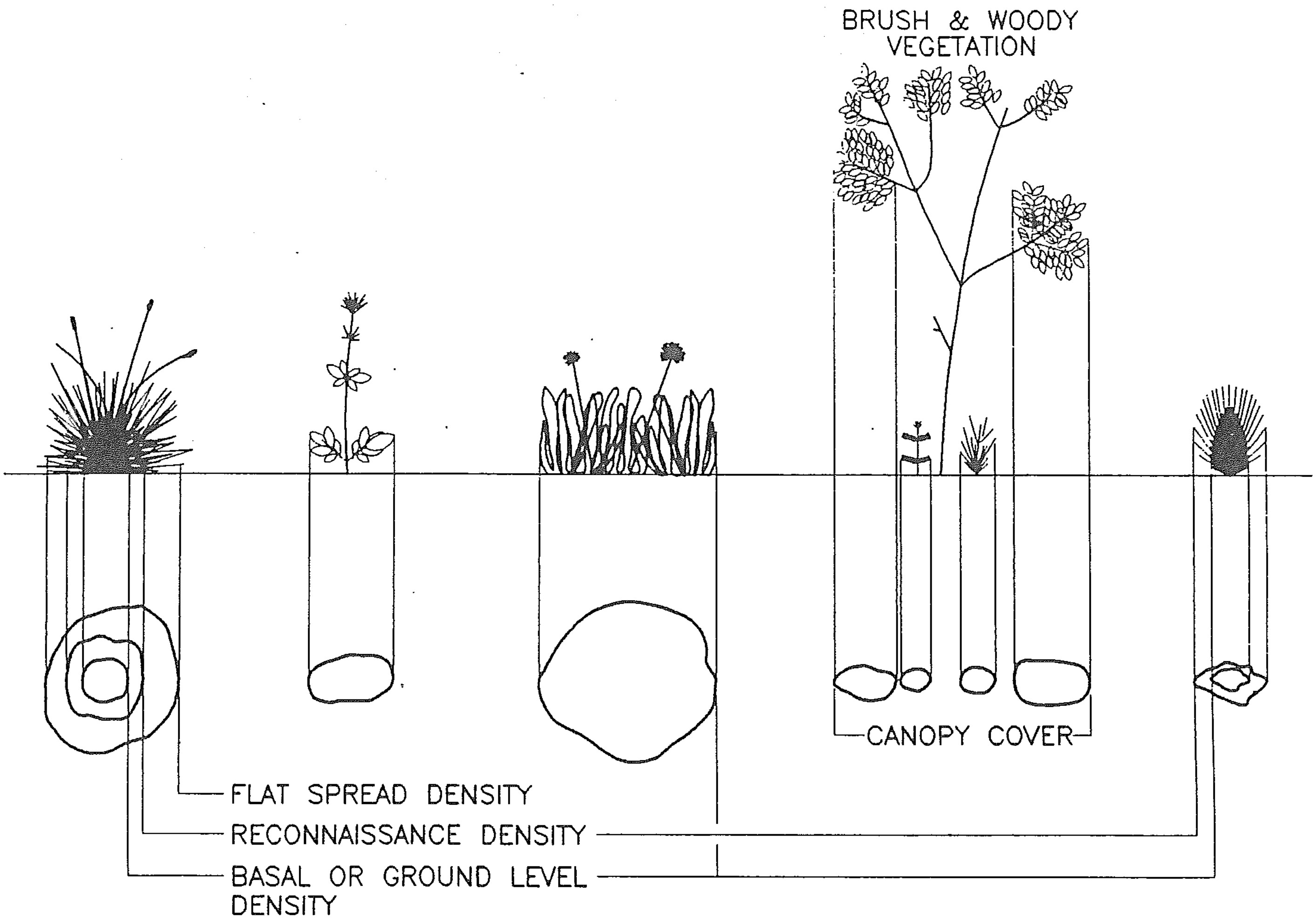
Land Use – categorizes the land into several broad categories of usage, including rangeland, agricultural and urban. Land use is further subdivided by ground cover type and hydrologic condition. Particularly for agricultural land use, the land treatment can be a major consideration (i.e. terracing, crop rotation, etc.). In areas of human activity, compaction of natural soils may change the runoff response. For urban areas the density of development, type of landscaping, treatment of idle land and network of drainage conveyances should all be considered.

Ground Cover Type and Cover Density – describes the type of vegetation in the watershed. Arid rangeland areas may have weeds, grasses, sagebrush, desert shrubs, etc. Areas of greater rainfall may have piñon–juniper, continuous grasses, deciduous or coniferous woods, etc. Agricultural lands may be in pasture, in crops, fallow, etc. In urban areas the ground cover type is closely related with the land use. The percentage of impervious area is the most important factor in urban areas. **Figure 3–9** provides a method for adjusting curve numbers to reflect the percent impervious area. Designers should assume that all of the impervious area is "connected." In rural and agricultural areas the ground cover density has a big effect on the runoff response of the watershed. For these areas the designer must estimate ground cover type and density at the time of year when large runoff events are most likely to occur. **Figure 3–7** shows how to estimate ground cover density.

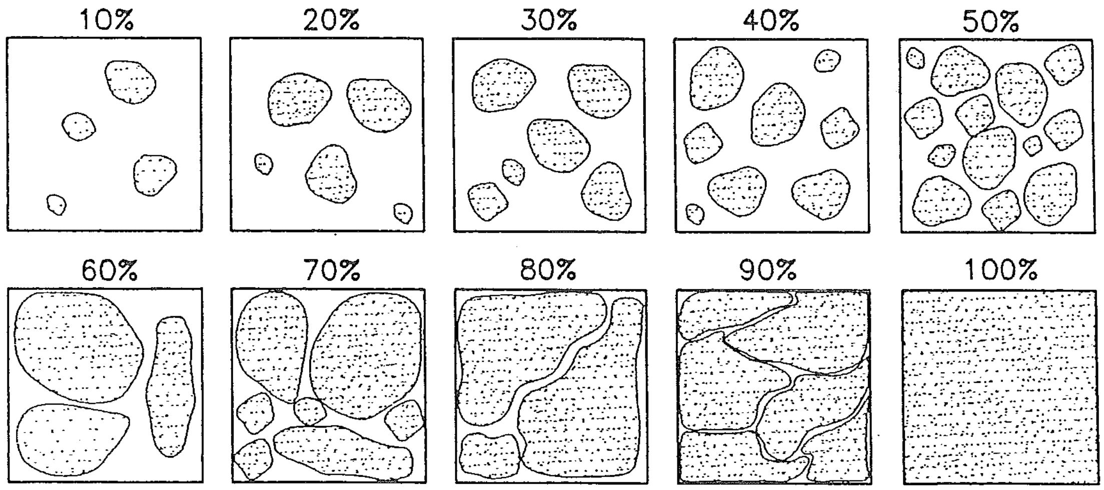
Hydrologic Condition – a "poor" hydrologic condition indicates impaired infiltration and therefore increased runoff. A "good" hydrologic condition indicates factors which encourage infiltration. For agricultural lands the hydrologic condition is a combination of factors including percent ground cover, canopy of vegetation, amount of year–round cover, percent of residue cover on the ground, grazing usage, and degree of roughness. For arid and semi–arid lands the percent ground cover determines the hydrologic condition.

Hydrologic Soil Group – categorizes the surface and subsurface soils in terms of their ability to absorb water. Sandy soils tend to fall into group "A," whereas clay soils and rock outcrops are usually in the "D" group. "A" soils are relatively permeable whereas "D" soils are not. SCS Soil Surveys include aerial photograph maps of soil series, and for each series a hydrologic soil group has been assigned. SCS Soil Surveys are available by county for the majority of New Mexico. Most of the soil surveys were performed through aerial photo interpretation of large areas and detailed field inspections at selected locations. In watershed areas where excavation or extensive reworking of the surface soils has occurred, the designer should use field inspections to confirm the hydrologic soil group of the present surface soils.

Antecedent Moisture Condition (AMC) – describes the amount of moisture in the soil at the time rainfall begins. Antecedent moisture is categorized into three conditions: dry (I), average (II) and wet (III). **Tables 3–1 through 3–4** list curve number values for various land use categories and average AMC. The assumption of AMC = II is valid for design watershed conditions on NMSHTD projects. For arid lands, an AMC of II may appear conservative, but represents conditions which could reasonably occur in conjunction with the design rainfall event. Occasionally a different AMC may be considered on a specific project. When required, the curve number for an average AMC may be adjusted as shown in **Table 3–5**.

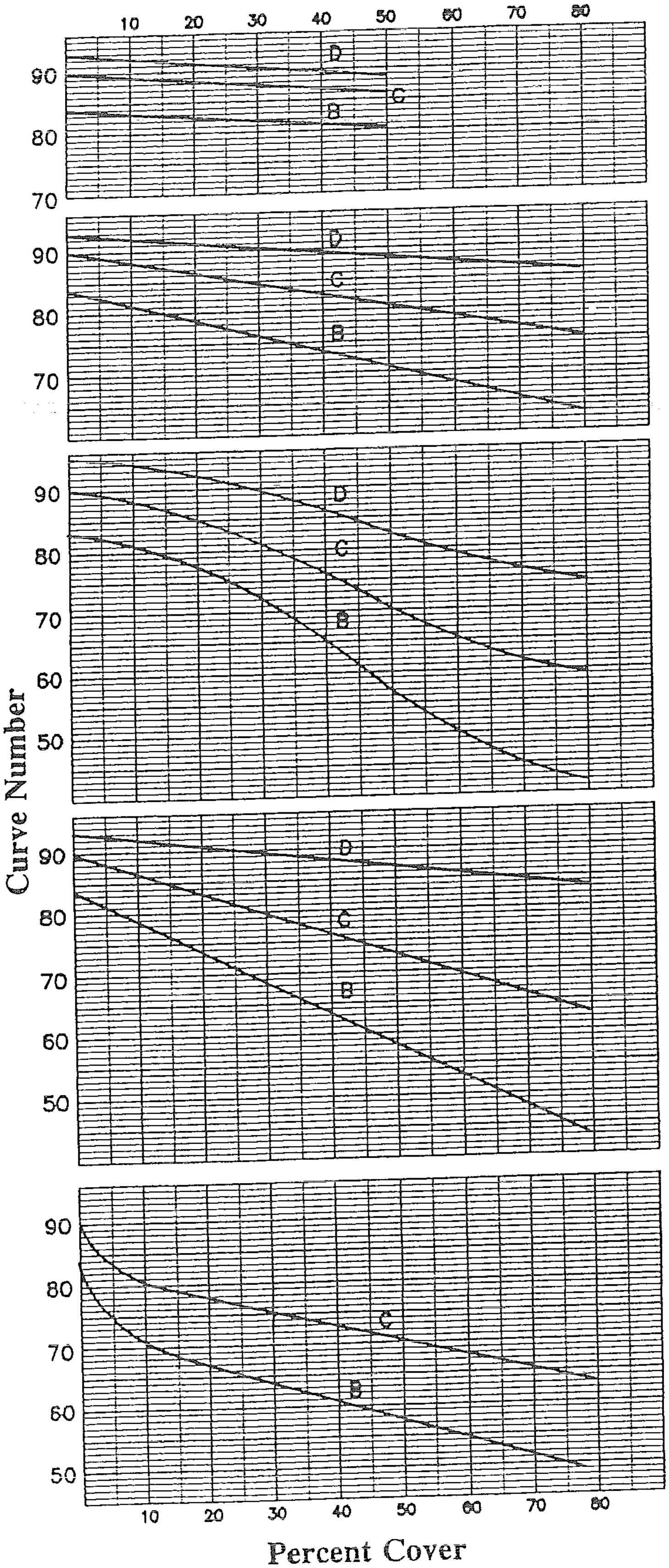


TYPES OF COVER DENSITIES FOR GRASSES, WEEDS, AND BRUSH.
USE BASAL DENSITIES FOR DESIGN



STANDARD METHOD OF MEASURING GROUND COVER DENSITY

Figure 3–7
Estimating Ground Cover 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.

Figure 3–8
Hydrologic Soil – Cover Complexes and Associated Curve Numbers

Adapted from SCS, Chapter 2 for NM, 1985

Table 3–1 — Runoff Curve Numbers for Arid and Semiarid Rangelands[1]
Source: USDA SCS, TR–55, 1986

Cover Description		Curve Numbers for Hydrologic Soil Group –			
Cover Type	Hydrologic Condition[2]	A[3]	B	C	D
Herbaceous—mixture of grass, weeds, and	Poor		80	87	93
low growing brush, with brush the	Fair		71	81	89
minor element.	Good		62	74	85
Oak–aspen—mountain brush mixture of oak	Poor				
brush,	Fair		66	74	79
aspen, mountain mahogany, bitter brush, maple,	Good		48	57	63
and other brush.			30	41	48
Piñon, 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,	Poor	63	77	85	88
greasewood, creosotebush, blackbrush, bursage,	Fair	55	72	81	86
palo verde, mesquite, and cactus.	Good	49	68	79	84

[1] Average runoff condition.

[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.

Table 3-2 — Runoff Curve Numbers for Cultivated Agricultural Lands[1]

Source: USDA SCS, TR-55, 1986

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.

[2] Crop residue cover applies only if residue is on at least 5% of the surface throughout the year.

[3] Hydrologic condition is based on combination of 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 in rotations, (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.

Table 3-3 — Runoff Curve Numbers for Other Agricultural Lands[1]
Source: USDA SCS, TR-55, 1986

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–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.

[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 3-4 — Runoff Curve Numbers Urban Areas[1]

Source: USDA SCS, TR-55, 1986

Cover Description		Curve Numbers for Hydrologic Soil Group –			
Cover Type and Hydrologic Condition	Average Percent Impervious Area[2]	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
Vacant lands (CN's are determined using cover types similar to those in **Table 3-3**).					

[1] Average runoff condition.

[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 3.9.

[3] CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.

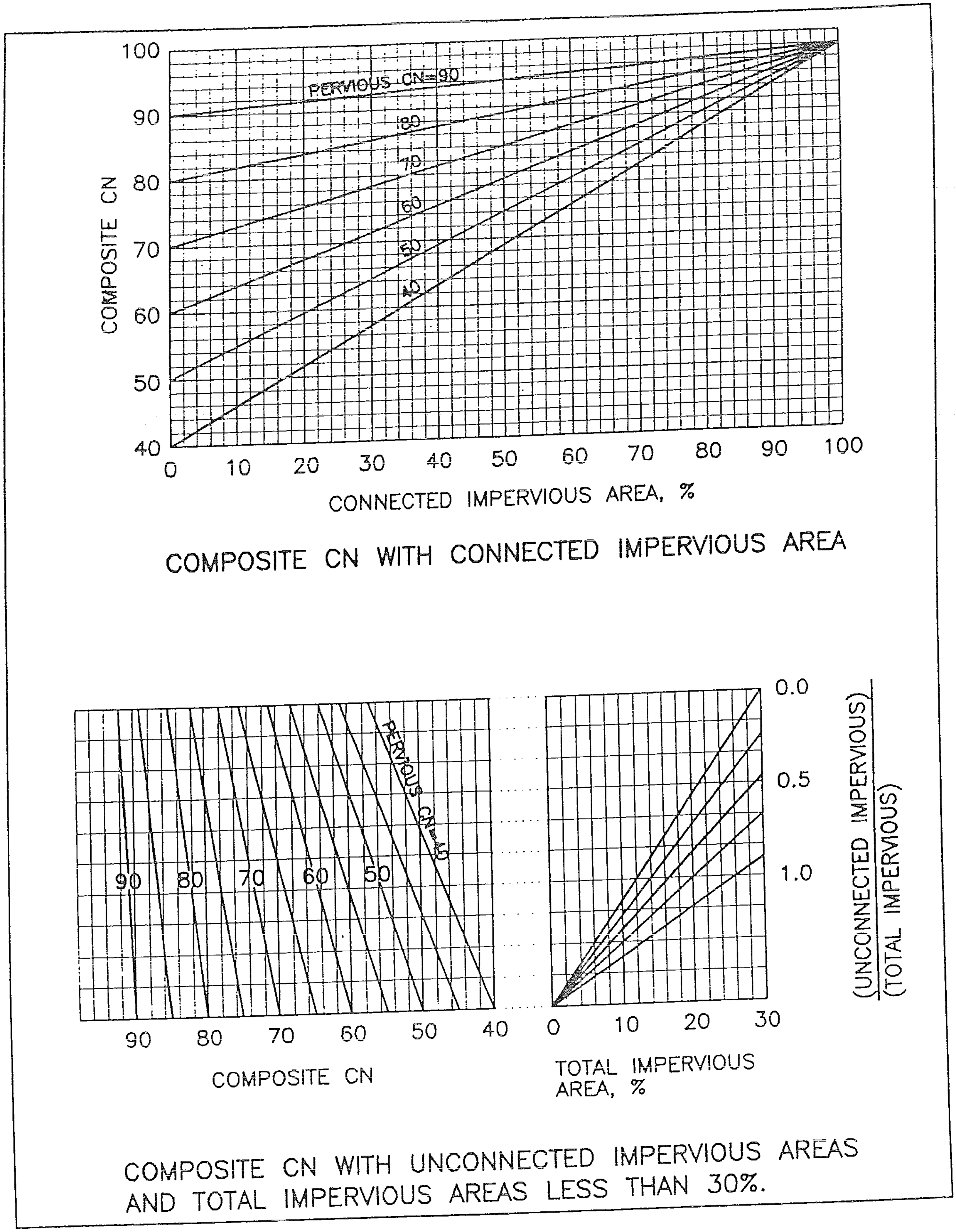
[4] Composite CN's for natural desert landscaping should be computed using Figure 3.9 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.

[5] Composite CN's to use for the design of temporary measures during grading and construction should be computed using Figure 3.9, based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.

Table 3-5 — Conversion from Average Antecedent Moisture Conditions to Dry and Wet Conditions

Source: USDA SCS, TR-55, 1986

CN for Average Conditions	Corresponding CN's for	
	Dry	Wet
100	100	100
95	87	98
90	78	96
85	70	94
80	63	91
75	57	88
70	51	85
65	45	82
60	40	78
55	35	74
50	31	70
45	26	65
40	22	60
35	18	55
30	15	50
25	12	43
15	6	30
5	2	13



COMPOSITE CN WITH CONNECTED IMPERVIOUS AREA

COMPOSITE CN WITH UNCONNECTED IMPERVIOUS AREAS AND TOTAL IMPERVIOUS AREAS LESS THAN 30%.

Figure 3-9
Composite CN for Urban Areas with Connected and Unconnected Impervious Areas

Adapted from SCS, TR-55, 1986

3.3.1.3.2 *CURVE NUMBER WEIGHTING*

When hydrologic conditions are consistent throughout the watershed, then use of a single curve number is appropriate. For watersheds where curve numbers vary by 10 or less, an area weighted curve number is sufficient. When curve numbers vary dramatically within the watershed, the designer should consider subdividing the watershed into different drainage sub–basins. An alternative to subdividing a highly variable drainage basin is to use a Runoff weighted curve number. Examples of each curve number weighting procedure are shown below.

Area Weighted Curve Number

40% of the drainage basin is characterized by CN = 65
60% of the drainage basin is characterized by CN = 73

the area weighted $$CN = \frac{(.40)\ (65) + (.60)\ (73)}{1.00} = 69.8$$ use CN = 70

Runoff Weighted Curve Number

40% of the drainage basin is characterized by CN = 88
60% of the drainage basin is characterized by CN = 72

Assume a design rainfall event of 2.0 inches.

Use **Figure 3–16** to estimate
1.0 inches of direct runoff from the CN = 88 land
and 0.3 inches of direct runoff from the CN = 72 land
the average runoff is calculated as

$$\frac{(.40)\ (1.0) + (.60)\ (.03)}{1.00} = 0.58\ \textit{inches}$$ average direct runoff

Use **Figure 3–16** to find a
runoff weighted curve number of CN = 80

Comparison of Methods

Recall that by the area weighted method we would have obtained a CN = 78.
The difference in this example is approximately 0.1 inches of direct runoff. This difference becomes particularly important for small rainfall amounts where lower CN values may not predict any runoff. In the example above a curve number difference of 2 resulted in a

$$\frac{0.58 - 0.50}{.50} = .16$$

the runoff weighted curve number predicts a 16% increase in runoff.

Use the criteria described above to select the best weighting method.

3.3.1.4 *TIME OF CONCENTRATION*

Time of Concentration is defined as the time required for runoff to travel from the hydraulically most distant part of the watershed to the point of interest. Time of concentration is one of the most important drainage basin characteristics needed to calculate the peak rate of runoff. An accurate estimate of a watershed's time of concentration is crucial to every type of hydrologic modeling.

The method used to calculate time of concentration must be consistent with the method of hydrologic analysis selected for design. Designers working on NMSHTD projects must use the time of concentration methods specified in this section for each hydrologic method. Mixing of methods is not allowed on NMSHTD projects. **Table 3–6** defines the correct time of concentration method to be used for each hydrologic method.

Within each watershed the designer must locate the primary watercourse. This is the watercourse that extends from the bottom of the watershed or drainage structure to the most hydraulically remote point in the watershed. Most designers begin at the bottom of the watershed and work their way upstream until the longest watercourse 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 gather together until a defined stream channel is formed. The water course is now large enough to be identified on a quadrangle topographic map.

Sections along the primary watercourse should be identified which are hydraulically similar. Time of concentration is estimated for each section of the watercourse. Time of concentration in any given watershed is simply the sum of flow travel times within hydraulically similar reaches along the longest watercourse. Time of concentration is determined from measured reach lengths and estimated average reach velocities. The basic equation for time of concentration is:

$$T_c = \left(\frac{L_1}{V_1} + \frac{L_2}{V_2} + \frac{L_3}{V_3} + \dots \frac{L_n}{V_n} \right) \frac{1}{60} \tag{3–17}$$

where

T_c = Time of concentration, minutes
V_1 = Average flow velocity in the uppermost reach of the watercourse, ft./sec.
L_1 = Length of the uppermost reach of the watercourse, ft.
V_2, V_3, ... = Average flow velocities in subsequent reaches progressing downstream, ft./sec.
L_2, L_3, ... = Lengths of subsequent reaches progressing downstream, ft.

Hydrologic Method	Watershed Condition	Time of Concentration Method
Rational Method	Un-gullied Watershed*	Upland Method
	Gullied Watershed*	Kirpich Formula
Simplified Peak Flow Method	Un-gullied Watershed*	Upland Method
	Gullied Watershed*	Kirpich Formula
	Watershed Partially Gullied	Upland Method for the Un-gullied Portion, then Kirpich Formula for the Gullied Portion**
USGS Regression Equations	—	NOT REQUIRED
Unit Hydrograph Method	No Defined Stream Channel	Upland Method
	Defined Stream Channel	Stream Hydraulic Method
Approved Urban Method	All Conditions	Use T_c Method Specified for the Approved Urban Method***

*A watershed is considered un-gullied if 10% or less of the primary watercourse exhibits gullying.

**Mixing T_c Methods in a watershed is only allowed with the Simplified Peak Flow Method.

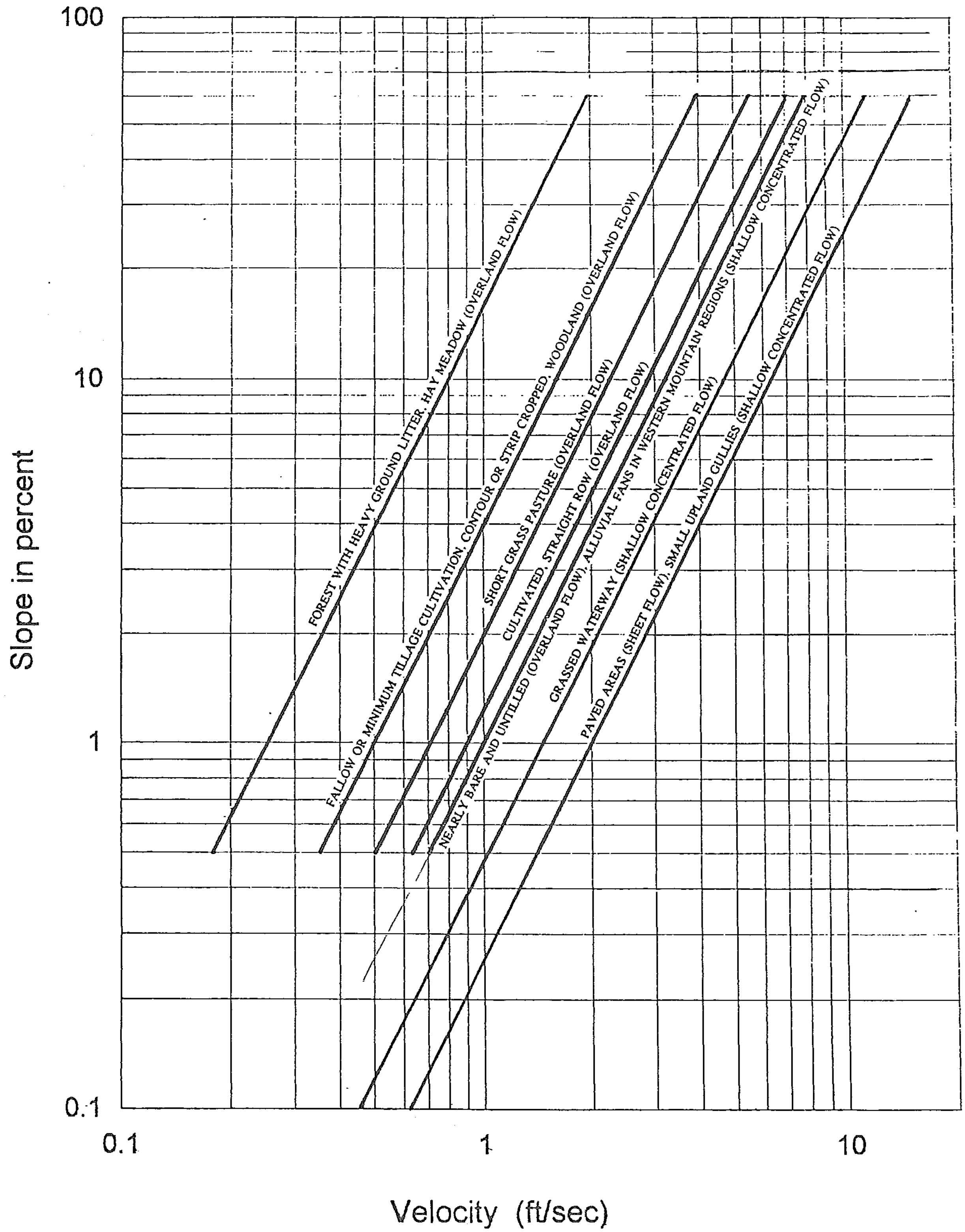
***When using AHYMO with the COMPUTE NM HYD routine, compute the time of concentration in accordance with the City of Albuquerque Design Process Manual. See *SECTIONS 3.2 AND 3.3.5* of this manual for limitations on the use of AHYMO.

Table 3-6
Time of Concentration Method Selection Chart

3.3.1.4.1 The Upland Method

The Upland Method is used to estimate travel times for overland flow and shallow concentrated flow conditions. Originally developed by the SCS, the upland method is limited to use in watersheds less than 2000 acres in size, or to the upper reaches of larger watersheds. For NMSHTD projects the Upland Method may be used for computing the time of concentration when using the Rational Method or the Simplified Peak Flow method on an un–gullied watershed.

At the very top of the watershed, sheet flow is the predominant flow regime. The overland flow lines in **Figure 3.10** may be used to estimate the velocity of sheet flow. Overland flow continues until the volume of water creates a shallow concentrated flow regime. In erosive soil formations with limited ground cover, the length of overland flow may be so short as to be negligible. Given the slope of the land and some knowledge of the ground cover conditions, **Figure 3.10** may be used to estimate the velocity of shallow concentrated flow. For NMSHTD 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 watercourse (by field inspection, or by a blue line on the USGS quadrangle topographic map), time of concentration should be computed by the Kirpich Method for the entire watershed. **When the Simplified Peak Flow method is being used for NMSHTD projects, the Upland Method may be used for the un–gullied portion of the watercourse, in combination with the Kirpich Formula for the gullied sections of the watercourse.**



Note: For watercourses with slopes less than 0.5 percent, use the overland flow velocity given for 0.5 percent, except for shallow concentrated flow where a flatter slope may be considered.

Figure 3-10
Flow Velocities for Overland and Shallow Concentrated Flows

Modified from SCS, NEH-4, 1972

3.3.1.4.2 Time of Concentration by the Kirpich Formula

This method is used to calculate time of concentration in gullied watersheds when using the Rational Method or the Simplified Peak Flow Method. The Kirpich Formula should be used 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. The Kirpich Formula is given as:

$$T_c = 0.0078 \ L^{0.77} \ S^{-0.385} \tag{3-18}$$

where

T_c = time of concentration, in minutes

L = length from drainage to outlet along the primary drainage path, in feet

S = average slope of the primary drainage path, in ft./ft.

The Kirpich Formula should generally be used for the entire drainage basin. The exception to this rule occurs when the Simplified Peak Flow Method is being used on NMSHTD projects and the watercourse has a mixture of gullied and un-gullied sections. In these situations, mixing of time of concentration methods is allowed. The Upland Method is used for the ungullied portion of the primary watercourse, and the Kirpich Formula is used for the gullied portion of the watercourse. The two times of concentration are added together to obtain the total time of concentration of the watershed. Typically the Kirpich Formula is only used for that portion of the watercourse shown in blue on the quadrangle topo map. **Mixing of time of concentration methods is only allowed with the Simplified Peak Flow Method for NMSHTD projects.**

3.3.1.4.3 The Stream Hydraulic Method

The stream hydraulic method is used when calculating peak flows by the Unit Hydrograph Method in a watercourse where a defined stream channel is evident (blue line, solid or broken, on a quadrangle topo map). The designer must measure or estimate the hydraulic properties of the stream channel, and must divide the total watercourse into channel reaches which are hydraulically similar. Field reconnaissance measurements of the stream channel are best, however sometimes direct measurements are not possible. The designer must determine the slope, channel cross section and an appropriate hydraulic roughness coefficient for each channel reach. Average slope is often determined from the topographic mapping of the watershed. Channel cross section should be measured in the field whenever possible. Roughness coefficients of the waterway should be based on actual observations of the watercourse or of nearby watercourses which are believed to be similar and which are more accessible.

Time of Concentration by the stream hydraulic method is simply the travel time in the stream channel. Channel flow velocities can be estimated from normal depth calculations for the watercourse. In addition to the average flow velocity, designers should compute the Froude Number of the flow. If the Froude number of the flow exceeds a value of 1.3, then the designer should verify that supercritical flow conditions can actually be sustained. For most earth lined channels the velocity calculation should be recomputed using a larger effective

3.3.3 *SIMPLIFIED PEAK FLOW METHOD*

The Simplified Peak Flow method estimates the peak rate of runoff and runoff volume from small to medium size watersheds. This method was developed by the Soil Conservation Service and revised by that agency for use in New Mexico ("Peak Rates of Discharge for Small Watersheds," Chapter 2, SCS, 1985). Infiltration and other losses are estimated using the SCS Curve Number (CN) methodology. Input parameters are consistent with those used in the SCS Unit Hydrograph method. The Simplified Peak Flow method is limited for NMSHTD use to single basins less than 5 square miles in area, and should not be used when T_c exceeds 8.0 hours. This method may be used on NMSHTD projects for those conditions identified in ***SECTION* 3.2** of this manual. This method should not be used for watersheds with perennial stream flow.

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 Time of Concentration of the watershed. 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 flows at elevations above 7500 ft. Drainage structures above this elevation should be evaluated by the unit hydrograph or USGS regression equation methods.

3.3.3.1 *APPLICATION*

<u>Step 1</u> – Gather Input Data

- Establish the appropriate Design Frequency Flood(s) for analysis
- Estimate the drainage area, A, in acres (***SECTION* 3.3.1.1**)
- Compute the Time of Concentration, T_c, in hours (***SECTION* 3.3.1.4**)
- Determine the appropriate runoff Curve Number, CN, for the drainage basin (*SECTION 3.3.1.3*)
- Obtain the 24–hour rainfall depth, P_{24}, for the appropriate design frequency, from *APPENDIX E*

Step 2 Determine the unit peak discharge, q_u, for the watershed. The unit peak discharge can be read from **Figure 3–18**, given the time of concentration, or calculated directly by the following equation:

(3–22)

$$q_u = 0.543\ T_c^{-0.812}\ 10^{-\frac{[\,|\,log(T_c) + 0.3\,| - log(T_c) - 0.3]^{1.5}}{10}}$$

where

q_u = unit peak discharge from the watershed, in cfs/ac–in
T_c = time of concentration, in hours

Note: for $T_c > 0.5$ hours, the last term of the equation, $10^{-\frac{[\,|\,log(T_c) + 0.3\,| - log(T_c) - 0.3]^{1.5}}{10}}$, is equal to 1.0

Step 3

Calculate the direct runoff from the watershed. The direct runoff is expressed as an average depth of water over the entire watershed, in inches. The direct runoff may be read from **Figure 3–17** using the 24–hour rainfall depth P_{24} in inches, and the runoff curve number, CN. The runoff depth may also be calculated from the following equation:

(3–23)

$$Q_d = \frac{[P_{24} - (200/CN) + 2]^2}{P_{24} + (800/CN) - 8}$$

where

Q_d = average runoff depth for the entire watershed, in inches

Step 4

Compute the peak discharge from the watershed by the following equation:

(3–24)

$$Q_p = A \bullet Q_d \bullet q_u$$

where

Q_p = peak discharge, in cfs
A = drainage area, in acres

Step 5

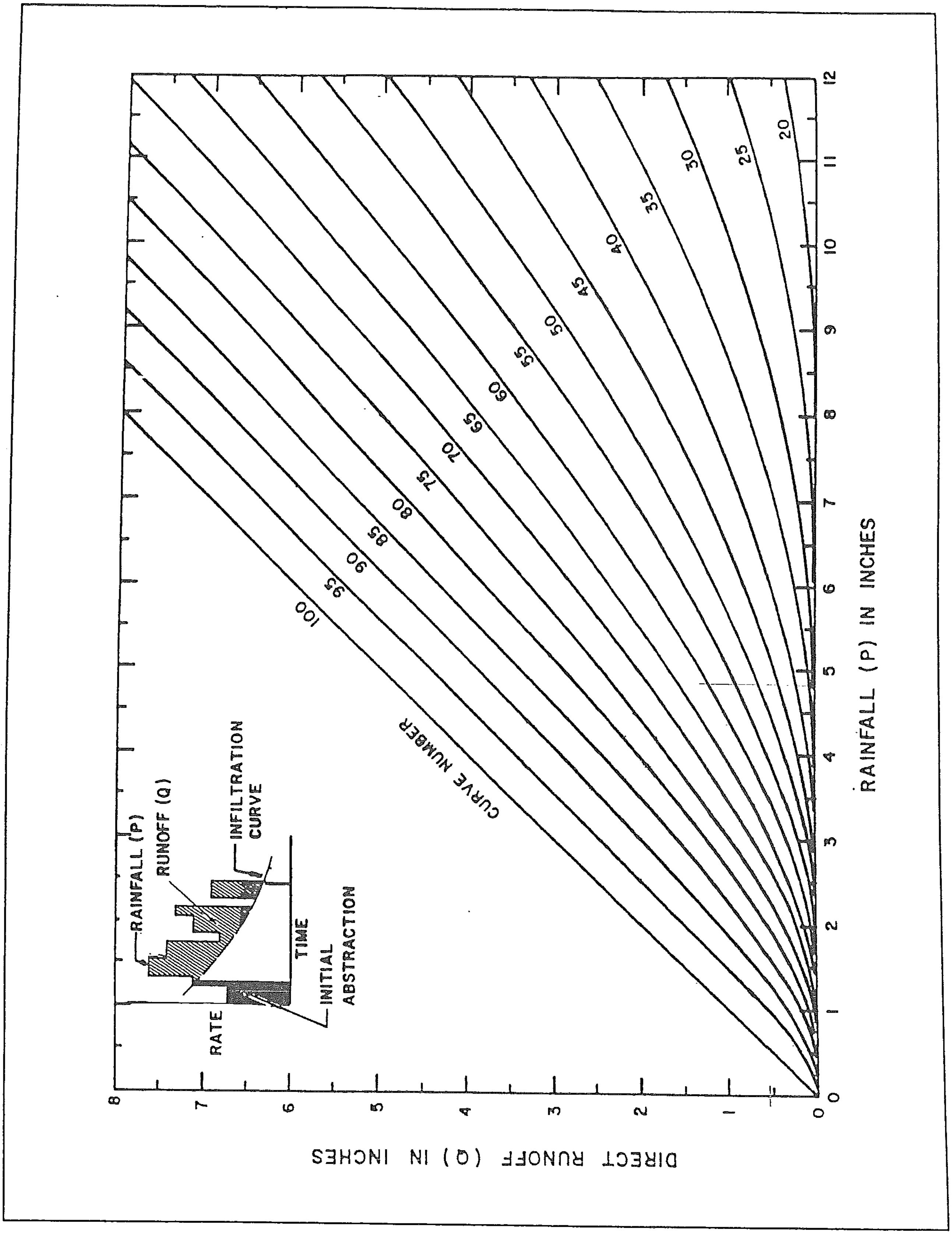
Compute the runoff volume, if required. The runoff volume is obtained by the equation:

(3–25)

$$Q_v = \frac{Q_d \bullet A}{12}$$

where

Q_v = runoff volume from the watershed, in ac–ft



Adapted from SCS, NEH-4, 1964

Figure 3-17
Estimating Direct Runoff

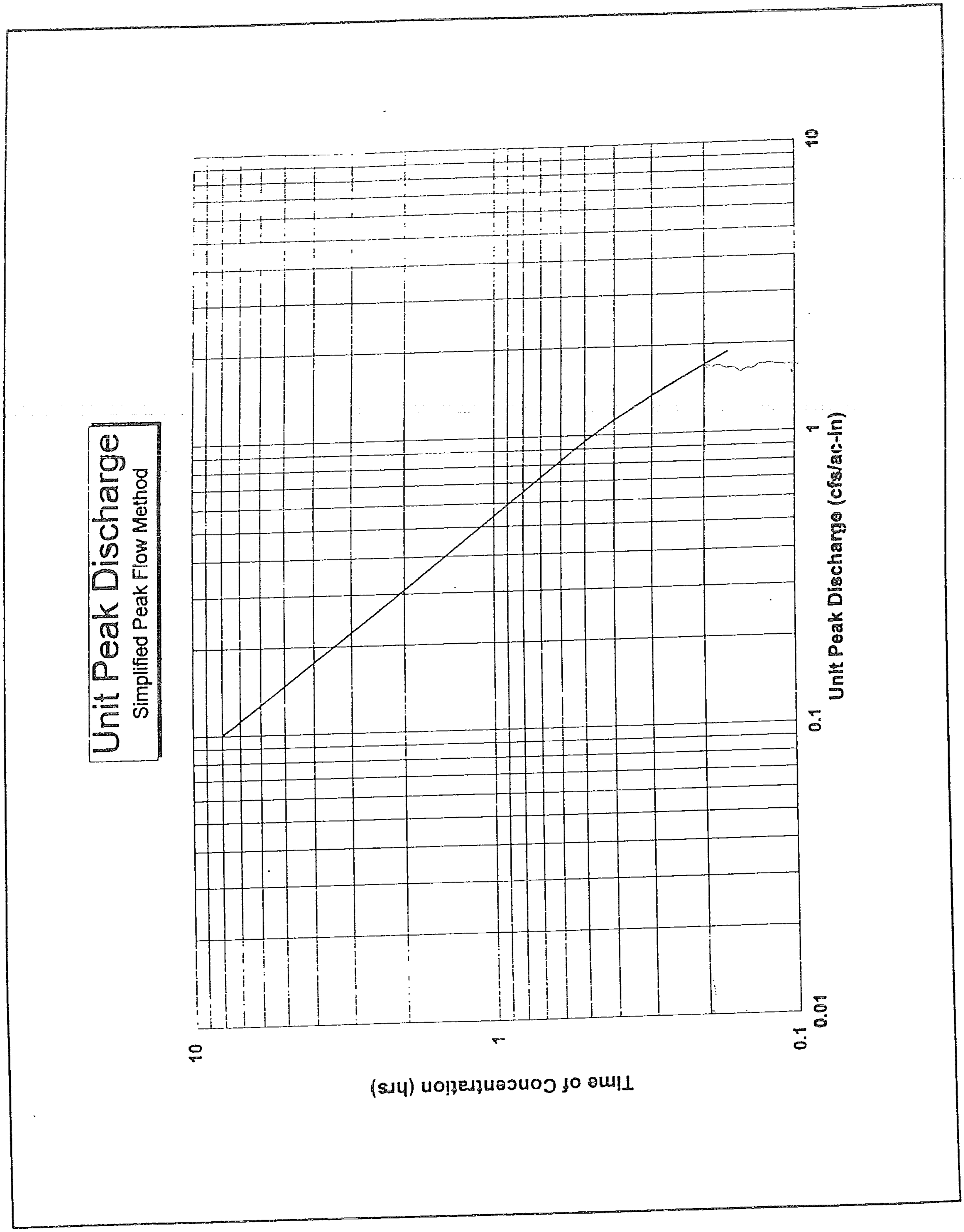


Figure 3-18
Unit Peak Discharge
for the Simplified Peak Flow Method

Step 6

Estimate Transmission Losses, if required. For watersheds less than 1.0 square miles in size there is no reduction factor applied. Where base flow is observed or known to occur, transmission losses should not be included. For large watersheds with sand or gravel bed channels, transmission losses may need to be considered. To compute transmission losses, follow the procedure in the SCS document NEH–4, Chapter 19, Transmission Losses, 1983.

Simplified Peak Flow Worksheet

Structure Location: ______________________________

Structure Description: ______________________________

Drainage Area: A = ______________ acres

Time of Concentration: T_c = ______________ hours

Weighted Runoff Curve Number: CN = ______________ —

Unit Peak Discharge (from **Figure 3–18**): q_u = ______________ cfs/ac–in

Design Frequency Flood	________–year	________–year
24–hour Rainfall Depth (*APPENDIX E*):	P_{24} = ________ in.	P_{24} = ________ in.
Direct Runoff (**Figure 3–17**):	**Qd** = ________ in.	**Qd** = ________ in.
Peak Discharge, **Qp** = **A** • **Qd** • **qu**:	**Qp** = ________ cfs	**Qp** = ________ cfs
Runoff Volume, **Qv** = **A** • **Qd/12**:	**Qv** = ________ ac–ft	**Qv** = ________ ac–ft

Transmission Losses, if applicable (computed by methods in SCS NEH 4, Chapter 19, 1983)

Predicted Runoff Volume:	Q_{pv} = ________ ac–ft	Q_{pv} = ________ ac–ft
Predicted peak Discharge:	Q_{pp} = ________ ac–ft	Q_{pp} = ________ ac–ft

Project Location: ______________________________

CN#: ______________________________

Date: ______________________________

Computed by: ______________ Checked by: ______________

Figure 3–19
Simplified
Peak Flow
Worksheet

3.3.3.2 *SIMPLIFIED PEAK FLOW METHOD EXAMPLE PROBLEMS*

Problem No. 3

Location: South of Deming, sparse desert brush
Elevation: 4,000 ft.
Design Frequency Flood: 50–year
Watershed Area: 250 acres (< 5 sq. mi., so okay for Simplified Peak Flow Method)
24–hour rainfall depth, 50–year return frequency, from **Figure E–11**, P_{50} = 3.3 inches

Compute the time of concentration.

The upper watershed shows significant erosion, with many gullies evident.
Assume overland flow occurs for the first 200 ft. at S = 0.035 ft./ft.
Assume shallow concentrated flow occurs for the remaining 600 ft. at S = 0.025 ft./ft. until a defined stream channel is evident on the quadrangle topographic map.
Select appropriate velocities from **Figure 3–10**.

$$T_{c\ upland} = \left(\frac{200\ ft.}{1.8\ ft./\text{sec.}} + \frac{600\ ft.}{3.1\ ft./\text{sec.}} \right) \frac{1}{60} = 5.1\ min.$$

The defined stream channel is a broad wash where larger flows really spread out. Channel length is measured as 3,000 ft. Bottom width ≈ 30 ft., S = 0.015 ft./ft., n = 0.030. *For this channel we can use the simplifying assumption that R = 1.*

Compute channel velocity based on Manning's equation.

$$V = \frac{1.49}{0.030} (1)^{\frac{2}{3}} (.015)^{\frac{1}{2}} = 6.08\ ft./\text{sec.}$$

$$T_{c\ stream\ hydraulic} = \left(\frac{3000\ ft.}{6.1\ ft./\text{sec.}} \right) \frac{1}{60} = 8.2\ min.$$

The total time of concentration for the watershed is

$$T_{c\ watershed} = 5.1 + 8.2 = 13.3\ minutes = 0.222\ hours$$

(0.222 hours is less than 8.0 hours, okay to use the simplified peak flow method.)

Select a representative runoff Curve Number.
- Vegetation: Desert brush
- HSG: A
- Hydrologic Condition: poor, minimal ground cover

From **Table 3–1**, select CN = 63

Compute the direct runoff using **Equation 3–23** (or obtain Q_d from **Figure 3–17**).

$$Q_d = \frac{\left(3.3 - \left(\frac{200}{63}\right) + 2\right)^2}{3.3 + \left(\frac{800}{63}\right) - 8} = 0.56 \ inches$$

Because the watershed is less than 1.0 square miles, Transmission Losses are not considered.

The unit peak discharge, q_u, is read from **Figure 3–18**, or calculated directly by **Equation 3–22**.

$q_u = 1.607 \ cfs/ac{-}in$

Compute the design frequency peak flow by **Equation 3–24**.

$Q_p = (250)\ (0.56)\ (1.607)$
$\underline{Q_p = 225 \ cfs}$

Problem No. 4

Location: North of Crownpoint, gently sloping rangeland
Elevation: 6,500 ft.
Design Frequency Flood: 50–year

Watershed Area: 600 acres (< 5 sq. mi., okay)
24–hour rainfall depth, 50–year return frequency, from **Figure E–11**, P_{50} = 2.2 inches

Compute the time of concentration.

The total length of the watercourse to the hydraulically most remote point in the drainage basin is 7,600 ft.

We are unable to inspect the entire watershed, therefore some assumptions are necessary:

Assume overland flow occurs for 400 ft. at S = 0.020 ft./ft.
Shallow concentrated flow is assumed for the remaining 1,200 ft. until a defined stream channel is observed on the quad sheet topo. S = 0.010 ft./ft.
Select appropriate velocities from **Figure 3–10**.

$$T_{c\ upland} = \left(\frac{200\ ft.}{1.4\ ft./\text{sec.}} + \frac{200\ ft.}{1.4\ ft./\text{sec.}} + \frac{1{,}200\ ft.}{2.0\ ft./\text{sec.}}\right)\frac{1}{60} = 14.8\ min.$$

The remainder of watercourse is a defined stream channel in alluvial material.
Length = 6,000 ft., Slope = 0.010 ft./ft.
The stream channel observed upstream of the highway has the following cross sectional properties:

15 ft. bottom, 1:1 sideslopes, cut banks approximately 4 ft. tall

We estimate Manning's n = 0.030, sand bed channel without vegetation.

Use the simplified procedure for moderate and narrow width channels to estimate flow velocity.

Estimate the flow depth from vegetation and old debris, d ≈ 3.0 ft.

Flow Area = 45 ft.^{2}

Wetted Perimeter = 21 ft.

The Hydraulic Radius, $R = \frac{A}{P} = \frac{45}{21} = 2.1$

Flow velocity computed by Manning's Equation is

$$V = \frac{1.49}{n} R^{\frac{2}{3}} S^{\frac{1}{2}} = \frac{1.49}{0.030} (2.1)^{\frac{2}{3}} (0.010)^{\frac{1}{2}} = 8.3 \ ft./sec.$$

$$T_{c \ stream \ hydraulic} = \left(\frac{6{,}000 \ ft.}{8.3 \ ft./sec.}\right) \frac{1}{60} = 12.0 \ min.$$

The total time of concentration for the watershed is

$$T_{c \ watershed} = 14.8 + 12.0 = 26.8 \ minutes = 0.447 \ hours$$

Select a representative runoff Curve Number.

Vegetation: Desert brush
HSG: B
Hydrologic Condition: 20% ground cover

From **Figure 3–8**, select CN = 82.5

Compute the direct runoff using **Equation 3–23** (or obtain Q_d from **Figure 3–17**).

$$Q_d = \frac{\left(2.2 - \left(\frac{200}{82.5}\right) + 2\right)^2}{2.2 + \left(\frac{800}{82.5}\right) - 8} = 0.81 \ inches$$

The unit peak discharge, q_u, is read from **Figure 3–18**, or calculated directly by **Equation 3–22**.

$q_u = 1.037 \ cfs/ac{-}in$

Compute the design frequency peak flow by **Equation 3–24**.

$Q_p = (600)\ (0.81)\ (1.037)$
$\underline{Q_p = 504 \ cfs}$

As a check, compute the normal depth for this discharge.

for Q_p = 504 cfs, normal depth d = 3.14 ft.
This confirms our assumed depth. If the normal depth was substantially different from the assumed value then we would need to revise our T_c calculation accordingly.

Problem No. 5

Location: Near Chama, forested mountain terrain
Elevation: 7,500 ft.
Design Frequency Flood: 50–year

Watershed Area: 3,000 acres (< 5.0 square miles, okay for Simplified Peak Flow Method)
P_{24}, 50–year = 2.8 inches

Compute the time of concentration.

Unable to inspect the entire watershed, therefore some assumptions are necessary:

Assume overland flow occurs for 400 ft. at S = 0.100 ft./ft.
Shallow concentrated flow is assumed for the remaining 2,200 ft. until a defined stream channel is observed on the quad sheet topo. S = 0.060 ft./ft.
Select appropriate velocities from **Figure 3–10**.

$$T_{c\ upland} = \left(\frac{100\ ft.}{0.8\ ft./\text{sec.}} + \frac{300\ ft.}{2.8\ ft./\text{sec.}} + \frac{2{,}200\ ft.}{5.0\ ft./\text{sec.}} \right) \frac{1}{60} = 11.2\ min.$$

The remainder of watercourse is a defined stream channel.

Since there is not any real data on the stream channel geometry and no good evidence of flow depths, use the iterative procedure.

Estimate the peak discharge using the USGS statewide small basin regression equations.

From **Table 3–7** we find the 50–year return frequency equation to be

$$Q_{small\ basin} = 7.92 \text{ x } 10^2 \bullet A^{0.45}$$

$$Q_{smallbasin} = 792 \left(\frac{3{,}000}{640} \right)^{0.45} = 1{,}587\ cfs$$

For the SCS iterative procedure, the flow rate used to compute channel flow velocity is assumed to be 2/3 of the estimated peak flow.

$$Q_{velocity} = 2/3\ (1587) = 1{,}063\ cfs$$

The length of stream channel has been measured as 14,500 ft. from the quad sheet topo.

Assume a channel geometry: 10 ft. bottom, 2:1 sideslopes, n = 0.045, slope = 0.035 ft./ft.

By normal depth calculation, Velocity, V = 12.4 ft./sec.

The travel time is then

$$T_{c\ stream\ hydraulic} = \left(\frac{14{,}500\ ft.}{12.4\ ft./sec.}\right)\frac{1}{60} = 19.5\ min.$$

Total Time of Concentration for the watershed is

$$T_{c\ watershed} = 11.2 = 19.5 = 30.7\ min. = 0.512\ hours$$

Select a representative Runoff Curve Number.

Vegetation: Woods
HSG: C
Hydrologic Condition: Fair

From **Table 3–3**, choose CN = 73

Compute the direct runoff (**Equation 3–23**), or use **Figure 3–17**.

$$Q_d = \frac{\left(2.8 - \left(\frac{200}{73}\right) + 2\right)^2}{2.8 + \left(\frac{800}{73}\right) - 8} = 0.74\ inches$$

Channel seepage was observed, so transmission losses are neglected.

The unit peak discharge is given by **Equation 3–22**, or may be obtained directly from **Figure 3–18**.

Since T_c = 0.512 hours > 0.5 hours, **Equation 3–22** reduces to

$q_u = (0.543)\ T_c^{-.812}$
$q_u = (0.543)\ (0.512)^{-.812}$
$q_u = 0.94\ cfs/ac\text{–}in$

The design frequency peak flow is given by **Equation 3–24**.

$Q_p = (3000)\ (0.74)\ (0.94)$
$\underline{Q_p = 2087\ cfs}$

Since the calculated Q_p is more than 20% different than the estimated Q_p, the time of concentration for the stream hydraulic reach should be revised.

$$Q_{velocity} = 2/3\ (2087) = 1398\ cfs$$

For the same channel geometry, V = 13.3 ft./sec.

$$T_{c\ stream\ hydraulic} = \left(\frac{14{,}500\ ft.}{13.3\ ft./\text{sec.}}\right)\frac{1}{60} = 18.2\ min.$$

$$T_{c\ watershed} = 11.2 = 18.2 = 29.4\ min. = 0.490\ hours$$

q_u = 0.97 cfs/ac–in

Q_p = (3000) (0.74) (0.97)
$\underline{Q_p = 2153\ cfs}$

This peak flow is within 10% of the Q_p used to estimate channel flow velocity, so no further iterations are required.

The peak flow calculated using the simplified peak flow method is somewhat larger than the estimated peak flow using the USGS small basin regression equation. Remember that the USGS equation is valid for the entire state, regardless of vegetation or elevation. Since we have used a runoff Curve Number to model the runoff response of this watershed, the Q_p calculated by the simplified peak flow method is probably better. Also, the observed channel seepage suggests using the higher peak flow value. Use Q_p = 2153 cfs for design.

3.3.4 USGS Regression Equations for New Mexico

Stream gage data and associated rainfall data from sites around New Mexico have been compiled by the United States Geological Survey (USGS) (Waltemeyer, 1986; Thomas and Gold, 1982). These watersheds were evaluated to find basin and climatic characteristics which are statistically significant in predicting peak flow rates at the stream gages. Regression equations were developed which predict the peak rate of runoff from watersheds within certain physiographic regions of New Mexico for different return period events.

The most recent set of USGS regression equations for New Mexico (Waltemeyer, 1996) were developed using 201 gaging stations, the majority of which are in New Mexico. Flood discharges for selected exceedance probabilities were determined for each streamflow gaging station. Logarithms of annual peak flows were fitted to a log Pearson Type III probability distribution to develop flood frequency curves according to standard techniques (Interagency Advisory Committee on Water Data, 1982). New Mexico was divided into eight physiographic regions, yielding regression equations with the best data fit. **Figure 3–20** shows the eight regions within New Mexico. The NMSHTD has selected these equations for predicting peak rates of runoff for larger NMSHTD drainage basins (see *Section 3.2*). These USGS regression equations may be used in rural areas, or in urban areas as described in *Section 3.3.4.2*. The USGS regression equations are also the preferred hydrologic analysis method when sizing drainage structures for perennial streams.

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, JUNE 2006, NOAA ATLAS 14, VOLUME 1, VERSION 5 , POINT PRECIPITATION FREQUENCY ESTIMATES FOR LATITUDE: 32.2031°, LONGITUDE: -103.5431°, PDS-BASED POINT PRECIPITATION FREQUENCY ESTIMATES WITH 90% CONFIDENCE INTERVALS (IN INCHES)

NOAA Atlas 14, Volume 1, Version 5
Location name: Jal, New Mexico, US*
Latitude: 32.2031°, Longitude: -103.5431°
Elevation: 3581 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	0.314 (0.280-0.353)	0.406 (0.361-0.457)	0.540 (0.480-0.607)	0.644 (0.570-0.723)	0.785 (0.691-0.880)	0.895 (0.783-1.00)	1.01 (0.880-1.13)	1.13 (0.977-1.26)	1.29 (1.11-1.45)	1.42 (1.21-1.59)
10-min	0.478 (0.425-0.537)	0.618 (0.549-0.695)	0.822 (0.730-0.924)	0.981 (0.868-1.10)	1.20 (1.05-1.34)	1.36 (1.19-1.52)	1.54 (1.34-1.72)	1.72 (1.49-1.92)	1.97 (1.68-2.20)	2.16 (1.84-2.42)
15-min	0.592 (0.527-0.666)	0.765 (0.681-0.862)	1.02 (0.905-1.15)	1.22 (1.08-1.36)	1.48 (1.30-1.66)	1.69 (1.48-1.89)	1.91 (1.66-2.13)	2.13 (1.84-2.38)	2.44 (2.09-2.73)	2.68 (2.28-3.00)
30-min	0.797 (0.709-0.897)	1.03 (0.917-1.16)	1.37 (1.22-1.54)	1.64 (1.45-1.84)	2.00 (1.76-2.24)	2.28 (1.99-2.54)	2.57 (2.23-2.87)	2.87 (2.48-3.21)	3.28 (2.81-3.67)	3.61 (3.06-4.04)
60-min	0.987 (0.878-1.11)	1.28 (1.13-1.44)	1.70 (1.51-1.91)	2.03 (1.79-2.27)	2.47 (2.17-2.77)	2.82 (2.46-3.15)	3.18 (2.77-3.55)	3.55 (3.07-3.97)	4.06 (3.48-4.54)	4.46 (3.79-5.00)
2-hr	1.16 (1.03-1.31)	1.50 (1.33-1.70)	2.03 (1.80-2.29)	2.44 (2.16-2.75)	3.02 (2.65-3.39)	3.48 (3.04-3.90)	3.97 (3.44-4.44)	4.49 (3.86-5.02)	5.21 (4.43-5.83)	5.79 (4.88-6.49)
3-hr	1.23 (1.09-1.39)	1.59 (1.42-1.79)	2.13 (1.89-2.40)	2.57 (2.27-2.88)	3.18 (2.79-3.56)	3.67 (3.20-4.10)	4.19 (3.63-4.69)	4.74 (4.08-5.30)	5.52 (4.69-6.18)	6.15 (5.17-6.91)
6-hr	1.41 (1.25-1.58)	1.81 (1.61-2.04)	2.40 (2.14-2.71)	2.89 (2.56-3.24)	3.57 (3.15-4.00)	4.13 (3.61-4.62)	4.72 (4.10-5.28)	5.35 (4.61-5.98)	6.25 (5.32-6.99)	6.98 (5.88-7.82)
12-hr	1.57 (1.40-1.77)	2.02 (1.80-2.28)	2.67 (2.37-3.01)	3.21 (2.84-3.61)	3.96 (3.48-4.44)	4.57 (3.98-5.12)	5.22 (4.52-5.84)	5.91 (5.07-6.61)	6.89 (5.84-7.71)	7.69 (6.45-8.63)
24-hr	1.65 (1.48-1.86)	2.13 (1.90-2.40)	2.86 (2.55-3.21)	3.45 (3.06-3.87)	4.31 (3.80-4.83)	5.00 (4.38-5.60)	5.75 (5.00-6.44)	6.56 (5.64-7.36)	7.72 (6.54-8.69)	8.67 (7.25-9.80)
2-day	1.76 (1.57-2.00)	2.27 (2.02-2.58)	3.06 (2.71-3.46)	3.71 (3.27-4.19)	4.65 (4.07-5.24)	5.42 (4.71-6.12)	6.27 (5.38-7.07)	7.18 (6.09-8.13)	8.50 (7.08-9.67)	9.60 (7.88-11.0)
3-day	1.84 (1.63-2.09)	2.37 (2.10-2.69)	3.21 (2.83-3.64)	3.90 (3.42-4.42)	4.91 (4.28-5.56)	5.76 (4.96-6.52)	6.68 (5.70-7.58)	7.69 (6.48-8.75)	9.16 (7.58-10.5)	10.4 (8.47-12.0)
4-day	1.91 (1.68-2.17)	2.46 (2.17-2.81)	3.35 (2.94-3.81)	4.09 (3.58-4.65)	5.17 (4.49-5.87)	6.09 (5.22-6.92)	7.09 (6.03-8.08)	8.20 (6.88-9.37)	9.82 (8.08-11.3)	11.2 (9.06-13.0)
7-day	2.16 (1.90-2.46)	2.79 (2.46-3.18)	3.81 (3.35-4.34)	4.66 (4.08-5.30)	5.90 (5.12-6.71)	6.95 (5.97-7.91)	8.10 (6.89-9.25)	9.37 (7.88-10.7)	11.2 (9.26-13.0)	12.8 (10.4-14.9)
10-day	2.36 (2.10-2.69)	3.05 (2.71-3.47)	4.17 (3.69-4.74)	5.10 (4.50-5.79)	6.47 (5.65-7.33)	7.63 (6.59-8.63)	8.91 (7.61-10.1)	10.3 (8.68-11.7)	12.4 (10.2-14.2)	14.2 (11.5-16.3)
20-day	2.91 (2.60-3.25)	3.74 (3.35-4.19)	4.98 (4.45-5.58)	5.98 (5.33-6.69)	7.40 (6.55-8.26)	8.55 (7.51-9.55)	9.77 (8.52-10.9)	11.1 (9.54-12.4)	12.9 (11.0-14.6)	14.5 (12.1-16.5)
30-day	3.39 (3.05-3.79)	4.34 (3.90-4.86)	5.73 (5.13-6.40)	6.83 (6.10-7.63)	8.37 (7.43-9.36)	9.61 (8.47-10.7)	10.9 (9.54-12.2)	12.3 (10.6-13.8)	14.2 (12.1-16.1)	15.8 (13.3-17.9)
45-day	3.98 (3.56-4.45)	5.11 (4.57-5.72)	6.76 (6.05-7.55)	8.05 (7.18-8.98)	9.83 (8.72-11.0)	11.2 (9.90-12.6)	12.7 (11.1-14.2)	14.2 (12.4-16.0)	16.4 (14.0-18.5)	18.0 (15.2-20.5)
60-day	4.55 (4.08-5.06)	5.83 (5.24-6.50)	7.63 (6.85-8.49)	9.01 (8.07-10.0)	10.9 (9.69-12.1)	12.3 (10.9-13.7)	13.8 (12.1-15.3)	15.3 (13.3-17.0)	17.3 (14.9-19.4)	18.8 (16.1-21.3)

[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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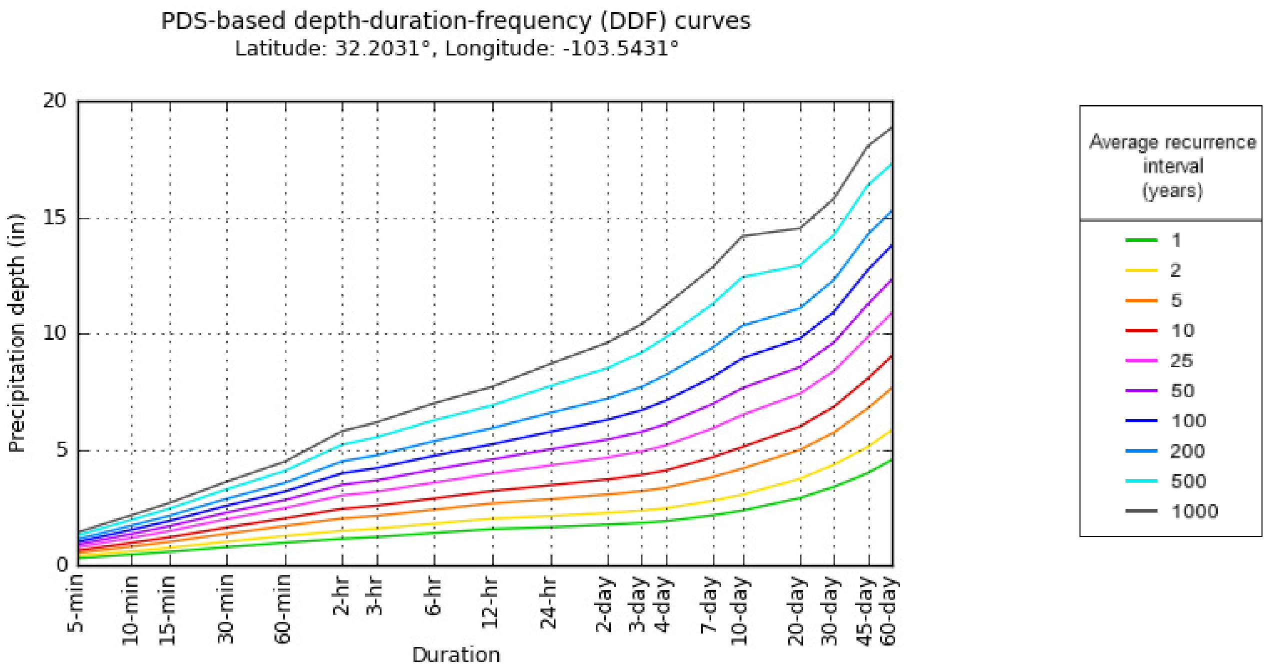
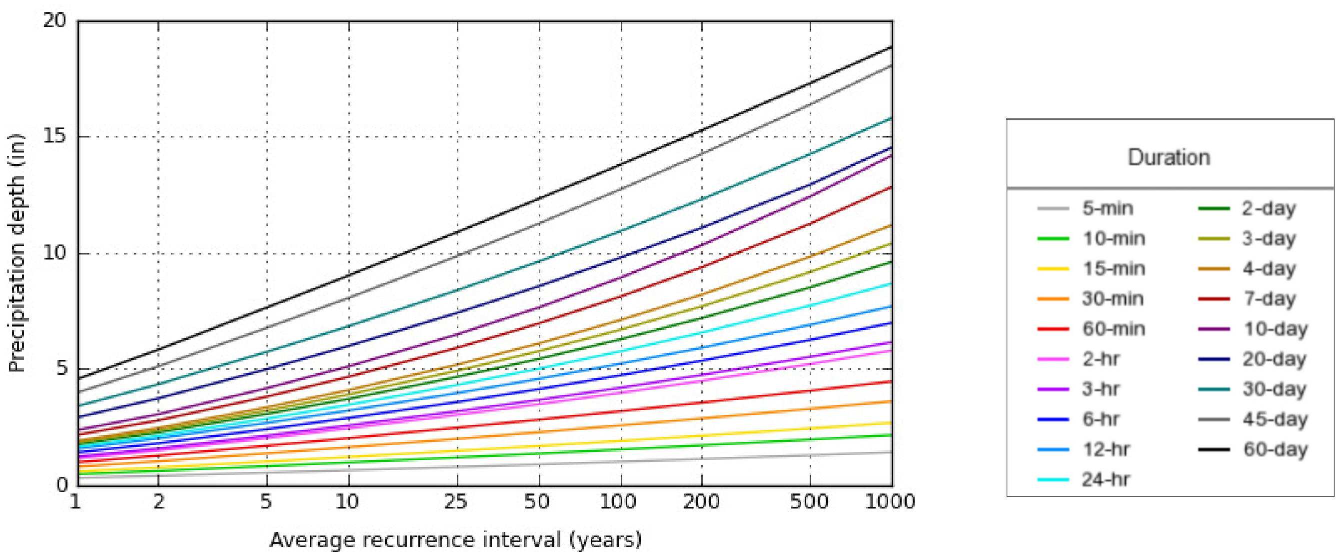
PF graphical





NOAA Atlas 14, Volume 1, Version 5 Created (GMT): Mon Feb 1 20:55:33 2016

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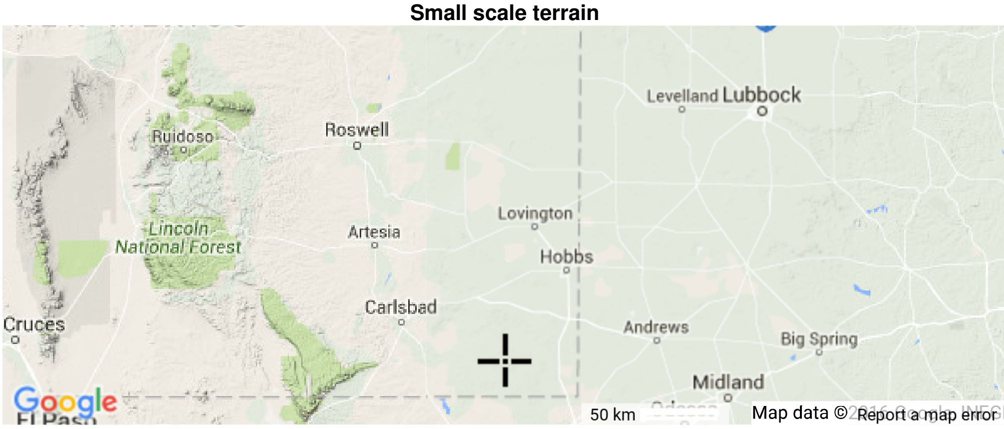
Maps & aerials

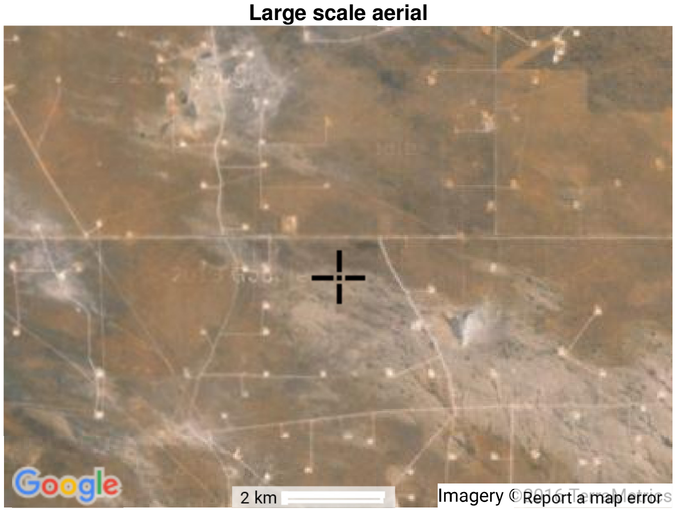
Small scale terrain



Large scale terrain

Large scale map

Large scale aerial



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US Department of Commerce
National Oceanic and Atmospheric Administration
National Weather Service
National Water Center
1325 East West Highway
Silver Spring, MD 20910
Questions?: HDSC.Questions@noaa.gov

APPLICATION FOR PERMIT
OWL LANDFILL SERVICES, LLC

VOLUME III: LANDFILL ENGINEERING CALCULATIONS
SECTION 8: DRAINAGE CALCULATIONS

ATTACHMENT III.3.C

AUTODESK,® INC, 2016, *STORM AND SANITARY ANALYSIS*, MODEL OUTPUT

Project Description

File Name .... OWL Post Construction Analysis_TR-20.SPF
Description ....
P:\acad 2003\560.01.02\PERMIT PLANS\WholeSite-3D-Sheets2-7,9,12,13-novol.dwg

Project Options

Flow Units .... CFS
Elevation Type .... Elevation
Hydrology Method .... SCS TR-20
Time of Concentration (TOC) Method .... SCS TR-55
Link Routing Method .... Hydrodynamic
Enable Overflow Ponding at Nodes .... YES
Skip Steady State Analysis Time Periods .... NO

Analysis Options

Start Analysis On	Jan 26, 2016	00:00:00
End Analysis On	Jan 27, 2016	12:00:00
Start Reporting On	Jan 26, 2016	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...	9
Nodes..	11
Junctions	5
Outfalls	2
Flow Diversions	1
Inlets	0
Storage Nodes	3
Links..	8
Channels	7
Pipes	0
Pumps	0
Orifices	0
Weirs	1
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	Rain Gage-01	Time Series	OWL site	Intensity	inches	New Mexico	Lea	25	4.31	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	{Site.2}.RunOff_E	62.44	72.00	4.31	1.68	8.75	243.96	0 00:11:39
2	{Site.2}.RunOff_N	60.21	72.00	4.31	1.68	8.43	134.04	0 00:29:28
3	{Site.2}.RunOff_S	58.04	72.00	4.31	1.68	8.13	247.24	0 00:09:21
4	{Site.2}.RunOff_W	67.83	72.00	4.31	1.68	9.50	279.91	0 00:10:13
5	{Site.2}.RunOn_1	45.54	72.00	4.31	1.68	6.38	86.81	0 00:35:56
6	{Site.2}.RunOn_2	77.13	72.00	4.31	1.68	10.81	193.05	0 00:25:03
7	{Site.2}.RunOn_3	40.44	72.00	4.31	1.68	5.66	63.40	0 00:45:22
8	{Site.2}.RunOn_4-EVAP-PONDS	34.70	72.00	4.31	1.68	4.86	150.92	0 00:08:51
9	{Site.2}.RunOn_5	18.33	72.00	4.31	1.68	2.57	53.24	0 00:20:09

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	RunOffE-In	Junction	3593.50	3597.00	3593.50	3597.00	0.00	234.92	3597.00	0.00	0.00	0 07:10	22.11	15.00
2	RunOn1RunOffN-In	Junction	3578.00	3580.00	3578.00	3580.00	0.00	217.16	3580.82	0.00	0.18	0 00:00	0.00	0.00
3	RunOn2In	Junction	3577.00	3580.00	3577.00	3580.00	0.00	191.48	3580.00	0.00	0.00	0 07:15	20.01	19.00
4	RunOn3-In	Junction	3570.00	3572.00	3570.00	3572.00	0.00	174.47	3572.31	0.00	0.69	0 00:00	0.00	0.00
5	SE-ConvergePt	Junction	3561.00	3564.50	3561.00	3564.50	0.00	238.09	3564.16	0.00	0.34	0 00:00	0.00	0.00
6	RetentionSpillway	Outfall	3546.00					13.50	3546.00					
7	WesternOutfall	Outfall	3550.00					334.02	3552.80					
8	S-RunOff-FlowSplit	Flow Diversions	3563.00	3565.50	3563.00		0.00	220.16	3565.97				0.00	0.00
9	EvapPonds	Storage Node	3579.00	3581.00	3579.00		125000.00	137.27	3580.69				0.00	0.00
10	SE-DrainPond	Storage Node	3538.00	3557.00	3538.00		125000.00	216.04	3552.33				0.00	0.00
11	SW-DrainPond	Storage Node	3545.00	3550.00	3545.00		107652.00	202.77	3548.32				0.00	0.00

Link Summary

SN	Element ID	Element Type	From (Inlet) Node	To (Outlet) Node	Length	Inlet Invert Elevation	Outlet Invert Elevation	Average Slope	Diameter or Height	Manning's Roughness	Peak Flow	Design Flow Capacity	Peak Flow/ Design Flow Ratio	Peak Flow Velocity	Peak Flow Depth	Peak Flow Depth/ Total Depth Ratio	Total Time Surcharged	Reported Condition
					(ft)	(ft)	(ft)	(%)	(in)		(cfs)	(cfs)		(ft/sec)	(ft)		(min)	
1	EastBoundChannel	Channel	RunOn2In	RunOn3-In	2227.00	3577.00	3570.00	0.3100	36.000	0.0320	111.67	141.45	0.79	3.75	2.62	0.88	0.00	
2	EastChan2SE_Pond	Channel	SE-ConvergePt	SE-DrainPond	865.00	3561.00	3557.00	0.4600	42.000	0.0320	215.88	329.23	0.66	5.38	2.63	0.75	0.00	
3	EastLFBermTop	Channel	RunOffE-In	SE-ConvergePt	3800.00	3594.00	3561.00	0.8700	36.000	0.0320	105.52	114.49	0.92	8.45	2.85	0.95	0.00	
4	SEdiagonalChan	Channel	RunOn3-In	SE-ConvergePt	913.00	3570.00	3561.00	0.9900	30.000	0.0320	170.02	203.04	0.84	5.74	2.40	0.96	0.00	
5	S-RunOff2SE_pond	Channel	S-RunOff-FlowSplit	SE-DrainPond	1175.00	3564.00	3557.00	0.6000	24.000	0.0320	25.54	35.98	0.71	3.55	1.59	0.83	0.00	
6	S-RunOff2SW_pond	Channel	S-RunOff-FlowSplit	SW-DrainPond	1479.07	3563.00	3550.00	0.8800	30.000	0.0320	152.12	163.82	0.93	6.18	2.31	0.93	0.00	
7	WestBoundChan	Channel	RunOn1RunOffN-In	WesternOutfall	3127.89	3578.00	3550.00	0.9000	36.000	0.0320	207.70	238.72	0.87	6.35	2.80	0.94	0.00	
8	Weir-01	Weir	SW-DrainPond	RetentionSpillway		3545.00	3546.00				13.50							

Subbasin Hydrology

Subbasin : {Site.2}.RunOff_E

Input Data

Area (ac) 62.44
Weighted Curve Number 72.00
Rain Gage ID .... Rain Gage-01

Composite Curve Number

Soil/Surface Description	Area (acres)	Soil Group	Curve Number
-	62.44	-	72.00
Composite Area & Weighted CN	62.44		72.00

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 :	.08	0.00	0.00
Flow Length (ft) :	100	0.00	0.00
Slope (%) :	5	0.00	0.00
2 yr, 24 hr Rainfall (in) :	2.50	0.00	0.00
Velocity (ft/sec) :	0.36	0.00	0.00
Computed Flow Time (min) :	4.65	0.00	0.00

Shallow Concentrated Flow Computations	Subarea A	Subarea B	Subarea C
Flow Length (ft) :	1449.64	0.00	0.00
Slope (%) :	16	0.00	0.00
Surface Type :	Unpaved	Unpaved	Unpaved
Velocity (ft/sec) :	6.45	0.00	0.00
Computed Flow Time (min) :	3.75	0.00	0.00

Channel Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.03	0.00	0.00
Flow Length (ft) :	1339.38	0.00	0.00
Channel Slope (%) :	1	0.00	0.00
Cross Section Area (ft^2) :	19.1	0.00	0.00
Wetted Perimeter (ft) :	11.8	0.00	0.00
Velocity (ft/sec) :	6.85	0.00	0.00
Computed Flow Time (min) :	3.26	0.00	0.00

Total TOC (min) 11.65

Subbasin Runoff Results

Total Rainfall (in) 4.31
Total Runoff (in) 1.68
Peak Runoff (cfs) 243.96
Weighted Curve Number 72.00
Time of Concentration (days hh:mm:ss) 0 00:11:39

Subbasin : {Site.2}.RunOff_N

Input Data

Area (ac) 60.21
Weighted Curve Number 72.00
Rain Gage ID .... Rain Gage-01

Composite Curve Number

Soil/Surface Description	Area (acres)	Soil Group	Curve Number
-	60.21	-	72.00
Composite Area & Weighted CN	60.21		72.00

Time of Concentration

Sheet Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.08	0.00	0.00
Flow Length (ft) :	100	0.00	0.00
Slope (%) :	5	0.00	0.00
2 yr, 24 hr Rainfall (in) :	2.50	0.00	0.00
Velocity (ft/sec) :	0.36	0.00	0.00
Computed Flow Time (min) :	4.65	0.00	0.00

Shallow Concentrated Flow Computations	Subarea A	Subarea B	Subarea C
Flow Length (ft) :	1504.30	0.00	0.00
Slope (%) :	15	0.00	0.00
Surface Type :	Unpaved	Unpaved	Unpaved
Velocity (ft/sec) :	6.25	0.00	0.00
Computed Flow Time (min) :	4.01	0.00	0.00

Channel Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.03	0.00	0.00
Flow Length (ft) :	1593.22	0.00	0.00
Channel Slope (%) :	.1	0.00	0.00
Cross Section Area (ft^2) :	13.5	0.00	0.00
Wetted Perimeter (ft) :	18.45	0.00	0.00
Velocity (ft/sec) :	1.28	0.00	0.00
Computed Flow Time (min) :	20.82	0.00	0.00

Total TOC (min) 29.48

Subbasin Runoff Results

Total Rainfall (in) 4.31
Total Runoff (in) 1.68
Peak Runoff (cfs) 134.04
Weighted Curve Number 72.00
Time of Concentration (days hh:mm:ss) 0 00:29:29

Subbasin : {Site.2}.RunOff_S

Input Data

Area (ac) 58.04
Weighted Curve Number 72.00
Rain Gage ID .... Rain Gage-01

Composite Curve Number

Soil/Surface Description	Area (acres)	Soil Group	Curve Number
-	58.04	-	72.00
Composite Area & Weighted CN	58.04		72.00

Time of Concentration

Sheet Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.08	0.00	0.00
Flow Length (ft) :	100	0.00	0.00
Slope (%) :	5	0.00	0.00
2 yr, 24 hr Rainfall (in) :	2.50	0.00	0.00
Velocity (ft/sec) :	0.36	0.00	0.00
Computed Flow Time (min) :	4.65	0.00	0.00

Shallow Concentrated Flow Computations	Subarea A	Subarea B	Subarea C
Flow Length (ft) :	1477.27	0.00	0.00
Slope (%) :	16	0.00	0.00
Surface Type :	Unpaved	Unpaved	Unpaved
Velocity (ft/sec) :	6.45	0.00	0.00
Computed Flow Time (min) :	3.82	0.00	0.00

Channel Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.03	0.00	0.00
Flow Length (ft) :	366.88	0.00	0.00
Channel Slope (%) :	1	0.00	0.00
Cross Section Area (ft^2) :	19.1	0.00	0.00
Wetted Perimeter (ft) :	11.8	0.00	0.00
Velocity (ft/sec) :	6.85	0.00	0.00
Computed Flow Time (min) :	0.89	0.00	0.00

Total TOC (min) 9.36

Subbasin Runoff Results

Total Rainfall (in) 4.31
Total Runoff (in) 1.68
Peak Runoff (cfs) 247.24
Weighted Curve Number 72.00
Time of Concentration (days hh:mm:ss) 0 00:09:22

Subbasin : {Site.2}.RunOff_W

Input Data

Area (ac)	67.83
Weighted Curve Number	72.00
Rain Gage ID	Rain Gage-01

Composite Curve Number

Soil/Surface Description	Area (acres)	Soil Group	Curve Number
-	67.83	-	72.00
Composite Area & Weighted CN	67.83		72.00

Time of Concentration

Sheet Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.08	0.00	0.00
Flow Length (ft) :	100	0.00	0.00
Slope (%) :	5	0.00	0.00
2 yr, 24 hr Rainfall (in) :	2.50	0.00	0.00
Velocity (ft/sec) :	0.36	0.00	0.00
Computed Flow Time (min) :	4.65	0.00	0.00

Shallow Concentrated Flow Computations	Subarea A	Subarea B	Subarea C
Flow Length (ft) :	1498.48	0.00	0.00
Slope (%) :	16	0.00	0.00
Surface Type :	Unpaved	Unpaved	Unpaved
Velocity (ft/sec) :	6.45	0.00	0.00
Computed Flow Time (min) :	3.87	0.00	0.00

Channel Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.03	0.00	0.00
Flow Length (ft) :	700	0.00	0.00
Channel Slope (%) :	1	0.00	0.00
Cross Section Area (ft^2) :	19.1	0.00	0.00
Wetted Perimeter (ft) :	11.8	0.00	0.00
Velocity (ft/sec) :	6.85	0.00	0.00
Computed Flow Time (min) :	1.70	0.00	0.00

Total TOC (min) 10.22

Subbasin Runoff Results

Total Rainfall (in)	4.31
Total Runoff (in)	1.68
Peak Runoff (cfs)	279.91
Weighted Curve Number	72.00
Time of Concentration (days hh:mm:ss)	0 00:10:13

Subbasin : {Site.2}.RunOn_1

Input Data

Area (ac) 45.54
Weighted Curve Number 72.00
Rain Gage ID .... Rain Gage-01

Composite Curve Number

Soil/Surface Description	Area (acres)	Soil Group	Curve Number
-	45.54	-	72.00
Composite Area & Weighted CN	45.54		72.00

Time of Concentration

Sheet Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.08	0.00	0.00
Flow Length (ft) :	100	0.00	0.00
Slope (%) :	.5	0.00	0.00
2 yr, 24 hr Rainfall (in) :	2.50	0.00	0.00
Velocity (ft/sec) :	0.14	0.00	0.00
Computed Flow Time (min) :	11.67	0.00	0.00

Shallow Concentrated Flow Computations	Subarea A	Subarea B	Subarea C
Flow Length (ft) :	580.37	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) :	8.48	0.00	0.00

Channel Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.03	0.00	0.00
Flow Length (ft) :	2148.11	0.00	0.00
Channel Slope (%) :	.1	0.00	0.00
Cross Section Area (ft^2) :	19.1	0.00	0.00
Wetted Perimeter (ft) :	11	0.00	0.00
Velocity (ft/sec) :	2.27	0.00	0.00
Computed Flow Time (min) :	15.78	0.00	0.00

Total TOC (min) 35.94

Subbasin Runoff Results

Total Rainfall (in) 4.31
Total Runoff (in) 1.68
Peak Runoff (cfs) 86.81
Weighted Curve Number 72.00
Time of Concentration (days hh:mm:ss) 0 00:35:56

Subbasin : {Site.2}.RunOn_2

Input Data

Area (ac) 77.13
Weighted Curve Number 72.00
Rain Gage ID .... Rain Gage-01

Composite Curve Number

Soil/Surface Description	Area (acres)	Soil Group	Curve Number
-	77.13	-	72.00
Composite Area & Weighted CN	77.13		72.00

Time of Concentration

Sheet Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.08	0.00	0.00
Flow Length (ft) :	100	0.00	0.00
Slope (%) :	.5	0.00	0.00
2 yr, 24 hr Rainfall (in) :	2.50	0.00	0.00
Velocity (ft/sec) :	0.14	0.00	0.00
Computed Flow Time (min) :	11.67	0.00	0.00

Shallow Concentrated Flow Computations	Subarea A	Subarea B	Subarea C
Flow Length (ft) :	650.31	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) :	6.73	0.00	0.00

Channel Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.03	0.00	0.00
Flow Length (ft) :	2114.97	0.00	0.00
Channel Slope (%) :	.6	0.00	0.00
Cross Section Area (ft²) :	19.1	0.00	0.00
Wetted Perimeter (ft) :	11.8	0.00	0.00
Velocity (ft/sec) :	5.30	0.00	0.00
Computed Flow Time (min) :	6.65	0.00	0.00

Total TOC (min) 25.05

Subbasin Runoff Results

Total Rainfall (in) 4.31
Total Runoff (in) 1.68
Peak Runoff (cfs) 193.05
Weighted Curve Number 72.00
Time of Concentration (days hh:mm:ss) 0 00:25:03

Subbasin : {Site.2}.RunOn_3

Input Data

Area (ac) 40.44
Weighted Curve Number 72.00
Rain Gage ID .... Rain Gage-01

Composite Curve Number

Soil/Surface Description	Area (acres)	Soil Group	Curve Number
-	40.44	-	72.00
Composite Area & Weighted CN	40.44		72.00

Time of Concentration

Sheet Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.08	0.00	0.00
Flow Length (ft) :	100	0.00	0.00
Slope (%) :	.5	0.00	0.00
2 yr, 24 hr Rainfall (in) :	2.50	0.00	0.00
Velocity (ft/sec) :	0.14	0.00	0.00
Computed Flow Time (min) :	11.67	0.00	0.00

Shallow Concentrated Flow Computations	Subarea A	Subarea B	Subarea C
Flow Length (ft) :	1529.22	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) :	22.36	0.00	0.00

Channel Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.03	0.00	0.00
Flow Length (ft) :	4033.14	0.00	0.00
Channel Slope (%) :	.75	0.00	0.00
Cross Section Area (ft^2) :	19.1	0.00	0.00
Wetted Perimeter (ft) :	11.8	0.00	0.00
Velocity (ft/sec) :	5.93	0.00	0.00
Computed Flow Time (min) :	11.34	0.00	0.00

Total TOC (min) 45.37

Subbasin Runoff Results

Total Rainfall (in) 4.31
Total Runoff (in) 1.68
Peak Runoff (cfs) 63.40
Weighted Curve Number 72.00
Time of Concentration (days hh:mm:ss) 0 00:45:22

Subbasin : {Site.2}.RunOn_4-EVAP-PONDS

Input Data

Area (ac) 34.70
Weighted Curve Number 72.00
Rain Gage ID .... Rain Gage-01

Composite Curve Number

Soil/Surface Description	Area (acres)	Soil Group	Curve Number
-	34.70	-	72.00
Composite Area & Weighted CN	34.70		72.00

Time of Concentration

Sheet Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.08	0.00	0.00
Flow Length (ft) :	100	0.00	0.00
Slope (%) :	1	0.00	0.00
2 yr, 24 hr Rainfall (in) :	2.50	0.00	0.00
Velocity (ft/sec) :	0.19	0.00	0.00
Computed Flow Time (min) :	8.85	0.00	0.00

Total TOC (min) 8.85

Subbasin Runoff Results

Total Rainfall (in) 4.31
Total Runoff (in) 1.68
Peak Runoff (cfs) 150.92
Weighted Curve Number 72.00
Time of Concentration (days hh:mm:ss) 0 00:08:51

Subbasin : {Site.2}.RunOn_5

Input Data

Area (ac) 18.33
Weighted Curve Number 72.00
Rain Gage ID .... Rain Gage-01

Composite Curve Number

Soil/Surface Description	Area (acres)	Soil Group	Curve Number
-	18.33	-	72.00
Composite Area & Weighted CN	18.33		72.00

Time of Concentration

Sheet Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.08	0.00	0.00
Flow Length (ft) :	100	0.00	0.00
Slope (%) :	.5	0.00	0.00
2 yr, 24 hr Rainfall (in) :	2.50	0.00	0.00
Velocity (ft/sec) :	0.14	0.00	0.00
Computed Flow Time (min) :	11.67	0.00	0.00

Shallow Concentrated Flow Computations	Subarea A	Subarea B	Subarea C
Flow Length (ft) :	576.27	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) :	6.67	0.00	0.00

Channel Flow Computations	Subarea A	Subarea B	Subarea C
Manning's Roughness :	.03	0.00	0.00
Flow Length (ft) :	741.70	0.00	0.00
Channel Slope (%) :	1	0.00	0.00
Cross Section Area (ft²) :	19.1	0.00	0.00
Wetted Perimeter (ft) :	11.8	0.00	0.00
Velocity (ft/sec) :	6.85	0.00	0.00
Computed Flow Time (min) :	1.81	0.00	0.00

Total TOC (min) 20.15

Subbasin Runoff Results

Total Rainfall (in) 4.31
Total Runoff (in) 1.68
Peak Runoff (cfs) 53.24
Weighted Curve Number 72.00
Time of Concentration (days hh:mm:ss) 0 00:20:09

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	RunOffE-In	3593.50	3597.00	3.50	3593.50	0.00	3597.00	0.00	0.00	0.00
2	RunOn1RunOffN-In	3578.00	3580.00	2.00	3578.00	0.00	3580.00	0.00	0.00	0.00
3	RunOn2In	3577.00	3580.00	3.00	3577.00	0.00	3580.00	0.00	0.00	0.00
4	RunOn3-In	3570.00	3572.00	2.00	3570.00	0.00	3572.00	0.00	0.00	0.00
5	SE-ConvergePt	3561.00	3564.50	3.50	3561.00	0.00	3564.50	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	RunOffE-In	234.92	234.92	3597.00	3.50	0.00	0.00	3594.10	0.60	0 07:03	0 07:10	22.11	15.00
2	RunOn1RunOffN-In	217.16	217.16	3580.82	2.82	0.00	0.18	3578.20	0.20	0 07:23	0 00:00	0.00	0.00
3	RunOn2In	191.48	191.48	3580.00	3.00	0.00	0.00	3577.24	0.24	0 07:10	0 07:15	20.01	19.00
4	RunOn3-In	174.47	63.39	3572.31	2.31	0.00	0.69	3570.18	0.18	0 07:31	0 00:00	0.00	0.00
5	SE-ConvergePt	238.09	0.00	3564.16	3.16	0.00	0.34	3561.33	0.33	0 07:28	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	EastBoundChannel	2227.00	3577.00	0.00	3570.00	0.00	7.00	0.3100	Trapezoidal	3.000	18.000	0.0320	0.5000	0.5000	0.0000	0.00	No
2	EastChan2SE_Pond	865.00	3561.00	0.00	3557.00	19.00	4.00	0.4600	Trapezoidal	3.500	24.000	0.0320	0.5000	0.5000	0.0000	0.00	No
3	EastLFBermTop	3800.00	3594.00	0.50	3561.00	0.00	33.00	0.8700	Trapezoidal	3.000	9.400	0.0320	0.5000	0.5000	0.0000	0.00	No
4	SEdiagonalChan	913.00	3570.00	0.00	3561.00	0.00	9.00	0.9900	Trapezoidal	2.500	17.500	0.0320	0.5000	0.5000	0.0000	0.00	No
5	S-RunOff2SE_pond	1175.00	3564.00	1.00	3557.00	19.00	7.00	0.6000	Trapezoidal	2.000	9.000	0.0320	0.5000	0.5000	0.0000	0.00	No
6	S-RunOff2SW_pond	1479.07	3563.00	0.00	3550.00	5.00	13.00	0.8800	Trapezoidal	2.500	16.000	0.0320	0.5000	0.5000	0.0000	0.00	No
7	WestBoundChan	3127.89	3578.00	0.00	3550.00	0.00	28.00	0.9000	Trapezoidal	3.000	18.000	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	EastBoundChannel	111.67	0 07:28	141.45	0.79	3.75	9.90	2.62	0.88	0.00		
2	EastChan2SE_Pond	215.88	0 07:28	329.23	0.66	5.38	2.68	2.63	0.75	0.00		
3	EastLFBermTop	105.52	0 07:18	114.49	0.92	8.45	7.50	2.85	0.95	0.00		
4	SEdiagonalChan	170.02	0 07:31	203.04	0.84	5.74	2.65	2.40	0.96	0.00		
5	S-RunOff2SE_pond	25.54	0 07:12	35.98	0.71	3.55	5.52	1.59	0.83	0.00		
6	S-RunOff2SW_pond	152.12	0 07:12	163.82	0.93	6.18	3.99	2.31	0.93	0.00		
7	WestBoundChan	207.70	0 07:23	238.72	0.87	6.35	8.21	2.80	0.94	0.00		

Storage Nodes

Storage Node : EvapPonds

Input Data

Invert Elevation (ft) 3579.00
Max (Rim) Elevation (ft) 3581.00
Max (Rim) Offset (ft) 2.00
Initial Water Elevation (ft) 3579.00
Initial Water Depth (ft) 0.00
Ponded Area (ft^2) 125000.00
Evaporation Loss 0.00

Output Summary Results

Peak Inflow (cfs) 137.27
Peak Lateral Inflow (cfs) 137.27
Peak Outflow (cfs) 0.00
Peak Exfiltration Flow Rate (cfm) 0.00
Max HGL Elevation Attained (ft) 3580.69
Max HGL Depth Attained (ft) 1.69
Average HGL Elevation Attained (ft) 3580.25
Average HGL Depth Attained (ft) 1.25
Time of Max HGL Occurrence (days hh:mm) 1 00:30
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 : SE-DrainPond

Input Data

Invert Elevation (ft) 3538.00
Max (Rim) Elevation (ft) 3557.00
Max (Rim) Offset (ft) 19.00
Initial Water Elevation (ft) 3538.00
Initial Water Depth (ft) 0.00
Ponded Area (ft^2) 125000.00
Evaporation Loss 0.00

Output Summary Results

Peak Inflow (cfs) 216.04
Peak Lateral Inflow (cfs) 0.00
Peak Outflow (cfs) 0.00
Peak Exfiltration Flow Rate (cfm) 0.00
Max HGL Elevation Attained (ft) 3552.33
Max HGL Depth Attained (ft) 14.33
Average HGL Elevation Attained (ft) 3548.58
Average HGL Depth Attained (ft) 10.58
Time of Max HGL Occurrence (days hh:mm) 1 12:00
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 : SW-DrainPond

Input Data

Invert Elevation (ft) 3545.00
Max (Rim) Elevation (ft) 3550.00
Max (Rim) Offset (ft) 5.00
Initial Water Elevation (ft) 3545.00
Initial Water Depth (ft) 0.00
Ponded Area (ft^2) 107652.00
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	Weir-01	Trapezoidal	No	3548.00	3.00	22.00	1.00	3.33

Output Summary Results

Peak Inflow (cfs) 202.77
Peak Lateral Inflow (cfs) 52.02
Peak Outflow (cfs) 13.50
Peak Exfiltration Flow Rate (cfm) 0.00
Max HGL Elevation Attained (ft) 3548.32
Max HGL Depth Attained (ft) 3.32
Average HGL Elevation Attained (ft) 3547.46
Average HGL Depth Attained (ft) 2.46
Time of Max HGL Occurrence (days hh:mm) 0 07:53
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

APPLICATION FOR PERMIT
OWL LANDFILL SERVICES, LLC

VOLUME III: LANDFILL ENGINEERING CALCULATIONS
SECTION 4: HELP MODEL

TABLE OF CONTENTS

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APPLICATION FOR PERMIT
OWL LANDFILL SERVICES, LLC

VOLUME III: LANDFILL ENGINEERING CALCULATIONS
SECTION 4: HELP MODEL

1.0 INTRODUCTION

OWL Landfill Services, LLC (OWL) is proposing to permit, construct, and operate a "Surface Waste Management Facility" for oil field waste processing and disposal services. The proposed OWL Facility is subject to regulation under the New Mexico (NM) Oil and Gas Rules, specifically 19.15.36 NMAC, administered by the Oil Conservation Division (OCD). The Facility has been designed in compliance with the requirements of 19.15.36 NMAC, and will be constructed, operated, and closed in compliance with a Surface Waste Management Facility Permit issued by the OCD.

The OWL Facility is one of the first designed to the new more stringent standards that, for instance, mandate double liners and leak detection for land disposal. The new services that OWL will provide fill a necessary void in the market for technologies that exceed current OCD requirements.

1.1 Site Location

The OWL site is located approximately 22 miles northwest of Jal, adjacent to the south of NM 128 in Lea County, NM. The OWL site is comprised of a 560-acre ± tract of land located within a portion of Section 23, Township 24 South, Range 33 East, Lea County, NM (**Figure IV.1.1**). Site access will be provided on the south side of NM 128. The approximate center, surface, coordinates of the OWL site are Latitude 32.203105577 and Longitude -103.543122319.

1.2 Description

The OWL Surface Waste Management Facility will comprise approximately 500 acres of the 560-acre site, and will include two main components: an oil field waste Processing Area and an oil field waste Landfill Disposal Area, as well as related infrastructure. Oil field wastes are anticipated to be delivered to the OWL Facility 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 Area.

2.0 DESIGN CRITERIA

An alternate design for the OWL Surface Waste Management Facility landfill liner system that includes the use of geosynthetics and geocomposites is proposed. In addition, an alternate design is proposed for its final cover system using on-site soils. The alternative liner and final cover are designed to meet the requirements of the New Mexico Oil Conservation Division (OCD) 19.15.36.14C NMAC. If an alternate liner design and alternate final cover design using geosynthetics or geocomposites 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

Throughout the past year and a half, OCD and its consultants have provided guidance and clarification to our understanding of 19.15.36 NMAC. The result of which has had an impact on the application of several design technical models and the associated effect on other design elements. One such impacted model is the United State Environmental Protection Agency (USEPA) Hydrologic Evaluation of Landfill Performance (HELP) Model which evaluates the performance of alternative liner designs, demonstrating the alternative design will perform as stipulated, i.e., *The operator shall design the landfill to prevent the "bathtub effect", (see citation above).* Updated application and associated input parameters resulted in the following revised sections to this document.

This document presents the results of modeling conducted using HELP Model to evaluate the performance of the alternate final cover system so as to not create a "bathtub effect" in the landfill, in which the percolation through the alternate final cover exceeds that of the alternate liner system. Also presented is a formal request for OCD approval to utilize the alternate liner design and allow the use of alternate soil gradation specifications for soils used in construction of the protective soil layer (PSL).

4.0 HELP MODEL METHODOLOGY

The methodology used to demonstrate that the performance of the alternate liner system design will prevent the bathtub effect relies on the USEPA's HELP Modeling program as referenced in 19.15.36.14C(9) NMAC. The demonstrations described below were performed using the HELP Model, Version 3.07a.

5.0 OVERVIEW OF DEMONSTRATION MODELING

Gordon Environmental, Inc. (GEI) has prepared performance demonstrations for an alternate landfill liner design and an alternate landfill final cover design. In the proposed alternate liner design, on-site soils in conjunction with a geocomposite are used for the leachate collection layer; a geonet is used as the leak detection layer; and a geocomposite clay liner (GCL) along with 6-inches of compacted subsurface soils are used to replace the prescribed clay barrier layer. In the proposed alternate final cover design, an evapotranspiration (ET) cover system is proposed.

Because the OWL Surface Waste Management Facility is planning to use alternate designs for its liner system and final cover system, the HELP model simulation analyses were organized to support three demonstrations:

- First, demonstrate the performance of the planned alternate liner system to establish a basis of comparative analysis for the planned alternative final cover system.
- Second, demonstrate that percolation through the alternate final cover top surface does not create a "bathtub effect" within the landfill.

- Third, demonstrate that percolation through the alternate final cover sideslopes do not create a “bathtub effect” within the landfill.

6.0 HELP MODEL DEMONSTRATION ANALYSES

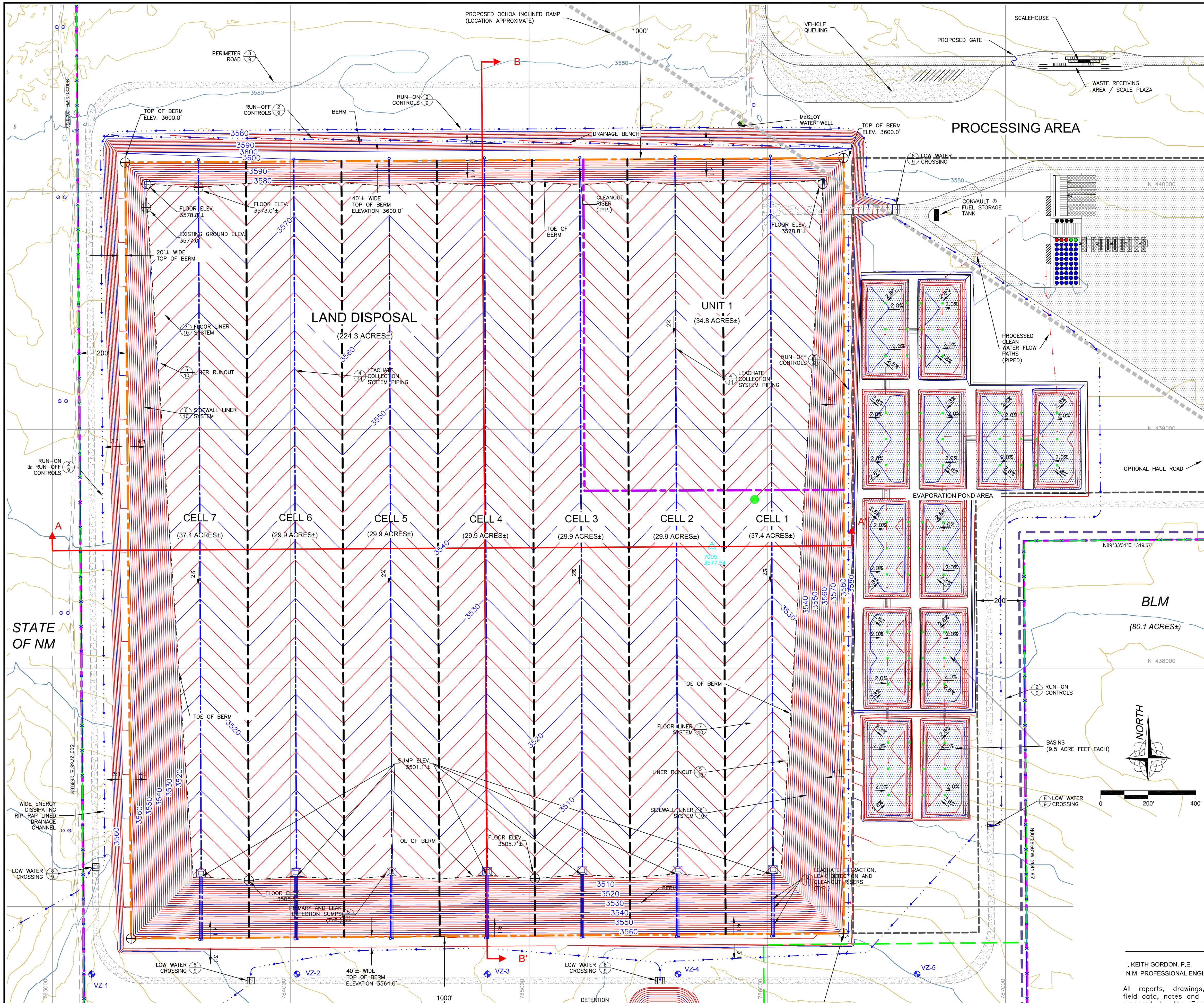
In each of the following three demonstrations, the input parameters for the HELP model have been selected utilizing guidance from the “Users Guide for Version 3” as provided by the USEPA (**Attachment III.4.C**).

6.1 Cell Design Parameters

Slope steepness and lateral drainage distance were derived from the design parameters for the cells in the landfill. The liner system in Unit 1 has the flattest floor slope and the longest lateral drainage distance (see **Figure III.4.1**). The top portion of the final cover system has a relatively uniform average slope of 2.8%; the longest lateral drainage distance occurs from the crown of the landfill to sideslope (see **Figure III.4.2**). Throughout these analyses, the following design parameters have been used:

- Liner system:
    - lateral drainage distance = 300 ft
    - slope = 2.8%
- Final cover system:
    - Top:
        - lateral drainage distance = 306 ft
        - slope = 2.3%
    - Sideslopes:
        - lateral drainage distance = 1269 ft
        - slope = 16.4%

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.D**).



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Date/Time:Sep. 21, 2016-09:22:47 ; LAYOUT: 04-Base

LEGEND

- SITE BOUNDARY WITH BEARING AND DISTANCE (559.5 ACRES±)
- SURFACE WASTE MANAGEMENT FACILITY BOUNDARY (500.0 ACRES)
- SOLID WASTE DISPOSAL AREA LIMITS (224.3 ACRES±)
- PROCESSING AREA LIMITS (81.0 ACRES±)
- UNIT 1 BOUNDARY (34.8 ACRES±)
- CELL BOUNDARY
- EXISTING 2FT CONTOUR
- EXISTING 10FT CONTOUR
- EXISTING 2FT DEPRESSION CONTOUR
- EXISTING 10FT DEPRESSION CONTOUR
- EXISTING UNPAVED ROAD
- EXISTING OVERHEAD ELECTRIC LINE
- EXISTING POWER POLE
- PROPOSED 2FT BASE GRADE CONTOUR
- PROPOSED 10FT BASE GRADE CONTOUR
- PROPOSED DEVON PIPELINE
- PROPOSED UNPAVED ROAD (GRAVEL)
- PROPOSED UNPAVED ROAD (SOIL)
- PROPOSED 3-STRAND BARBED WIRE FENCE
- FINAL DRAINAGE FLOW LINE AND DIRECTION OF FLOW
- PROPOSED PROCESS WATER FLOW PATHS
- PROPOSED VADOSE ZONE MONITORING WELL
- LEACHATE COLLECTION SUMP – WITH EXTRACTION, LEAK DETECTION AND CLEANOUT RISER PIPES
- BOREHOLE LOCATION
- PRODUCED WATER TANK
- CRUDE OIL RECOVERY TANK
- OIL SALES TANK
- CROSS-SECTION LOCATION
- SURVEY CONTROL POINT
- SITE GRID

NOTES:

1. AERIAL TOPOGRAPHIC SURVEY BY AEROTECH MAPPING INC., 6565 AMERICAN PARKWAY N.E., ALBUQUERQUE, NM 87111 PHONE: (520-561-6537) FAX (505 256-3328) EMAIL: TimBurrows@atmlv.com DATE OF PHOTOGRAPHY: 06-06-2015.
2. SURVEY CONTROL POINTS BY HARCROW SURVEYING, INC., 2314 W. MAIN ST., ARTESIA, NM 88210 PHONE: (575-746-2158).
3. THE DESIGN OF THE FACILITIES SHOWN IS PRELIMINARY. CONSTRUCTION PLANS AND SPECIFICATIONS FOR EACH MAJOR ELEMENT WILL BE SUBMITTED TO OCD IN ADVANCE OF INSTALLATION.

CONTROL POINT DATA

POINT	NORTHING	EASTING	PANEL ELEVATION	DESCRIPTION
7001	434845.57	782160.25	3530.07	PP-7001
7002	438508.13	782138.97	3561.67	PP-7002
7003	442131.34	782096.47	3600.88	PP-7003
7004	434859.95	785795.31	3548.81	PP-7004
7005	438509.15	785767.60	3577.54	PP-7005
7006	442207.86	785773.10	3598.28	PP-7006
7007	434883.93	789423.36	3567.38	PP-7007
7008	438485.16	789417.70	3577.60	PP-7008

NOTES:

1. ALL POINTS ARE FLUSH WITH THE GROUND.
2. THE COORDINATES AND ELEVATIONS FOR THE PHOTO CONTROL POINTS ON THE ABOVE REFERENCED PROJECT ARE MODIFIED (SURFACE) NEW MEXICO STATE PLANE COORDINATES – EAST ZONE, NAD 83 AND HAVE BEEN ADJUSTED USING AN "OPUS" SOLUTION". TO OBTAIN TRUE STATE PLANE GRID COORDINATES, MULTIPLY THE COORDINATES BELOW BY THE PROJECT AVERAGE COMBINED FACTOR CF = 0.99981059648. THE COORDINATES AND ELEVATIONS ARE EXPRESSED IN U. S. SURVEY FEET.

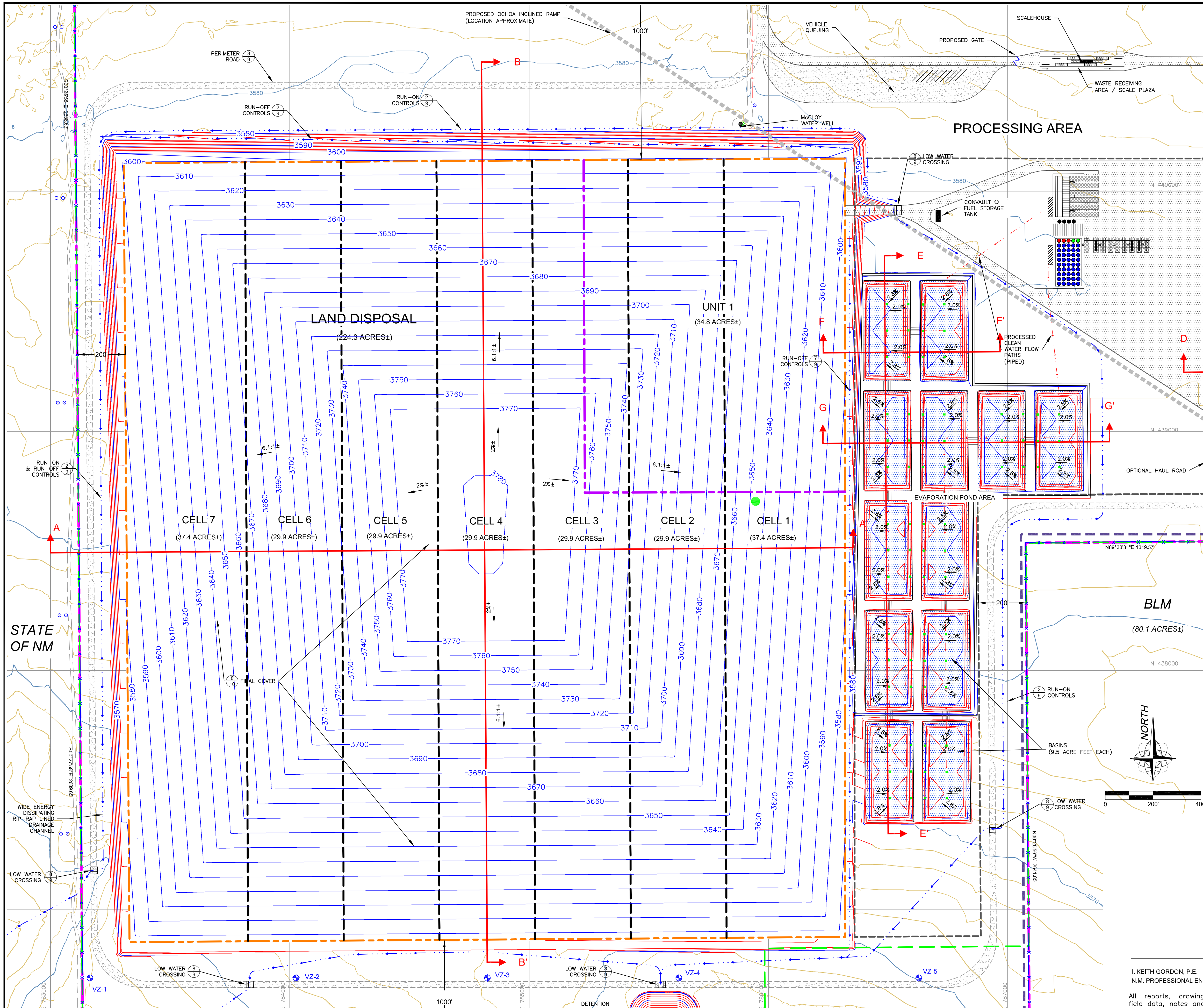
I. KEITH GORDON, P.E.
N.M. PROFESSIONAL ENGINEER NO. 10984

LANDFILL BASE GRADING PLAN

OWL LANDFILL SERVICES, LLC
LEA COUNTY, NEW MEXICO

Gordon Environmental, Inc.
Consulting Engineers
213 S. Camino del Pueblo
Bernalillo, New Mexico, USA
Phone: 505-867-6990
Fax: 505-867-6991

DATE: 09/02/2016	CAD: WholeSi~2.DWG	PROJECT #: 560.01.02
DRAWN BY: ASM	REVIEWED BY: CRK	FIGURE III.4.1
APPROVED BY: IKG	gei@gordonenvironmental.com	



LEGEND

SITE BOUNDARY WITH BEARING AND DISTANCE	S88°50'44.29"E 2647.98'	(559.5 ACRES±)
SURFACE WASTE MANAGEMENT FACILITY BOUNDARY		(500.0 ACRES)
SOLID WASTE DISPOSAL AREA LIMITS		(224.3 ACRES±)
PROCESSING AREA LIMITS		(81.0 ACRES±)
UNIT 1 BOUNDARY		(34.8 ACRES±)
CELL BOUNDARY		
EXISTING 2FT CONTOUR		
EXISTING 10FT CONTOUR	3550	
EXISTING 2FT DEPRESSION CONTOUR		
EXISTING 10FT DEPRESSION CONTOUR	3550	
EXISTING PAVED ROAD		
EXISTING UNPAVED ROAD		
EXISTING OVERHEAD ELECTRIC LINE	E	
EXISTING POWER POLE		
PROPOSED 2FT BASE GRADE CONTOUR		
PROPOSED 10FT BASE GRADE CONTOUR	3550	
PROPOSED DEVON PIPELINE		
PROPOSED UNPAVED ROAD (GRAVEL)		
PROPOSED UNPAVED ROAD (SOIL)		
PROPOSED 3-STRAND BARBED WIRE FENCE		
FINAL DRAINAGE FLOW LINE AND DIRECTION OF FLOW		
PROPOSED PROCESS WATER FLOW PATHS		
PROPOSED VADOSE ZONE MONITORING WELL	VZ-2	
BOREHOLE LOCATION		
PRODUCED WATER TANK		
CRUDE OIL RECOVERY TANK		
OIL SALES TANK		
MECHANICAL EVAPORATOR LOCATION		
CROSS-SECTION LOCATION	A A'	
SURVEY CONTROL POINT	7008 3577.60	
SITE GRID	E 7880000 N 439000	

NOTES:

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7004	434859.95	785795.31	3548.81	PP-7004
7005	438509.15	785767.60	3577.54	PP-7005
7006	442207.86	785773.10	3598.28	PP-7006
7007	434883.93	789423.36	3567.38	PP-7007
7008	438485.16	789417.70	3577.60	PP-7008

NOTES:

1. ALL POINTS ARE FLUSH WITH THE GROUND.
2. THE COORDINATES AND ELEVATIONS FOR THE PHOTO CONTROL POINTS ON THE ABOVE REFERENCED PROJECT ARE MODIFIED (SURFACE) NEW MEXICO STATE PLANE COORDINATES – EAST ZONE, NAD 83 AND HAVE BEEN ADJUSTED USING AN "OPUS" SOLUTION". TO OBTAIN TRUE STATE PLANE GRID COORDINATES, MULTIPLY THE COORDINATES BELOW BY THE PROJECT AVERAGE COMBINED FACTOR CF = 0.99981059648. THE COORDINATES AND ELEVATIONS ARE EXPRESSED IN U. S. SURVEY FEET.

I. KEITH GORDON, P.E.
N.M. PROFESSIONAL ENGINEER NO. 10984

LANDFILL FINAL GRADING PLAN

OWL LANDFILL SERVICES, LLC
LEA COUNTY, NEW MEXICO

Gordon Environmental, Inc.
Consulting Engineers
213 S. Camino del Pueblo
Bernalillo, New Mexico, USA
Phone: 505-867-6990
Fax: 505-867-6991

DATE: 09/08/2016	CAD: WholeSi~2.DWG	PROJECT #: 560.01.02
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APPROVED BY: IKG	gei@gordonenvironmental.com	

NOT FOR CONSTRUCTION
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Date/Time:Sep. 21, 2016-09:31:07 ; LAYOUT: 05-Final

6.2 Alternate Liner Demonstration

The HELP model simulation analysis has been performed to support the EPA's HELP model as per 19.15.36.14C(9) NMAC.

6.2.1 Liner System Design

The design for the alternate liner system includes the following layers from the top down:

- 24-inches protective soil layer (on-site soils) (k = 4.2 x 10^{-5} cm/sec)
- FML (60-mil smooth HDPE)
- 200-mil geonet (k = 10 cm/sec)
- FML (60-mil smooth HDPE)
- GCL (k = 3.0 x 10^{-9} cm/sec)

6.2.2 HELP Model Input Parameters

6.2.2.1 Soils

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. As part of its design for the alternate liner system, OWL proposes to use on-site soils in the protective soil layer that contain no more than 30% fines by weight and a uniformity coefficient less than 10.

Geotechnical analyses of on-site soils indicate that the soils available at the OWL Surface Waste Management Facility site consist primarily of sand with varying amounts of fines (CL, low plasticity sandy clay, SP-SM, poorly graded sand – silty sand) and that they meet the proposed criteria for the protective soil layer. **Attachment III.4.B** provides a summary of geotechnical test results. The on-site soil that OWL proposes to use when it places the PSL is within the range of soil type used in this modeling based on sieve analyses and hydraulic conductivity (**Attachment III.4.B**). The type of soil used to represent the protective soil layer in the simulation is listed below:

Soil Description	HELP Model Soil Type	USCS Soil Type
low plasticity sandy clay	12	CL

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.

6.2.2.2 Environmental

All of the simulation analyses for HELP modeling demonstrations were performed using identical environmental loading conditions. Precipitation and temperature data were derived from the Western Regional Climatic Center's database. The nearest location with sufficient data is Ochoa, New Mexico. Solar radiation data was synthetically generated by the HELP model based on coefficients for Midland, Texas. Midland, Texas 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.C**). Evapotranspiration data (e.g., average wind speed and seasonal relative humidity) was obtained from Ochoa, New Mexico, with coefficients for Midland, TX. The evaporative zone depth was set to 24 inches and the maximum leaf area index was set to 0.0, i.e., bare ground. The surface layer, PSL, was modeled as having no vegetation.

6.2.2.3 Initial Conditions

The following alternate liner component default values for HELP Model Soil Texture Classes and Material Characteristics were used in the simulations:

- Protective Soil Layer
  - Soil Texture Class – 12
  - Total Porosity (vol/vol) - 0.471
  - Field Capacity (vol/vol) - 0.342
  - Wilting Point (vol/vol) – 0.210
  - Saturated Hydraulic Conductivity (cm/sec) – 4.2×10^{-5}
- Primary Liner
  - 60-mil smooth HDPE
  - Material Characteristic – 35
  - Saturated Hydraulic Conductivity (cm/sec) – 2.0×10^{-13}
- Leak Detection System
  - 200-mil Geonet

    - Material Characteristic – 20
    - Saturated Hydraulic Conductivity (cm/sec) – 1.0 x 10^{+1}
- Secondary Liner
    - 60-mil smooth HDPE
    - Material Characteristic – 35
    - Saturated Hydraulic Conductivity (cm/sec) – 2.0 x 10^{-13}
- GCL(Geosynthetic Clay Liner)
    - Material Characteristic – 17
    - Total Porosity (vol/vol) - 0.750
    - Field Capacity (vol/vol) - 0.747
    - Wilting Point (vol/vol) - 0.400
    - Saturated Hydraulic Conductivity (cm/sec) – 3.0 x 10^{-9}

6.2.3 Alternate Liner Simulation Analysis

In the alternate liner simulation analyses, the landfill has been assumed to be in an open condition with no waste present. All precipitation is retained within the landfill; there is no runoff. The FML was represented by using the default parameters for Material Characteristic type 35 from the HELP model. The input parameters used to represent the alternative liner system are provided in **Table III.4.1**.

6.2.4 Alternate Liner Demonstration Results

According to 19.15.36.14C(9), an alternate liner system is considered acceptable when supported by EPA's HELP model. Performance has been demonstrated to be sufficient in protection of the environment. The performance measure is the average annual rate of percolation through the bottom of the liner system and the head upon the liner. This is evaluated by the percolation rates calculated using the HELP model. The average annual percolation rate is summarized in **Table III.4.2**.

TABLE III.4.1
Alternative Liner System
OWL Landfill Services, LLC

Simulation	**Protective Drainage Layer**			**Primary FML**			**Leak Detection Layer**			**Secondary FML**			**Geocomposite Clay Liner**		
	HELP Model Soil Type	Layer Thickness (in)	K_{sat} (cm/s)	HELP Model Soil Type	FML	K_{sat} (cm/s)	HELP Model Soil Type	Layer Thickness (in)	K_{sat} (cm/s)	HELP Model Soil Type	Layer Thickness (in)	K_{sat} (cm/s)	HELP Model Soil Type	Layer Thickness (in)	K_{sat} (cm/s)
Alternative Liner System	12	24	4.2×10^{-5}	35	60-mil HDPE	2.0×10^{-13}	20	200-mil Geonet	10	35	60-mil HDPE	2.0×10^{-13}	17	0.23	2.0×10^{-13}

TABLE III.4.2
Performance Results for Alternate Liner System
OWL Landfill Services. LLC

Liner System	Soil Type for Protective Soil Layer	Average Annual Percolation Rate Through Bottom Liner (in/yr)	Average Annual Head on Primary HDPE Liner Layer 2 (in)
Alternate	12	0.00000	0.000

For the alternate liner system analyzed using on-site soils, the average annual percolation rate calculated is zero. In addition, the hydraulic head on the FML remains less than 12 inches. This simulation demonstrates that, for soils available on-site for use as the protective soil layer, the alternate liner system design provides performance that is supported by HELP modeling in accordance 19.15.36.14C(9).

6.3 Alternate Final Cover Demonstration

Two HELP model simulation analysis have been performed to support the alternative final cover demonstrations. In these demonstrations, the performance of the alternative final cover system is compared to the performance of the alternate liner system as to not to create a "bathtub effect" where percolation though the alternate final cover exceeds that of the alternate liner system.

6.3.1 Alternate Final Cover System Design

The alternate final cover system includes the following layers from the top down:

- 24-in. erosion/vegetative layer
- 6-in. barrier layer

6.3.2 HELP Model Input Parameters

6.3.2.1 Soils

The type of soil that was used to represent the barrier layer and erosion/vegetative layer in the simulation for the alternate final cover demonstration is listed below:

Soil Description	HELP Model Soil Type	USCS Soil Type
low plasticity sandy clay	12	CL

Default values from the HELP model were assigned to the porosity, field capacity and wilting point and an assumed hydraulic conductivity was used for each soil type as listed in Section 6.3.2.3 Initial Conditions.

The erosion/vegetative layer was assigned a HELP model soil type number that is the same as the barrier layer, and is most representative of conditions in the field for final cover construction activities. The HELP model automatically accounts for the effects of root penetration and decay whenever vegetation is assumed to be present on the surface layer.

6.3.2.2 Environmental

All of the simulation analyses for HELP modeling demonstrations were performed using identical environmental loading conditions. Precipitation and temperature data were derived from the Western Regional Climatic Center's database. The nearest location with sufficient data is Ochoa, New Mexico. Solar radiation data was synthetically generated by the HELP model based on coefficients for Midland, Texas. Midland, Texas 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.C**). Evapotranspiration data (e.g., average wind speed and seasonal relative humidity) was obtained from Ochoa, New Mexico, with coefficients from Midland TX. The evaporative zone depth was set to 24 inches and the maximum leaf area index was set to 1.2, as noted on Figure 3 in HELP Engineering Document for Version 3. Vegetation on the cover was modeled as "poor grass".

6.3.2.3 Initial Conditions

The following alternate final cover component default values for HELP Model Soil Texture Classes were used in the simulations:

- Erosion/Vegetative Soil Layer
  - Soil Texture Class – 12
  - Total Porosity (vol/vol) - 0.417
  - Field Capacity (vol/vol) - 0.342
  - Wilting Point (vol/vol) – 0.210
  - Saturated Hydraulic Conductivity (cm/sec) – 4.2×10^{-5}
- Barrier Soil Layer
  - Soil Texture Class – 12
  - Total Porosity (vol/vol) - 0. 417
  - Field Capacity (vol/vol) - 0.342
  - Wilting Point (vol/vol) – 0.210
  - Saturated Hydraulic Conductivity (cm/sec) – 4.2×10^{-5}

6.3.4 Alternate Cover Demonstration Results

According to 19.15.36.14C(9), an alternative cover is considered acceptable if its performance has been demonstrated to prevent the "bathtub effect". The measure is the average annual rate of percolation through the primary (upper-most) FML layer of the liner system and bottom layer of the cover system (Barrier Layer). Performance is evaluated by comparing the percolation rates calculated for the alternate cover system to that calculated for the alternate liner system. The average annual percolation rates calculated for the two systems are summarized in **Table III.4.3**.

TABLE III.4.3
Performance Results for Alternate Liner and Alternate Final Cover Systems
OWL Landfill Services, LLC

System	HELP Model Material Type		Average Annual Percolation Rate (in/yr)
	Primary FML Layer	Barrier Layer	
Alternate Final Cover	-	12	0.00000
Alternate Liner	12	-	0.00000

When the alternate cover system is modeled, in conjunction with, HELP model soil type 12, the rate of percolation calculated for the alternate final cover system is equivalent to the percolation rate calculated for the alternate liner system. The performance of the alternate final cover system design using soil type 12 prevents the “bathtub effect” as noted in 19.15.36.14C(9) NMAC.

7.0 CONCLUSIONS AND REQUEST FOR APPROVAL

OWL has prepared performance demonstrations for its alternate liner system design and for its alternate final cover system design. These analyses were based on 19.15.36.14C(9) NMAC when supported by the HELP model; and the analyses demonstrate the following:

- For the alternate liner simulation analysis, the average annual percolation rate calculated through the alternate liner system design is zero. This simulation demonstrates that the alternate liner system design provides superior performance. Therefore, the alternate liner system design meets the OCD demonstration requirements.
- In the alternate final cover simulation analyses, when the infiltration layer is modeled using HELP model soil type 12 and a hydraulic conductivity of 4.2×10^{-5} cm/sec, the average annual percolation rate calculated for the alternate final cover system is zero. Therefore, for this soil type, the performance of the alternate final cover system design meets the OCD demonstration requirements.
- In the simulation analyses, for the liner and final cover, the calculations for the fifth year demonstrates a zero percolation rate.

The HELP modeling for the analyses presented in this document demonstrates that the performance of the alternate liner and cover system designs meets the requirements of 19.15.36.14C NMAC. For the purposes of this demonstration, both the alternate liner design and the alternate cover design have been shown to be effective using soils available on the OWL site.

To allow OWL flexibility in using on-site soils as well as offsite materials to construct the protective soil layer, the erosion/vegetative layer and the barrier layer, this document serves as a request to OCD for approval to use the alternate liner and cover system designs and to construct those systems using soils that contain 30% fines and has a uniformity coefficient (Cu) less than 10.