

# **DRAINAGE DESIGN MANUAL**

**New Mexico  
Department of Transportation**



**July 2018**

# Drainage Design Manual

New Mexico Department of Transportation



Prepared by:

Smith Engineering Company



Occam Engineers Inc.



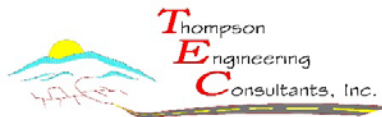
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# Foreword

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

Drainage Design Criteria, Fourth Revision, June 2007.

New Mexico Department of Transportation.

Drainage Manual Volume 1, Hydrology, 1995.

New Mexico State Highway and Transportation Department.

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

New Mexico State Highway and Transportation Department.

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

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# 100 INTRODUCTION

## 101 Drainage Design Manual Purpose and Use

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

### Previous Manuals and Documents

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

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

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

### Hotlinks and Cross-References

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

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

tables, equations, sections, appendices and example problems within this document are shown in **bold text**.

### **Drainage Design Manual Update**

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

Federal Highway Administration (FHWA) for hydraulics, erosion, sediment transport, scour and countermeasure design (for erosion and scour). The FHWA website hotlink listed here provides a full index of all current and archived FHWA publications.

[https://www.fhwa.dot.gov/engineering/hydraulics/library\\_listing.cfm?archived=false](https://www.fhwa.dot.gov/engineering/hydraulics/library_listing.cfm?archived=false)

Natural Resources Conservation Service (previously the Soil Conservation Service) Part 630, Hydrology, National Engineering Handbook, Chapters 1-22. Note that various Chapters have different dates. The Natural Resources Conservation Service (NRCS) website hotlink listed here will access this document.

<https://www.nrcs.usda.gov/wps/portal/nrcs/detailfull/national/water/manage/hydrology/?cid=stelprdb1043063>

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

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

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

### **The Drainage Analysis and Design Process Basics**

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

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

The results should be verified by considering the history and experience as reported by the local patrol foreman, local records, high water marks, historic aerial photography, “rules of thumb”,

and other computational methods. Conducting many drainage analyses will provide the experience that leads to developing good judgment, and will assist in exercising prudent engineering practice.

### **Drainage Infrastructure Past Performance**

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

### **Summary of Research**

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

### **Hydrology**

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

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

### **Hydraulics**

The previous Volume II (1998) manual was developed during a period when there was a nationwide push to convert highway design to metric standards. Since that time, the universal metrification effort has been largely abandoned in most DOTs around the United States

including the NMDOT. Many of the updates in this Drainage Design Manual with respect to Volume II, are related to conversion to English standard units from metric units.

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

## 102 Acronyms

AASHTO – American Association of State Highway and Transportation Officials

ADT – Average Daily Traffic

AMAFCA – Albuquerque Metropolitan Arroyo Flood Control Authority

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

BLM – Bureau of Land Management

BMP – Best Management Practice

CoCoRAS – Community Collaborative Rainfall, Hail and Snow Network

CFR – Code of Federal Regulations

COA – City of Albuquerque

CWA – Clean Water Act

DACFC – Doña Ana County Flood Commission

DDB – Drainage Design Bureau

DOT – Department of Transportation

EBID – Elephant Butte Irrigation District

EDAC – Earth Data Analysis Center

EPA – Environmental Protection Agency

ESCAFCA – Eastern Sandoval County Arroyo Flood Control Authority

FEMA – Federal Emergency Management Agency

FHWA – Federal Highway Administration

FIRM – Flood Insurance Rate Map

FIS – Flood Insurance Study

GI – Green Infrastructure

GIS – Geographic Information System

LID – Low Impact Development

LIDAR – Light Detection and Ranging

MRCOG – Mid-Region Council of Governments

MRGCD – Middle Rio Grande Conservancy District

MS4s – Municipal Separate Storm Sewer Systems

NEXRAD – Next Generation Radar

NMDGF – New Mexico Department of Game and Fish

NMDOT – New Mexico Department of Transportation

NMED – New Mexico Environment Department

NMIMT – New Mexico Institute of Mining and Technology

NMOSE – New Mexico Office of the State Engineer

NOAA – National Oceanic and Atmospheric Administration

NPDES – National Pollution Discharge Elimination System

NRCS – Natural Resources Conservation Service

NWS – National Weather Service

PDE – Project Development Engineer

RGIS – Resource Geographic Information System (New Mexico) National Weather Service

ROW – Right-of-Way

RSE – Relative Standard Error

SCS – Soil Conservation Service (now the NRCS)

SSCAFCA – Southern Sandoval County Arroyo Flood Control Authority

SWMP – Storm Water Management Plan

TESCP – Temporary Erosion and Sediment Control Plan

TMDL – Total Maximum Daily Load

USACE – U.S. Army Corps of Engineers

USBLM – U.S. Bureau of Land Management

USBR – U.S. Bureau of Reclamation

USDA – U.S. Department of Agriculture

USEPA – U.S. Environmental Protection Agency

USFS – U.S. Forest Service

USFWS – U.S. Fish and Wildlife Service

USGS – U.S. Geological Survey

USWB – U.S. Weather Bureau

## 103 References

Federal Highway Administration (FHWA), Website. A full index of all current and archived FHWA publications are located at the following website.

[https://www.fhwa.dot.gov/engineering/hydraulics/library\\_listing.cfm?archived=false](https://www.fhwa.dot.gov/engineering/hydraulics/library_listing.cfm?archived=false)

NMSHTD, December 1995, "Drainage Manual, Volume 1, Hydrology", Easterling & Associates, Inc.

<http://dot.state.nm.us/content/dam/nmdot/Infrastructure/NMHydrologyManual.pdf>

NMSHTD, November 1998, "Drainage Manual, Volume II, Hydraulics, Sedimentation and Erosion", Resource Technology, Inc.

<http://dot.state.nm.us/content/dam/nmdot/Infrastructure/NMHydraulicManual.pdf>

NMDOT, June 2007, “Drainage Design Criteria for New Mexico Department of Transportation Projects, Fourth Revision”, Smith Engineering Company and the NMDOT Drainage Design Bureau Engineers.

<http://dot.state.nm.us/content/dam/nmdot/Infrastructure/drainageDesignCriteria.pdf>

NRCS, “Part 630 Hydrology, National Engineering Handbook”. Note that various Chapters have different dates.

[https://www.nrcs.usda.gov/wps/portal/nrcs/detailfull/national/water/manage/hydrology/?cid=stelp\\_rdb1043063](https://www.nrcs.usda.gov/wps/portal/nrcs/detailfull/national/water/manage/hydrology/?cid=stelp_rdb1043063)

## 200 DRAINAGE CRITERIA

### 201 Introduction

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

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

[http://dot.state.nm.us/content/dam/nmdot/Plans\\_Specs\\_Estimates/2014\\_Specs\\_For\\_Highway\\_And\\_Bridge\\_Construction.pdf](http://dot.state.nm.us/content/dam/nmdot/Plans_Specs_Estimates/2014_Specs_For_Highway_And_Bridge_Construction.pdf)

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

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

[http://dot.state.nm.us/content/dam/nmdot/Plans\\_Specs\\_Estimates/Design\\_Directives/2016/IDD-2016-11\\_\(Design\\_Exception\\_Variance\\_and\\_ADA\\_Design\\_Variance.pdf](http://dot.state.nm.us/content/dam/nmdot/Plans_Specs_Estimates/Design_Directives/2016/IDD-2016-11_(Design_Exception_Variance_and_ADA_Design_Variance.pdf)

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

### 202 Drainage Principles, Guidelines and Definitions

#### Principles and Guidelines

Drainage system design must consider the following principles and guidelines:

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

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

### **Definitions**

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

## **203 Storm Duration and Frequency Criteria**

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

### **Minor Arterials, Collectors and Local Roads**

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

### **Interstate Highways and Principal Arterials**

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



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

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

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

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

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

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

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

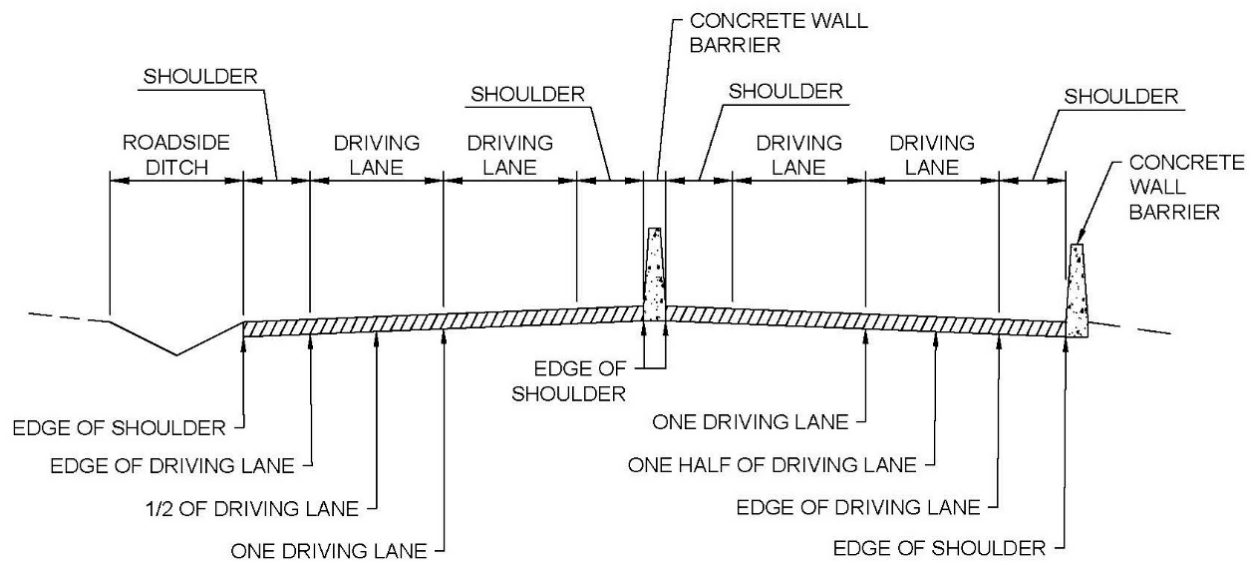
a - Check other flood frequencies as appropriate for greater scour depths

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

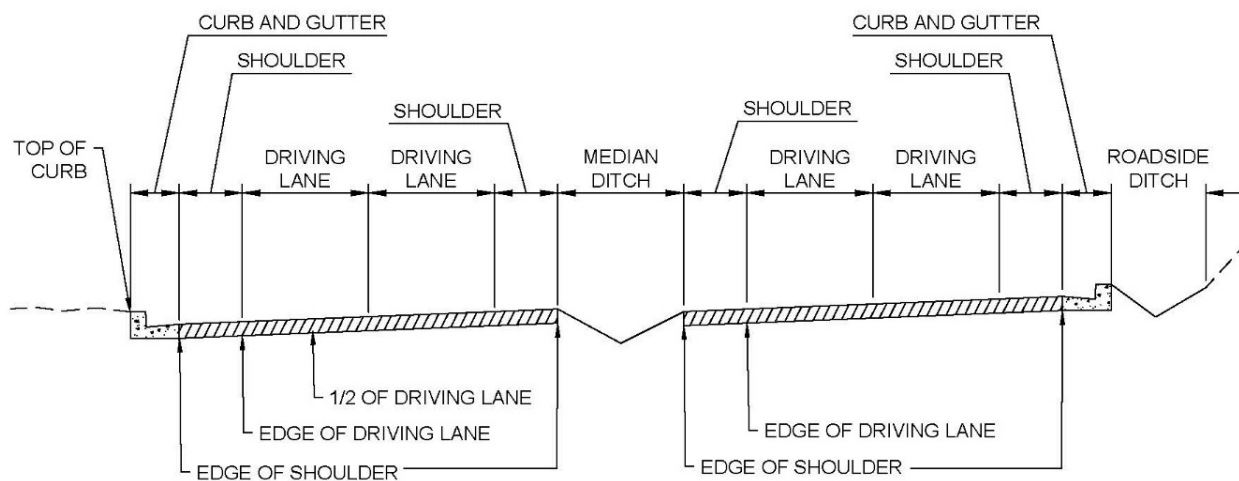
c - Concrete Wall Barrier criteria also apply to Concrete Barrier Railing and vertical barriers greater than 8" height

## 204 Hydraulic Criteria for Drainage Structures

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



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



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

**Table 204-1 Design Flood Hydraulic Criteria for Drainage Structures**

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

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

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

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

**Table 204-2 Check Flood Hydraulic Criteria for Drainage Structures**

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

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

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

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

## Peak Discharge Computation at Culverts and Bridges

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

## Bridge Scour

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

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

## Concrete Channels

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

## Channel Freeboard

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

[http://library.amlegal.com/nxt/gateway.dll/NewMexico/albuqdpdpm/albuquerque/newmexicocodevelopmentprocessma?f=templates\\$fn=default.htm\\$3.0\\$vid=amlegal:albuquerque\\_nm\\_mc\\$anc=JD\\_DPM](http://library.amlegal.com/nxt/gateway.dll/NewMexico/albuqdpdpm/albuquerque/newmexicocodevelopmentprocessma?f=templates$fn=default.htm$3.0$vid=amlegal:albuquerque_nm_mc$anc=JD_DPM)

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

### Trapezoidal Channels

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

where:

$V$  = velocity, ft/s

$d$  = flow depth, ft

$D_c$  = critical depth, ft

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

[http://library.amlegal.com/nxt/gateway.dll/NewMexico/albuqdp/albuquerque/newmexicocodevelopmentprocessma?f=templates\\$fn=default.htm\\$3.0\\$vid=amlegal:albuquerque\\_nm\\_mc\\$anc=JD\\_DPM](http://library.amlegal.com/nxt/gateway.dll/NewMexico/albuqdp/albuquerque/newmexicocodevelopmentprocessma?f=templates$fn=default.htm$3.0$vid=amlegal:albuquerque_nm_mc$anc=JD_DPM)

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

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

### Summary

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

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

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

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

Bridges - Debris	Estimate pier (if present) debris width and depth and account for conveyance loss in the hydraulic and scour analyses. Estimate based on urban or rural location, watershed and watercourse conditions.
Bridges - Sedimentation	Evaluate the structure and mitigate effects with respect to - significant changes to channel velocity, aggradation or degradation, scour, head cutting, and conveyance.
Culverts - Bulking and Debris Factor	Urban and Rural – For clear water calculations apply a 20% factor. For flows determined by regression equations or a USGS Bulletin 17C analysis of stream gage data, no additional bulking factor should be applied. Refer to <b>Section 402.11</b> for bulking factors.
Pipe (storm drain and culvert) - Material and Wall Thickness	Select wall thickness based on Corrosion Resistance Number – <b>Section 800</b> (NMDOT Spec. 570.2.3.1) and cover height.
Curb & Median Drop Inlet Grates - Clogging Factor	Inlet Grates on Grade - assume a 25% minimum grate clogging factor.
	Inlet Grates in Sag - assume a 50% clogging factor. Inlet grates in sag will require a minimum of one flanking inlet (an inlet near to and upstream of the sag inlet).
	Median Inlet Grates - assume a 50% grate clogging factor.
Concrete Wall Barrier - Clogging Factor (drainage slots)	Assume a 50% clogging factor due to minimal opening size. Wall barrier in sag will require a minimum of one flanking inlet (an inlet near to and upstream of the sag inlet).
Detour Drainage Structures	<p>Shall be designed to convey the 2-year flood as a minimum. However, some circumstances listed here may require larger flood events. Consult with the Drainage Design Bureau.</p> <ul style="list-style-type: none"> <li>- A long construction period (longer than 9 months)</li> <li>- Safety concerns due to roadway overtopping</li> <li>- Environmental concerns and potential for environmental damage</li> <li>- Potential for property damage and related economic consequences</li> </ul>
Waterstops/turnout humps	All turnouts to NMDOT ROW must be constructed with waterstops (humps), matching the height of the existing curb and gutter or having a minimum height of 4" if curb and gutter is not present. If full-height waterstops are not geometrically feasible, consult with the NMDOT Drainage Engineer for alternative configurations. Turnouts or driveways may discharge runoff to the NMDOT ROW provided that the contributing runoff is included in design calculations for the roadway and storm drain system. If NMDOT will discharge roadway runoff to private property, drop inlets, or other methods to reduce the runoff down the turnout should be installed immediately upstream of the turnout.
Adjacent Properties	Consider and avoid detrimental effects - flooding, sedimentation, or erosion - on adjacent property.
Irrigation Ditches	Ensure that the proposed design does not adversely affect irrigation ditches.
Channel or Stream Deterioration and Modifications	Evaluate the proposed structure and mitigate effects with respect to channel velocity, aggradation or degradation, scour, head cutting, and conveyance. Make allowance in channels for conveyance loss due to debris, vegetation and sedimentation.
Regulatory Requirements	Evaluate proposed structure/project and ensure that any channel or stream modifications meet the requirements of the U.S. Army Corps of Engineers, the NM Environment Department, U.S. Fish & Wildlife Service, U.S. EPA, FEMA, and other agencies.

## 206 Design Criteria for Storm Drains and Culverts

**Table 206-1 Design Criteria for Storm Drains and Culverts**

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



## 207 Design Criteria for Detention and Retention Ponds

### Jurisdictional Dams and Non-Jurisdictional Dams

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

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

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

### DETENTION AND RETENTION PONDS

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

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

### Pond Design Criteria (Detention and Retention Ponds)

- Sediment Bulking
  - Computed/simulated clear water hydrographs shall be increased by a sediment bulking factor to account for sediment volume within the water volume
  - Bulking factors will typically range from about 1.0 for a 100 percent urban impervious watershed including hard lined conveyance systems (no exposed soil or landscape areas), to a maximum factor of about 1.25 for a rural undeveloped or damaged watershed. **Section 402.11** presents more information and items to consider regarding determination of sediment bulking factors. **Figure 402-19** presents a range of bulking factors for various return period floods.
  - Obtain approval from the Drainage Design Bureau regarding sediment bulking factor assumptions and computed or selected values applied for pond analysis and design
  - Sediment bulking factors shall be applied in addition to the dead storage volume requirement (see **Table 207-1**). Dead storage design provides for additional design storage volume due to sediment deposition, and accounts for either lack of maintenance (sediment removal to maintain the design storage volume) and/or storage volume loss from frequent floods/sediment deposition between maintenance activities.
  - A maintenance schedule may be warranted, depending on accumulated sediment loads (volumes) and available storage space.
- Principal Spillways
  - Minimum outfall conduit diameter shall be 24 inches
  - Outfall conduit design maximum pressure and allowable joint pressure capacity shall be documented
  - Detention Ponds - spillways shall provide for floatable debris retention
  - Retention Ponds – do not have principal spillways
  - Outfall design shall include erosion/scour and energy dissipation structures

- Outfall conduit shall be oriented in the direction of, and outfall to, the natural watercourse
- Include water quality features as appropriate (e.g., trash racks, perforated riser)
- Outfall conduit through an embankment shall have piping protection
- Emergency Spillways
  - Detention Ponds - shall have an emergency spillway with sufficient capacity to pass the Check Flood without overtopping the embankment
  - Retention Ponds - shall have an emergency spillway with sufficient capacity to pass the Check Flood without overtopping the embankment
  - Spillways shall be directed to the natural watercourse
  - Spillway approach, crest, chute, and toe design shall include erosion/scour and energy dissipation structures
- Pond Embankments
  - Maximum pond side slopes and embankment slopes shall be 1 vertical to 3 horizontal (1V:3H) if an approved "seeded gravel mulch" is applied. Otherwise maximum slopes of 1V:6H or flatter are required to minimize rill/gulley erosion.
  - Maximum embankment height is defined as the vertical distance from the lowest point on the downstream embankment toe to the lowest point on the embankment crest as defined by the NM Office of the State Engineer Dam Safety Bureau (NMOSE, December 2010). This definition shall also apply to NMDOT pond embankments.
  - Embankment crest width shall be:
    - 12 feet minimum width if a maintenance access road on crest is required by NMDOT Drainage Design Bureau
    - Crest width may be less than 12 feet if a maintenance access road is not required, but not less than 3 feet. Crest widths less than 12 feet must be approved by the NMDOT Drainage Design Bureau
    - Crest width shall be designed in conjunction with embankment design and documented by geotechnical specifications and recommendations
    - Crest width requirements do not apply to retention ponds excavated below ground on all sides
- Maintenance Access Road to Pond Bottom
  - Required – maximum slope allowed shall be 1V:8H (12.5%)
  - Road surface shall be designed to ensure access and may include crushed gravel, base course, or other approved materials and design as required
  - Road should lead to principal spillway structure if possible
- Miscellaneous Pond Requirements
  - An approved permanent sediment stage indication marker (marked in 1 ft increments) shall be installed in all ponds and shall be located near the embankment toe and near the principal spillway
  - Grade detention pond bottoms to drain at minimum 0.5% slope towards the principal spillway. Retention pond bottoms may have 0% slope.

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

All designs must be approved by the NMDOT.

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

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

**Table 207-1 Criteria for Detention and Retention Ponds**

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

### Stormwater Quality MS4 Requirements

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

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

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

#### Pond Design Criteria

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

### Controlling Runoff from New Development and Re-development

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

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

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

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

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

\*Use 0.615 inches per the following paragraph.

Notes related to **Table 207-2** and information for the Albuquerque area follow.

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

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

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

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

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

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

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

## 208 References

AASHTO, 2001, “A Policy on Geometric Design of Highways and Streets, Fourth Edition”.

[http://nacto.org/docs/usdg/geometric\\_design\\_highways\\_and\\_streets\\_aashto.pdf](http://nacto.org/docs/usdg/geometric_design_highways_and_streets_aashto.pdf)

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

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

City of Albuquerque, October 2008, “Development Process Manual, Chapter 22, Drainage, Flood Control and Erosion Control”.

<http://library.amlegal.com/nxt/gateway.dll/New>

[Mexico/albuqdp/albuquerque/newmexicocodevelopmentprocessma?f=templates\\$fn=default.htm\\$3.0\\$vid=amlegal:albuquerque\\_nm\\_mc\\$anc=JD\\_DPM](http://library.amlegal.com/nxt/gateway.dll/NewMexico/albuqdp/albuquerque/newmexicocodevelopmentprocessma?f=templates$fn=default.htm$3.0$vid=amlegal:albuquerque_nm_mc$anc=JD_DPM)

EPA, Region 6, “Current Internet Download – Region 6 sMS4 General Permit, NMR04000 Stormwater General Permit for Small Municipal Separate Storm Sewer Systems (MS4s)”.

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

EPA, March 2015, Publication Number 832-R-15-009, “Estimating Pre-Development Hydrology for Urbanized Areas in New Mexico”.

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

FHWA, December 1995, “Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation’s Bridges”.

<https://www.fhwa.dot.gov/bridge/mtguide.pdf>

NMDOT Website, “Standard Specifications for Highway and Bridge Construction”.

[http://dot.state.nm.us/content/dam/nmdot/Plans\\_Specs\\_Estimates/2014\\_Specs\\_For\\_Highway\\_And\\_Bridge\\_Construction.pdf](http://dot.state.nm.us/content/dam/nmdot/Plans_Specs_Estimates/2014_Specs_For_Highway_And_Bridge_Construction.pdf)

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

NMDOT Website, “Standard Drawings”.

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

NMDOT, November 8, 2016, “Design Exception, Design Variance & ADA Design Variance Procedures”, Infrastructure Design Directive IDD-2016-11.

[http://dot.state.nm.us/content/dam/nmdot/Plans\\_Specs\\_Estimates/Design\\_Directives/2016/IDD-2016-11\\_\(Design\\_Exception\\_Variance\\_and\\_ADA\\_Design\\_Variance.pdf](http://dot.state.nm.us/content/dam/nmdot/Plans_Specs_Estimates/Design_Directives/2016/IDD-2016-11_(Design_Exception_Variance_and_ADA_Design_Variance.pdf)

NMOSE, December 31, 2010, “Rules and Regulations Governing Dam Design, Construction and Dam Safety”, New Mexico Office of the State Engineer Dam Safety Bureau.

<http://www.ose.state.nm.us/DS/Regs/19-25-12-NMAC-2010.pdf>

# **300 NMDOT DRAINAGE ANALYSES CHECKLISTS, REPORT AND CONSTRUCTION PLAN REQUIREMENTS**

## **301 Introduction**

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

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

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



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

## 302 Supplemental Data Sources

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

### Government Agencies:

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

### Irrigation Districts:

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

### Other Sources:

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

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

### 303 Field Inspection Checklists

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

### 304 Drainage Analysis Requirements

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

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

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

### Project Development and Drainage Tasks

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

### Typical Project Development Schedule and Milestones

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

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

## **305 Drainage Reports and Submittal Format**

### **Preliminary Drainage Report**

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

#### Items Required on the Cover Include:

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

#### Other Items Within the Report Include:

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

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

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

### **Final Drainage Report**

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

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

### **Drainage Report Checklist**

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

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

### **Drainage Reports Submittal Format**

The NMDOT Drainage Design Bureau will require the following items:

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

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

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

### **Municipal Separate Storm Sewer Systems (MS4s)**

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

## **306 Temporary Erosion and Sediment Control Plans**

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

NMDOT, August 2012, “National Pollutant Discharge Elimination System Manual - Stormwater Management Guidelines for Construction and Industrial Activities, - Revision 2”.

<http://dot.state.nm.us/content/dam/nmdot/Infrastructure/NPDESM.pdf>

## **307 Construction Plan Drainage Requirements**

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

Bridges - Annotate the plans with the following information:

- DA = drainage area in acres or square miles
- Q<sub>x</sub> = design peak flow rate in cfs = Design Flood flow; with “x” representing the Design Flood recurrence interval
- HW<sub>x</sub> = headwater in feet; listed as either depth from the upstream bridge invert to water surface at the upstream bridge deck, or the elevation of water surface; with “x” representing the recurrence interval

Through Culverts - Annotate the plans with the following information:

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

Drop Inlets - Annotate the plans with the following information:

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

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

- j.  $V_x$  = velocity in ft/s for the Design Flood flow; with “x” representing the Design Flood recurrence interval
- k.  $Q_x$  = Design peak flow rate in cfs = Design Flood flow; with “x” representing the Design Flood recurrence interval
- l.  $HGL_x$  = hydraulic grade line shown in profile; with “x” representing the recurrence interval

## 308 References

NMDOT, August 2012, “National Pollutant Discharge Elimination System Manual - Stormwater Management Guidelines for Construction and Industrial Activities, - Revision 2”.

<http://dot.state.nm.us/content/dam/nmdot/Infrastructure/NPDESM.pdf>

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

<https://www.epa.gov/tmdl>

## 400 HYDROLOGY

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

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

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

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

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

### 401 NMDOT Approach to Hydrologic Analyses

The NMDOT is tasked with providing transportation facilities that are reasonably safe for the public within the realities of budget and widely varying soils, topography and climate conditions. A safe roadway environment includes proper roadway drainage, and properly designed drainage structures. The NMDOT’s goal is to design and construct roadways and drainage structures that meet minimum design standards and do so within the realities of budgetary

constraints. **Section 200** of this Manual presents the current minimum drainage criteria that shall be applied for NMDOT projects.

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

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

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

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

NRCS, "Part 630 Hydrology, National Engineering Handbook". Note that various Chapters have different dates.

<https://www.nrcs.usda.gov/wps/portal/nrcs/detailfull/national/water/manage/hydrology/?cid=stelprdb1043063>

NRCS, June 1986, "TR-55, Urban Hydrology for Small Watersheds".

[https://www.nrcs.usda.gov/Internet/FSE\\_DOCUMENTS/stelprdb1044171.pdf](https://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/stelprdb1044171.pdf)

SCS, February 1985, "Peak Rates of Discharge for Small Watersheds, Chapter 2, Engineering Field Manual for Conservation Practices".

(Not available on the NRCS website or the internet)

The most pertinent sections from these references have been excerpted directly for ease of use. If further explanation or background information is required, the engineer is directed to the NRCS website where the complete National Engineering Handbook and TR-55 may be found.

**APPENDIX 5** contains a copy of the February 1985 document as it is not available on the NRCS website or the internet.

## Organization of the Hydrology Section of this Manual

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



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

### **401.1 Purposes Served by Hydrologic Analyses**

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

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

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

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

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

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

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

- Rational Formula Method – This Method is appropriate for simple watersheds of 160 acres or less and where only a peak runoff rate is needed, however is not to be used for runoff volume computations. **Section 403** describes the use of the Rational Formula Method.
- NRCS Simplified Peak Discharge Method – This Method is based on the SCS, February 1985 document titled, "Peak Rates of Discharge for Small Watersheds, Chapter 2, Engineering Field Manual for Conservation Practices", and in watersheds with areas up to 10 square miles. Refer to **Section 404.2** for limitations that must be observed with this Method. **Section 404** describes the NRCS Simplified Peak Discharge Method.

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

## 401.2 Selection of Hydrologic Method

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

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

substituted for the Simplified Peak Discharge Method and the Simplified Peak Discharge Method may be substituted for the Rational Formula Method.

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

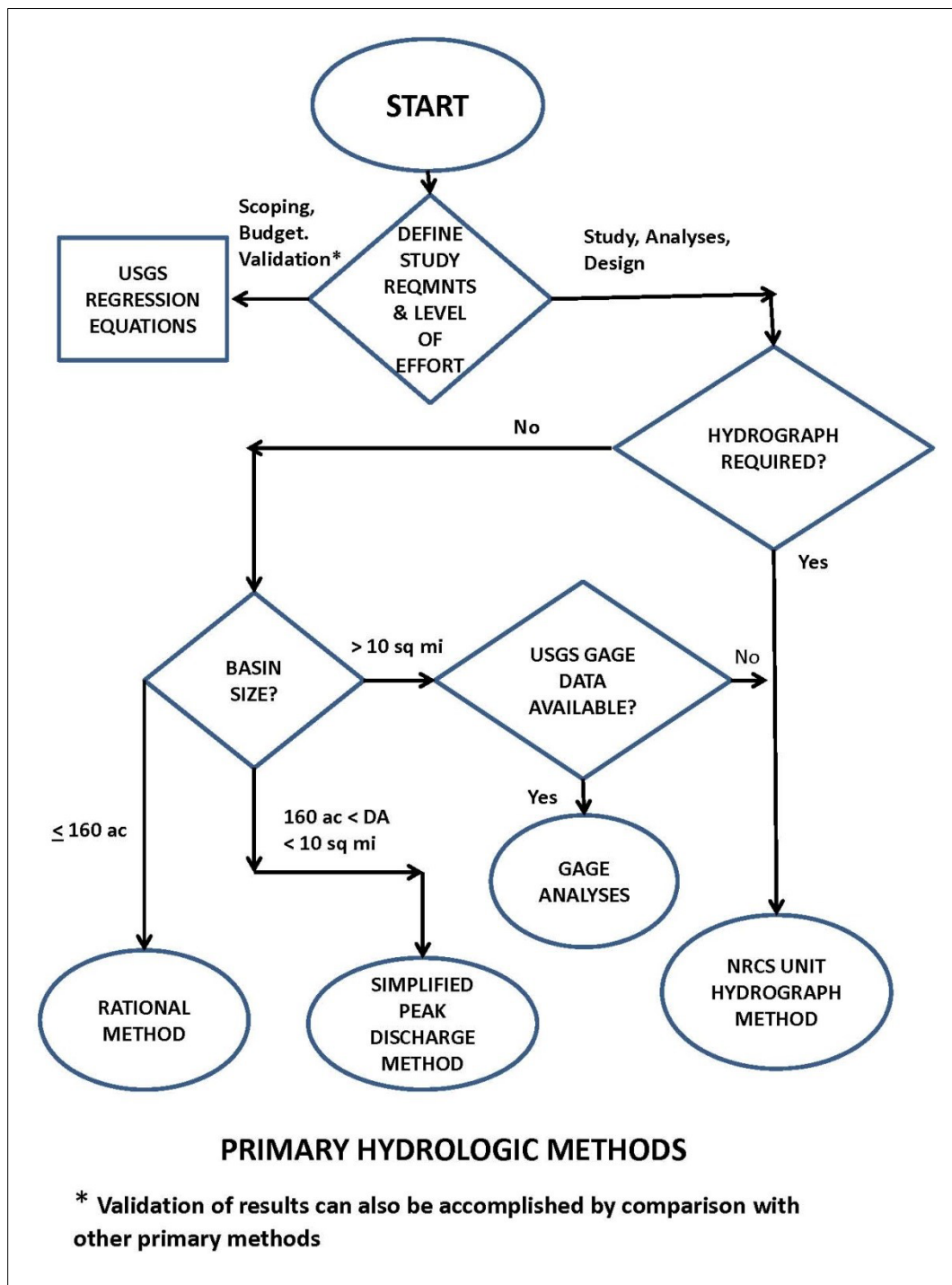
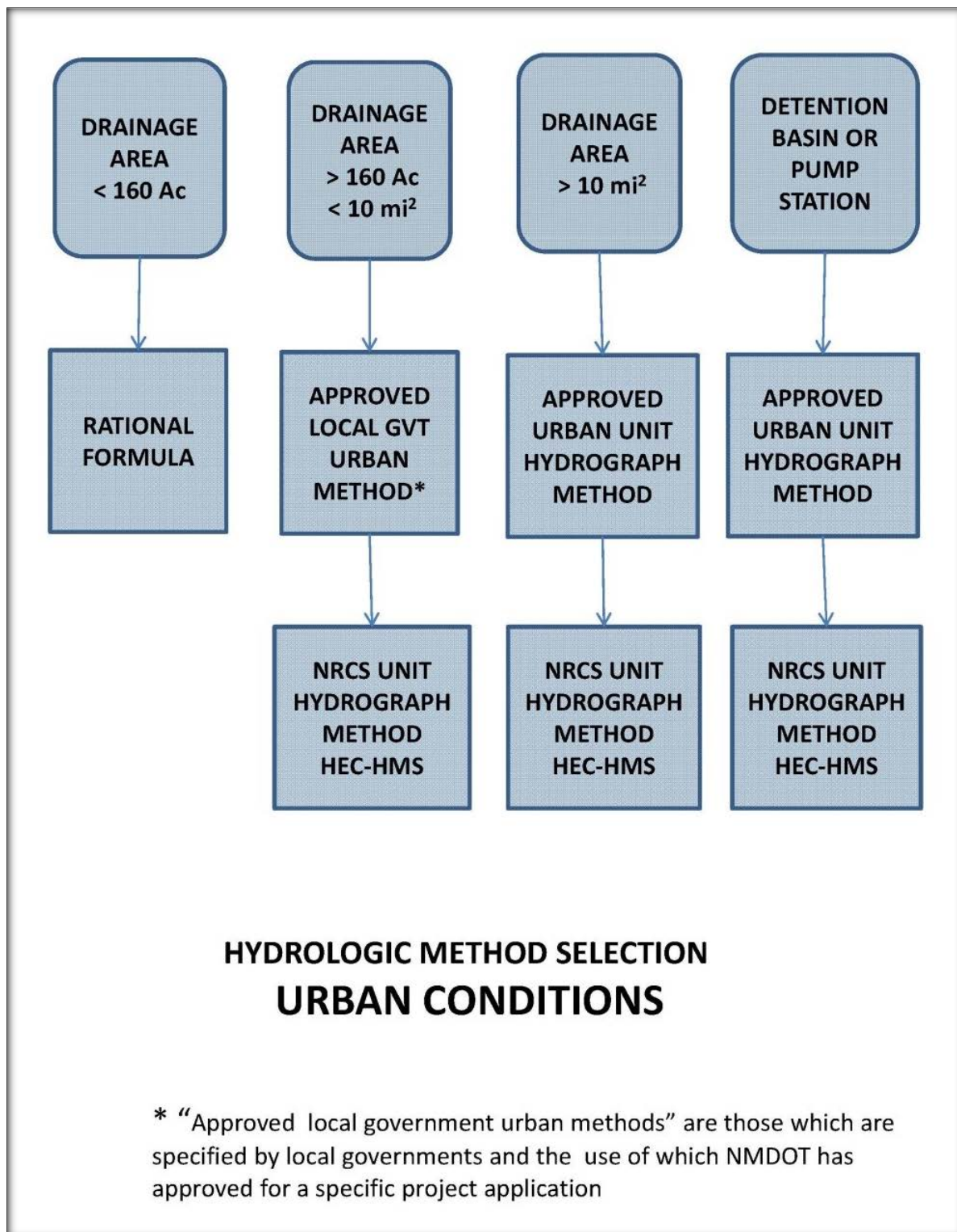


Figure 401-1 Hydrologic Method Selection – Rural Watersheds



**Figure 401-2 Hydrologic Method Selection – Urban Conditions**

### 401.3 Basic Requirements for Drainage Studies

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

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

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

#### Typical Project Development Schedule and Milestones

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

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

### 401.4 Drainage Field Inspection and Drainage Reports

#### **Drainage Field Inspection**

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

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

### **Preliminary and Final Drainage Reports**

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

## **401.5 References**

NRCS, June 1986, "TR-55, Urban Hydrology for Small Watersheds".

[https://www.nrcs.usda.gov/Internet/FSE\\_DOCUMENTS/stelprdb1044171.pdf](https://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/stelprdb1044171.pdf)

NRCS, "Part 630 Hydrology, National Engineering Handbook". Note that various Chapters have different dates.

<https://www.nrcs.usda.gov/wps/portal/nrcs/detailfull/national/water/manage/hydrology/?cid=stelprdb1043063>

Soil Conservation Service (NRCS), 1973, Rev. ed. February 1985, Rev. ed. 2014, "Peak Rates of Discharge for Small Watersheds, Chapter 2, Engineering Field Manual for Conservation Practices".

(Not available on the NRCS website or the internet, **APPENDIX 5** contains a copy)

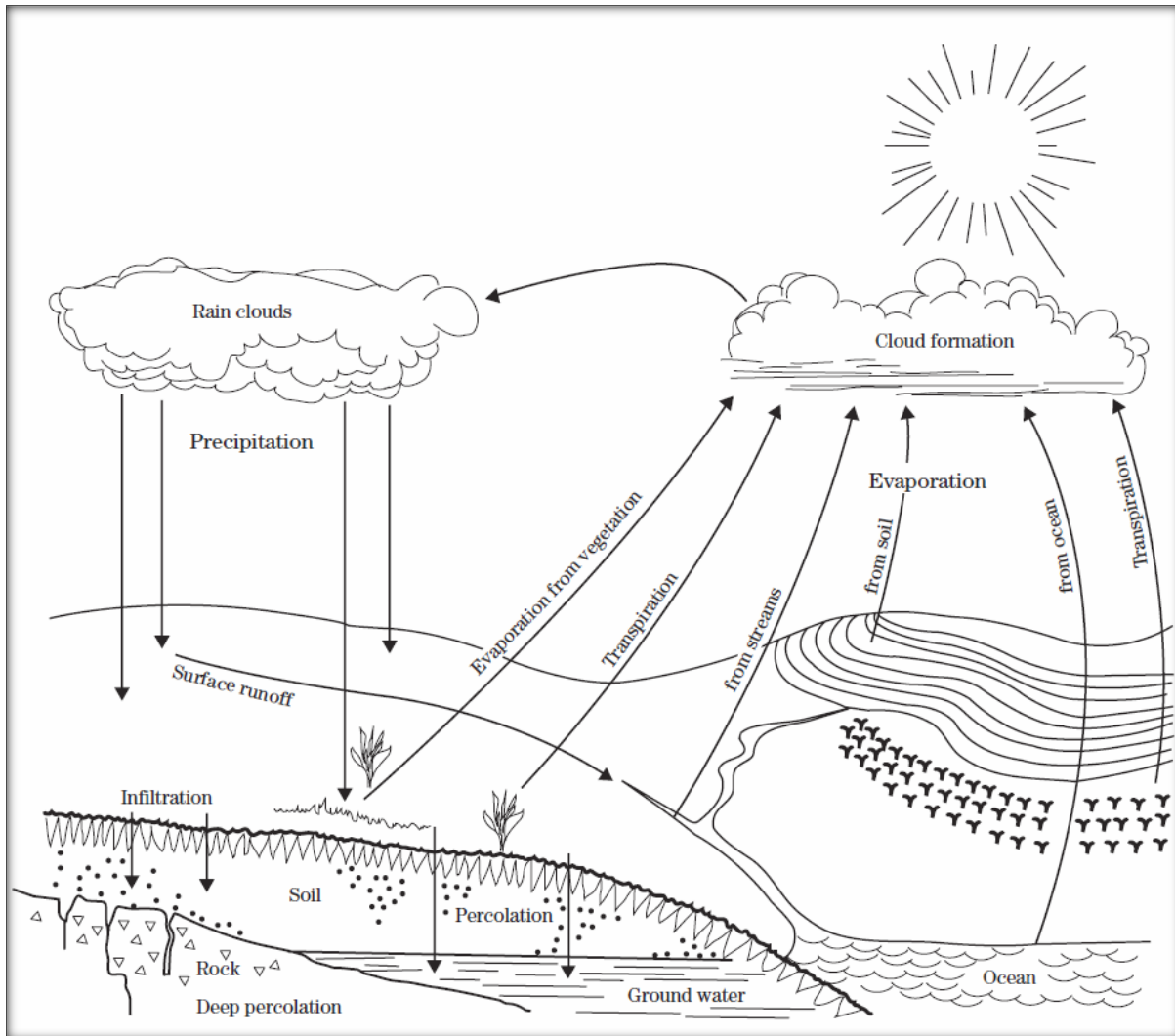
U.S. Army Corps of Engineers Hydrologic Modeling System HEC-HMS, 2015.

<http://www.hec.usace.army.mil/software/hec-hms/>

## 402 General Data Requirements for Hydrologic Analyses

To properly prepare hydrologic analyses, it is fundamental to have a solid grasp of the major physical processes, especially, between precipitation and the earth upon which it falls.

**Figure 402-1** depicts the hydrologic cycle in schematic form illustrating the processes and interactions.



Source: NRCS, 1997, "Part 630 Hydrology, National Engineering Handbook, Chapter 1 Introduction", Cover Page.

<http://www.wcc.nrcs.usda.gov/ftpref/wntsc/H&H/NEHhydrology/ch1.pdf>

**Figure 402-1 Hydrologic Cycle**

Hydrologic analyses are generally data intensive. Unlike structural and pavement design with known loads, the design discharges are unknown, and must be determined for each design project for each component within a project. No two drainage structures share exactly the same



circumstances (drainage area, shape, slope, soils, land use, rainfall, and design criteria), the specifics drive the design analysis.

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

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

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

#### **402.1 Record Drawings and Planned Improvements Information**

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

#### **402.2 Basin and Subbasin Delineation**

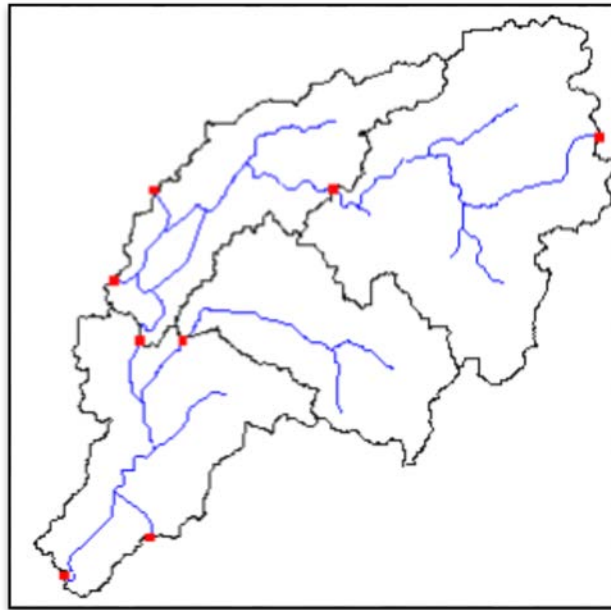
Regardless of the hydrologic analysis selected, the drainage basin area is always required. Basic to all hydrologic methods is the assumption that the basin or subbasin can be reasonably characterized by one set of hydrologic parameters (soils, slope, rainfall, vegetative cover, and land use). The further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

Good “rules of thumb” to follow regarding basin and subbasin sizing are that the length of a basin or subbasin should not exceed 4 times its width and that no subbasin should be more than 10 times larger than the smallest subbasin (NRCS, 2007, “Part 630 Hydrology, National Engineering Handbook, Chapter 16 Hydrographs”).

<http://www.wcc.nrcs.usda.gov/ftpref/wntsc/H&H/NEHhydrology/ch16.pdf>

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

size subbasins. Experience will lead to confidence in knowing how to delineate and size subbasins correctly. **Figure 402-2** is an example of the subbasin delineation process.



**Figure 402-2 Basin Delineation**

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

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

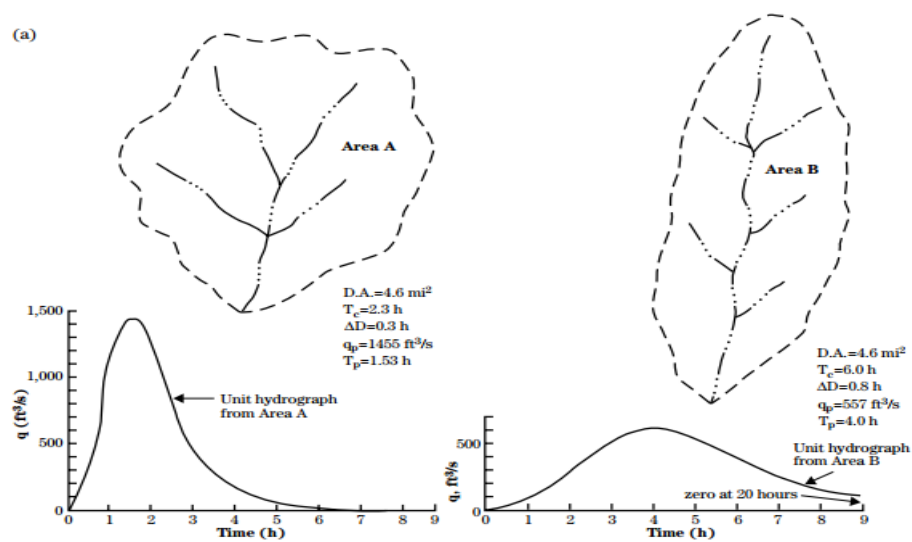
The total basin drainage area can be measured after the drainage basin has been defined. USGS maps are now available in digital format so that this measurement can be made with a GIS tool. A simple guideline should be employed to crosscheck the total drainage area by multiplying the average watershed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

Each drainage basin should be qualitatively assessed by the following:

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

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

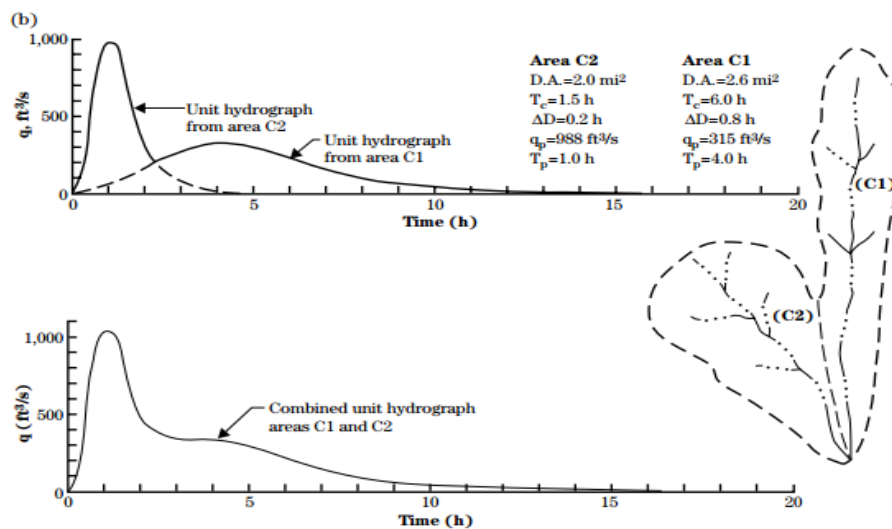
An additional consideration when delineating basins is the recognition of the effect that the basin shape can have on the shape (and peak rate) of the resulting hydrograph. **Figure 402-3** and **Figure 402-4** show the effects on the shape of the resultant hydrograph from different shaped drainage basins.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



Source: NRCS, 2007, "Part 630 Hydrology National Engineering Handbook, Chapter 16 Hydrographs", Figure 16-2(a), p. 16-5.

<http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=17755.wba>

**Figure 402-3 Basin Shape Effects on Hydrograph Shape**



Source: NRCS, 2007, "Part 630 Hydrology National Engineering Handbook, Chapter 16 Hydrographs", Figure 16-2(b), p. 16-6.

<http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=17755.wba>

**Figure 402-4 Combined Basin Effects on Shape of Hydrograph**

### 402.3 Rainfall Volume and Temporal Distribution Data

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

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

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

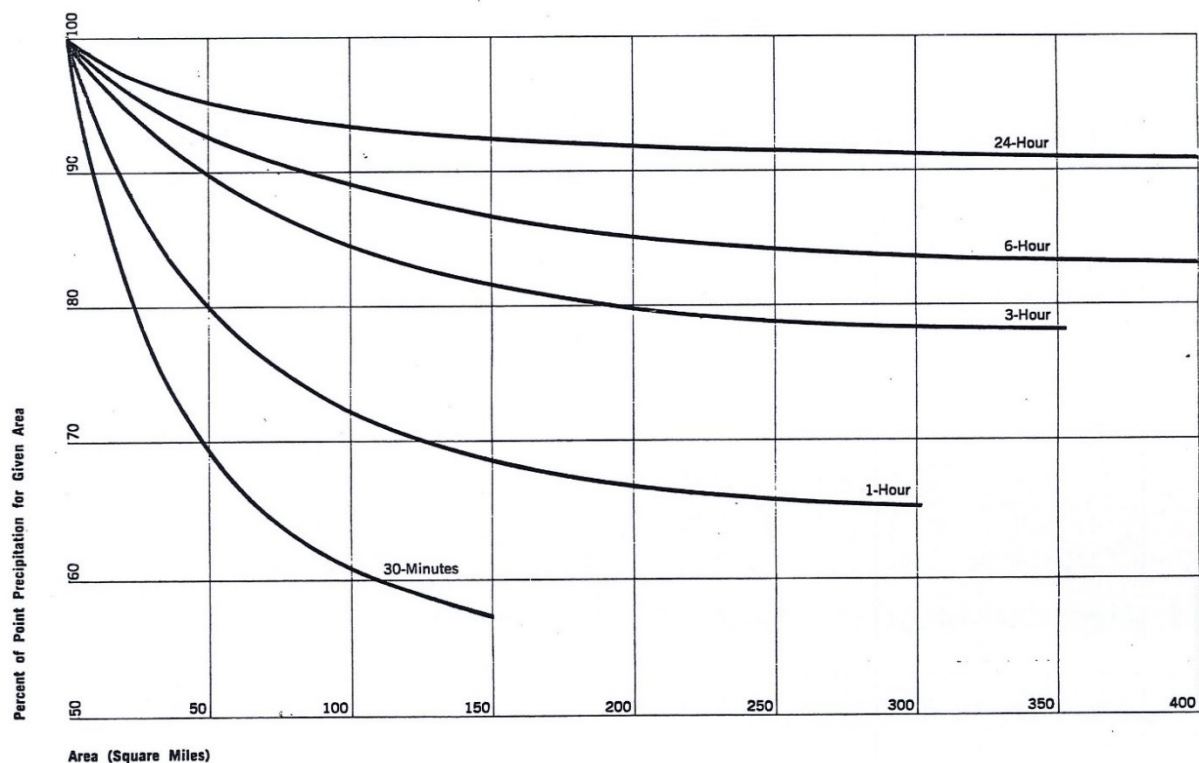
With the advent of digital rainfall data from NOAA Atlas 14 (2011), rainfall data acquisition is both simpler and more accurate than in the past when only large-scale paper copies of rainfall atlases were available (NOAA Atlas 2, 1973). The NOAA Atlas 14 rainfall data sets are more extensive and more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

[http://www.nws.noaa.gov/oh/hdsc/PF\\_documents/Atlas14\\_Volume1.pdf](http://www.nws.noaa.gov/oh/hdsc/PF_documents/Atlas14_Volume1.pdf).

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

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

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



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

**Figure 402-5 Area Reduction Factors for New Mexico**

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

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

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

## **402.4 Soils Data**

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

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

### **Group A**

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

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

### **Group B**

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

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



of all soil layers within 100 centimeters (40 inches) of the surface exceeds 4.0 micrometers per second (0.57 inches per hour) but is less than 10.0 micrometers per second (1.42 inches per hour).

### **Group C**

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

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

### **Group D**

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

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

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

<https://websoilsurvey.sc.egov.usda.gov/App/HomePage.htm>



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

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

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

NRCS, 2009, “Part 630 Hydrology, National Engineering Handbook, Chapter 7 Hydrologic Soils Groups”.

<http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=22526.wba>

## **402.5 Hydrologic Soil-Cover Complexes**

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

<http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=17758.wba>

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

### **402.5.1 Vegetation Effects**

Vegetation affects runoff as described here:

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

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

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

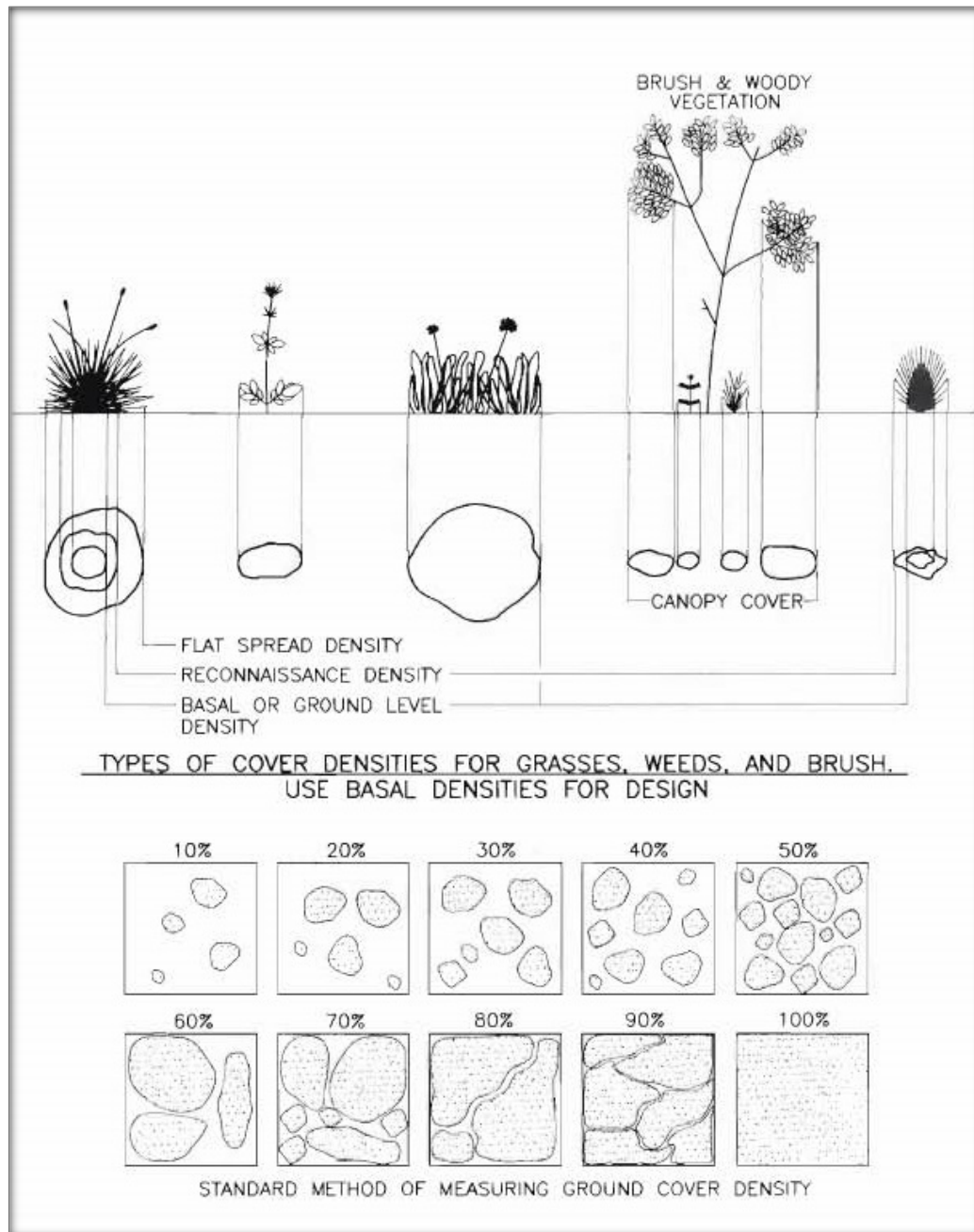
**Table 402-1 Vegetative Cover Classes – Grassland**

Source: NRCS, 2002, Part 630 Hydrology, National Engineering Handbook, Chapter 8 Land Use and Treatment Classes, Table 8-1, p. 8-3

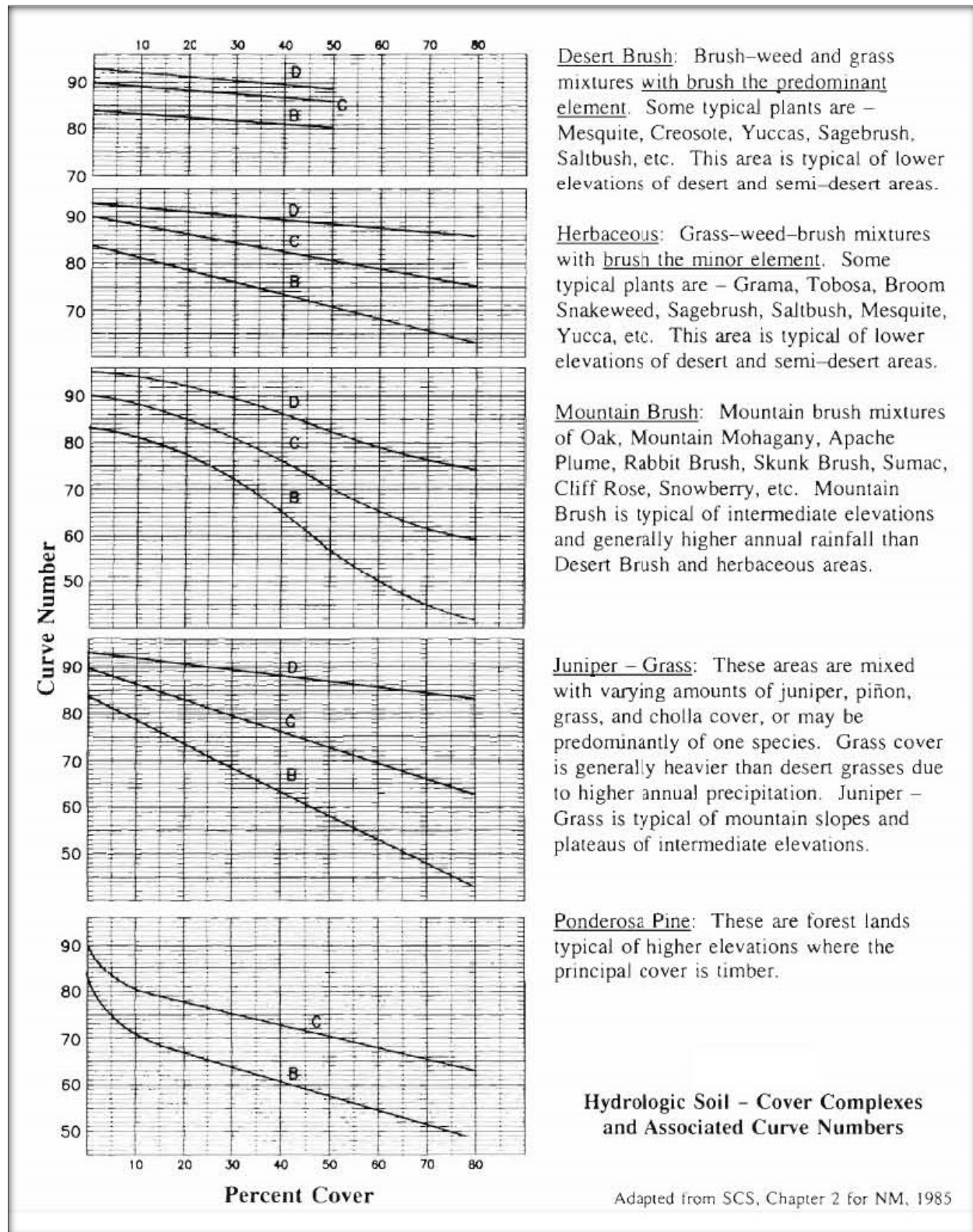
<https://directives.sc.egov.usda.gov/viewerFS.aspx?hid=21422>

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

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



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



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

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

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

[http://www.nrcs.usda.gov/Internet/FSE\\_DOCUMENTS/stelprdb1044175.pdf](http://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/stelprdb1044175.pdf)

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

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

## **402.6 Runoff Curve Number**

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

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

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

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

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

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

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

The following excerpts from Chapter 2 of “TR-55, Urban Hydrology for Small Watersheds”, (NRCS, June 1986) provide a relatively complete and clear explanation of the Curve Number, its determination, and its use in hydrologic analyses. A hotlink to the document is provided below.

[https://www.nrcs.usda.gov/Internet/FSE\\_DOCUMENTS/stelprdb1044171.pdf](https://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/stelprdb1044171.pdf)

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

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

(NRCS, June 1986, “TR-55, Urban Hydrology for Small Watersheds”, Eq. 2-3, p. 2-1)

[https://www.nrcs.usda.gov/Internet/FSE\\_DOCUMENTS/stelprdb1044171.pdf](https://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/stelprdb1044171.pdf)

where:

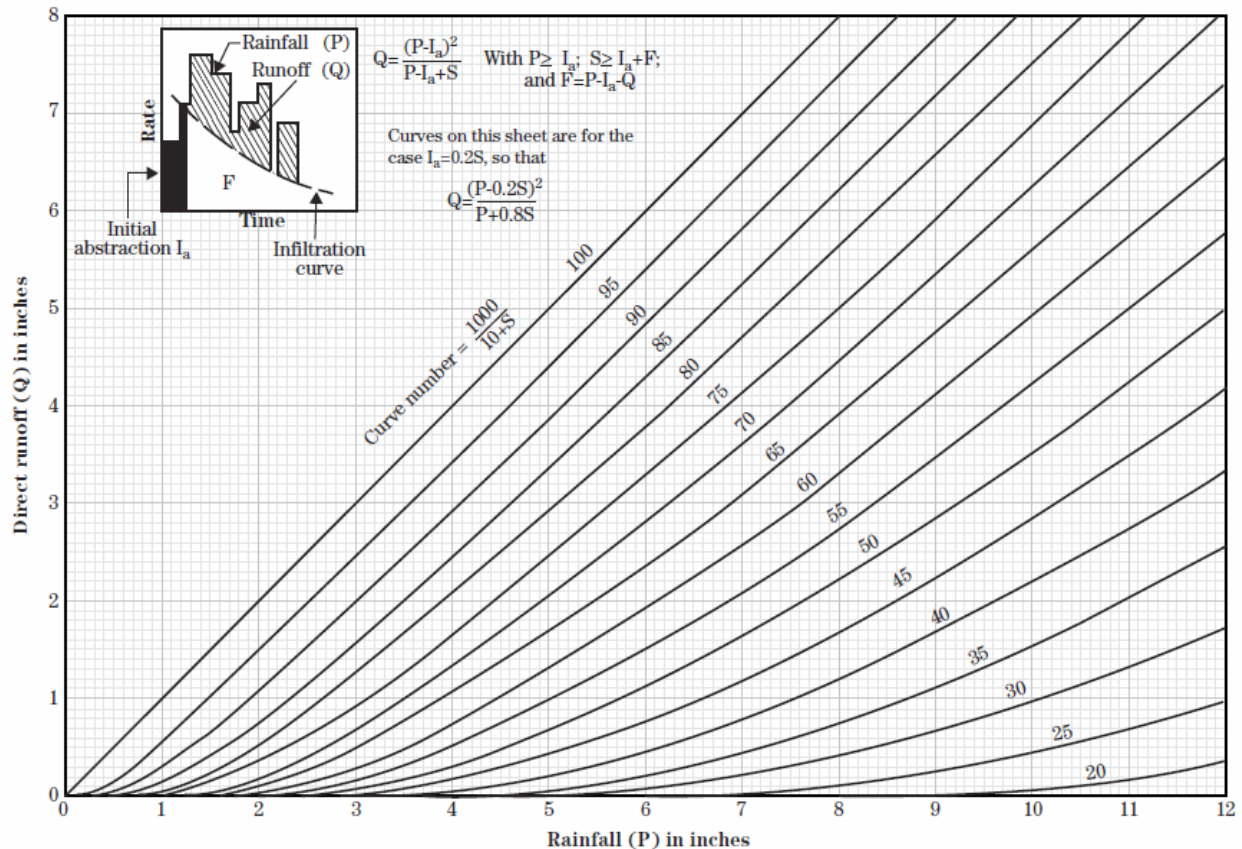
Q	=	runoff, inches
P	=	rainfall, inches
S	=	potential maximum soil moisture retention after runoff begins
CN	=	Runoff Curve Number

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

(NRCS, June 1986, “TR-55, Urban Hydrology for Small Watersheds”, Eq. 2-4, p 2.1)

[https://www.nrcs.usda.gov/Internet/FSE\\_DOCUMENTS/stelprdb1044171.pdf](https://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/stelprdb1044171.pdf)





Source: NRCS, 2004, "Part 630 Hydrology, National Engineering Handbook, Chapter 10 Estimation of Direct Runoff from Storm Rainfall", Figure 10-2, p. 10-4

<https://directives.sc.egov.usda.gov/17752.wba>

**Figure 402-8 Solution of Runoff Equation**

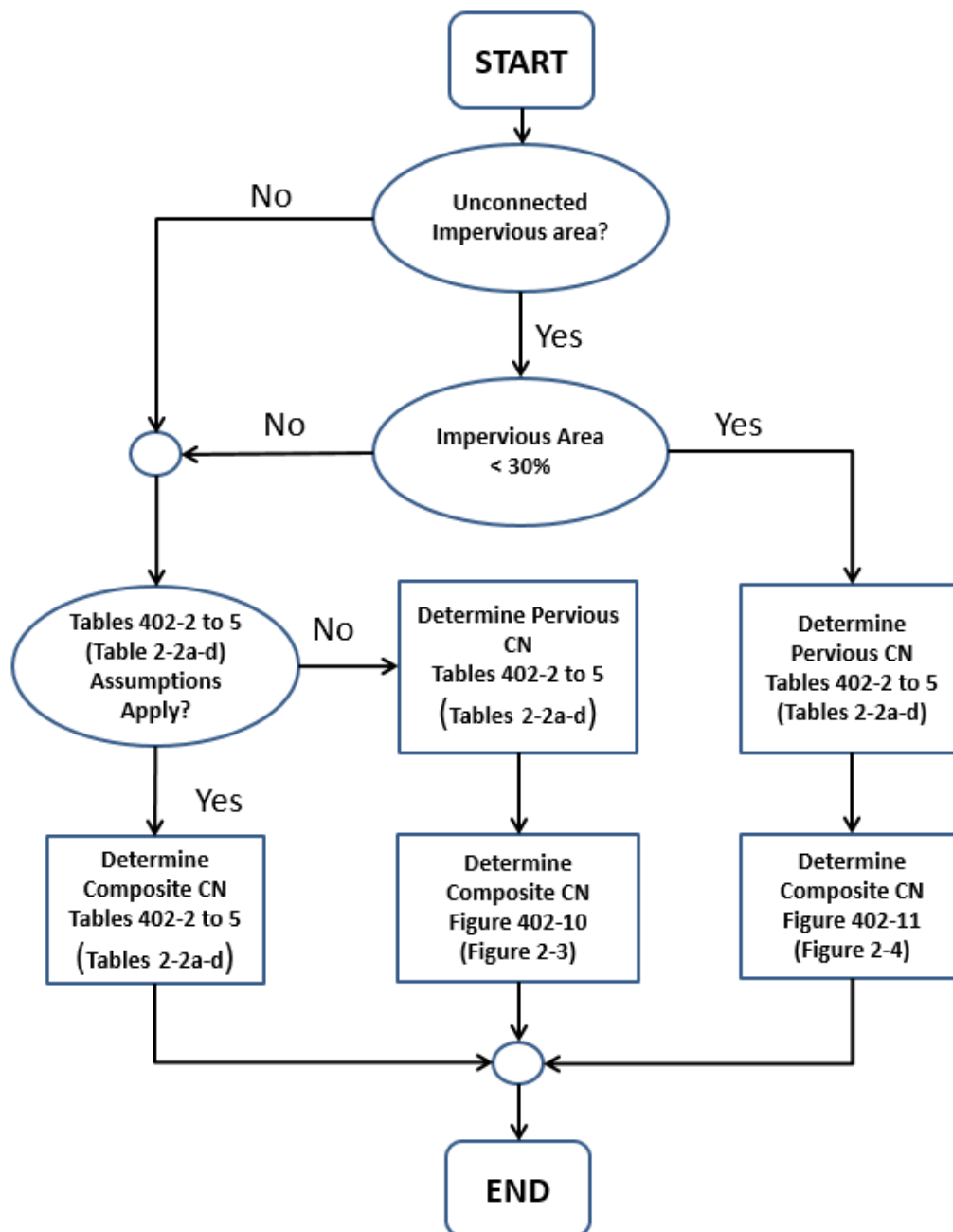
### Storm Duration and Storm Recurrence Interval

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

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

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

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



Source: NRCS, June 1986, "TR-55, Urban Hydrology for Small Watersheds",  
Figure 2-2, p. 2-4.

[https://www.nrcs.usda.gov/Internet/FSE\\_DOCUMENTS/stelprdb1044171.pdf](https://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/stelprdb1044171.pdf)

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



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

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

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

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

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

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

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

**Table 402-2 Runoff Curve Numbers for Urban Areas**

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

**Table 2-2a** Runoff curve numbers for urban areas <sup>1/</sup>

Cover description		Curve numbers for hydrologic soil group			
Cover type and hydrologic condition	Average percent impervious area <sup>2/</sup>	A	B	C	D
<i>Fully developed urban areas (vegetation established)</i>					
Open space (lawns, parks, golf courses, cemeteries, etc.) <sup>3/</sup> :					
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) <sup>4/</sup> .....		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
<i>Developing urban areas</i>					
Newly graded areas (pervious areas only, no vegetation) <sup>5/</sup> .....		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

<sup>1/</sup> Average runoff condition, and  $I_a = 0.2S$ .

<sup>2/</sup> The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions may be computed using figure 2-3 or 2-4.

<sup>3/</sup> CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.

<sup>4/</sup> Composite CN's for natural desert landscaping should be computed using figures 2-3 or 2-4 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.

<sup>5/</sup> Composite CN's to use for the design of temporary measures during grading and construction should be computed using figure 2-3 or 2-4 based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.

**Table 402-3 Runoff Curve Numbers for Cultivated Agricultural Lands**

Source: NRCS, June 1986, "TR-55, Urban Hydrology for Small Watersheds",  
Table 2-2b, p. 2-6.

[https://www.nrcs.usda.gov/Internet/FSE\\_DOCUMENTS/stelprdb1044171.pdf](https://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/stelprdb1044171.pdf)

**Table 2-2b** Runoff curve numbers for cultivated agricultural lands <sup>1/</sup>

Cover description			Curve numbers for hydrologic soil group			
Cover type	Treatment <sup>2/</sup>	Hydrologic condition <sup>3/</sup>	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
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

<sup>1/</sup> Average runoff condition, and  $I_a = 0.2S$

<sup>2/</sup> Crop residue cover applies only if residue is on at least 5% of the surface throughout the year.

<sup>3/</sup> Hydraulic condition is based on combination factors that affect infiltration and runoff, including (a) density and canopy of vegetative areas, (b) amount of year-round cover, (c) amount of grass or close-seeded legumes, (d) percent of residue cover on the land surface (good  $\geq 20\%$ ), 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 402-4 Runoff Curve Numbers for Other Agricultural Lands**

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

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Table 2-2c Runoff curve numbers for other agricultural lands <sup>1/</sup>

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

<sup>1/</sup> Average runoff condition, and  $I_a = 0.2S$ .

<sup>2/</sup> *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.

<sup>3/</sup> *Poor*: <50% ground cover.  
*Fair*: 50 to 75% ground cover.  
*Good*: >75% ground cover.

<sup>4/</sup> Actual curve number is less than 30; use CN = 30 for runoff computations.

<sup>5/</sup> 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.

<sup>6/</sup> *Poor*: Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.  
*Fair*: Woods are grazed but not burned, and some forest litter covers the soil.  
*Good*: Woods are protected from grazing, and litter and brush adequately cover the soil.

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

Source: NRCS, June 1986, "TR-55, Urban Hydrology for Small Watersheds TR-55",  
Table 2-2d, p. 2-8.

[https://www.nrcs.usda.gov/Internet/FSE\\_DOCUMENTS/stelprdb1044171.pdf](https://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/stelprdb1044171.pdf)

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**Table 2-2d** Runoff curve numbers for arid and semiarid rangelands <sup>1/</sup>

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

<sup>1/</sup> Average runoff condition, and  $I_a = 0.2S$ . For range in humid regions, use table 2-2c.

<sup>2/</sup> Poor: <30% ground cover (litter, grass, and brush overstory).

Fair: 30 to 70% ground cover.

Good: > 70% ground cover.

<sup>3/</sup> Curve numbers for group A have been developed only for desert shrub.

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

### Connected Impervious Areas

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

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

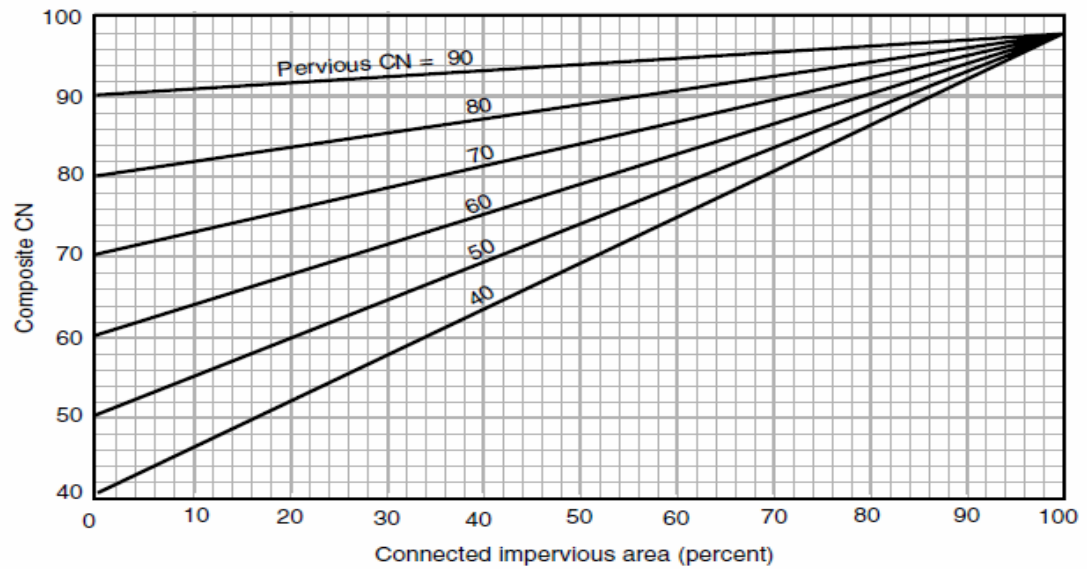
If not all of the impervious area is directly connected to the drainage system, and the impervious area percentages or the pervious land use assumptions in **Table 402-2** are not applicable, use **Figure 402-10** to compute a composite CN.

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

### Unconnected Impervious Areas

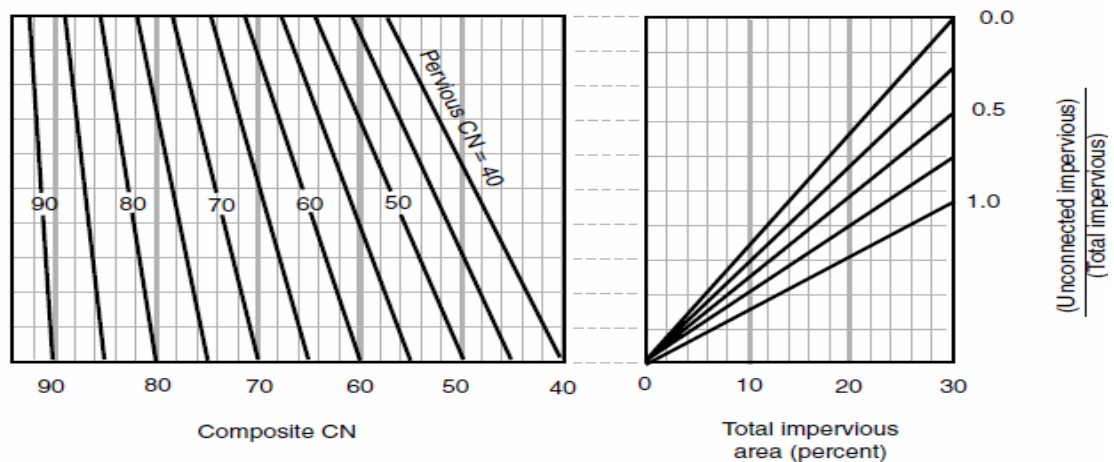
Runoff from unconnected (disconnected) impervious areas is that which spreads over a pervious area as sheet flow. To determine CN when all or part of the impervious area is not directly connected to the drainage system,

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

**Figure 2-3** Composite CN with connected impervious area.

Source: NRCS, June 1986, "TR-55, Urban Hydrology for Small Watersheds",  
Figure 2-3, p. 2-10.

[https://www.nrcs.usda.gov/Internet/FSE\\_DOCUMENTS/stelprdb1044171.pdf](https://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/stelprdb1044171.pdf)

**Figure 402-10 Composite CN with Connected Impervious Areas****Figure 2-4** Composite CN with unconnected impervious areas and total impervious area less than 30%

Source: NRCS, June 1986, "TR-55, Urban Hydrology for Small Watersheds",  
Figure 2-4, p. 2-10.

[https://www.nrcs.usda.gov/Internet/FSE\\_DOCUMENTS/stelprdb1044171.pdf](https://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/stelprdb1044171.pdf)

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

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

### Limitations of the Runoff Curve Number Method

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

### 402.6.1 Curve Number Weighting

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

#### Area Weighted Curve Number

Assume a design rainfall event of 2.0 inches.

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

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

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

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

#### Runoff Weighted Curve Number

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

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



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

The weighted runoff is calculated by:

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

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

### Comparison of Methods

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

#### Summary

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

## 402.7 Other Land Use Effects

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

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

## 402.8 Travel Time, Lag, and Time of Concentration

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

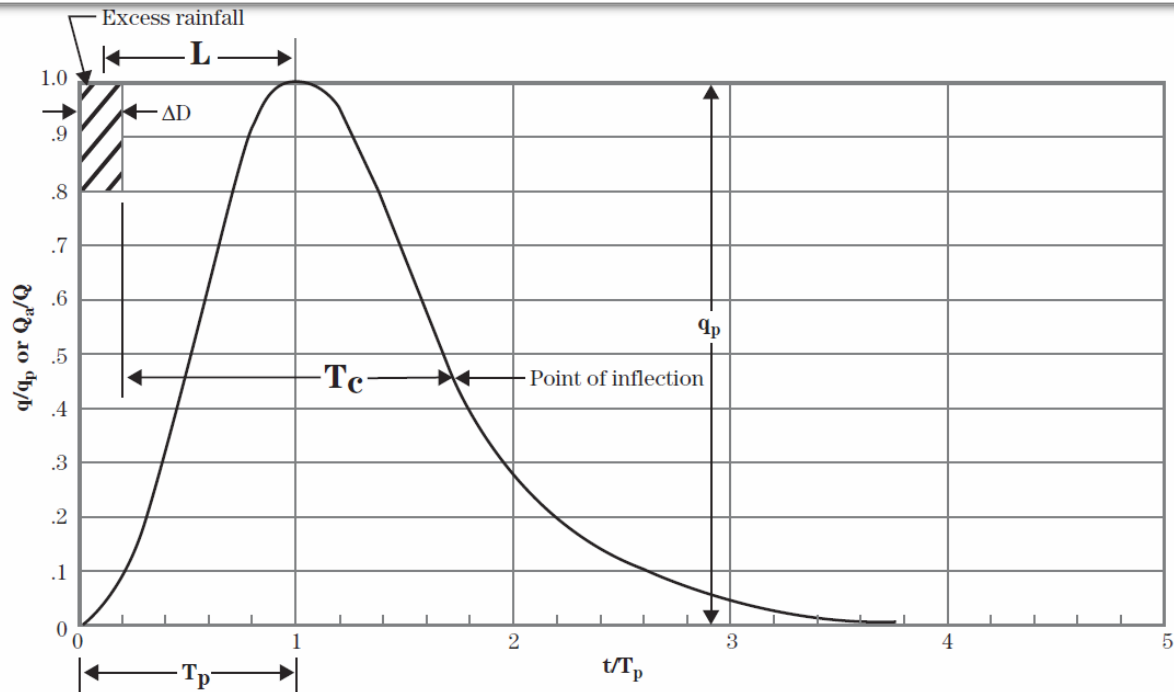
Lag (L) is the delay between the *centroid of excess rainfall* from a rainfall event over a watershed until runoff reaches its maximum flow rate. Conceptually, lag may be thought of as a weighted Time of Concentration (Tc) where, if for a given storm, the watershed is divided into subbasins, the time required for each subbasin runoff to arrive at the outfall is related to the

watershed peak by the relative contribution of each subbasin runoff in its individual lag time. In general, hydrologic modeling practice using the NRCS Unit Hydrograph Method, lag is a function of  $T_c$ .

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

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

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



where:

$L$  = Lag, h

$T_c$  = time of concentration, h

$T_p$  = time to peak, h

$\Delta D$  = duration of excess rainfall, h

$t/T_p$  = dimensionless ratio of any time to time to peak

$q$  = discharge rate at time  $t$ ,  $\text{ft}^3/\text{s}$

$q_p$  = peak discharge rate at time  $T_p$ ,  $\text{ft}^3/\text{s}$

$Q_a$  = runoff volume up to  $t$ , in

$Q$  = total runoff volume, in

Source: NRCS, 2010, "Part 630 Hydrology, National Engineering Handbook, Chapter 15 Time of Concentration", Figure 15-3, p. 15-4.

<http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=27002.wba>

**Figure 402-12 Graphical Representation of Relationships Between Lag,  $T_p$  and  $T_c$**

**Table 402-6** defines the appropriate Time of Concentration method to be used for each hydrologic method.

**Table 402-6 Selecting a Time of Concentration Calculation Method**

Hydrologic Method	Watershed Condition	Time of Concentration Method
Rational Formula Method (Section 403)	Un-gullied Watershed*	Upland Method
	Gullied Watershed*	Kirpich Equation (Kerby-Kirpich Method for Valley Areas)
Simplified Peak Discharge Method (Section 404)	Un-gullied Watershed*	Upland Method
	Gullied Watershed*	Kirpich Formula (Kerby-Kirpich Method for Valley Areas)
	Watershed Partially Gullied	Upland Method for the Un-Gullied Portion, then Kirpich Equation for the Gullied Portion
USGS Regression Equations	varies	Not Required
Unit Hydrograph Method (Section 405)	No Defined Stream Channel	Upland Method
	Defined Stream Channel	Iterative Method within the Stream Hydraulic Method
Approved Urban Method	All Conditions	Use Tc Method Specified for the Approved Urban Method

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

Within each watershed, the engineer begins by locating the flow path to the most hydraulically remote point in the watershed. This is the flow path that extends from the bottom of the watershed, or drainage structure, to the most hydraulically distant (in time) point in the watershed. Generally, this process is begun at the bottom of the watershed and is continued upstream until the longest (in time) flow path has been found. At the top of the watershed, a defined watercourse may not exist. In these areas, overland flow will be the dominant flow type. As the runoff proceeds downstream, overland flows will naturally begin to coalesce, gradually concentrating together. Shallow concentrated flow often has enough force to shape small gullies in erosive soils. Gullies eventually combine until a well-defined stream channel is formed. The

watercourse is, often at this point, large enough to be identified on a USGS quadrangle topographic map, or clearly visible in aerial photography depending on its quality.

Reaches along the primary watercourse should be divided into those which are hydraulically similar. In larger watersheds, the reaches may be sufficiently distinct to justify separate estimates of  $T_c$  for each reach of the watercourse.  $T_c$  in any given watershed is simply the sum of travel times within hydraulically similar reaches along the most remote (in time) flow path.  $T_c$  is determined from measured reach lengths and estimated average reach velocities.

The basic equation for Time of Concentration is:

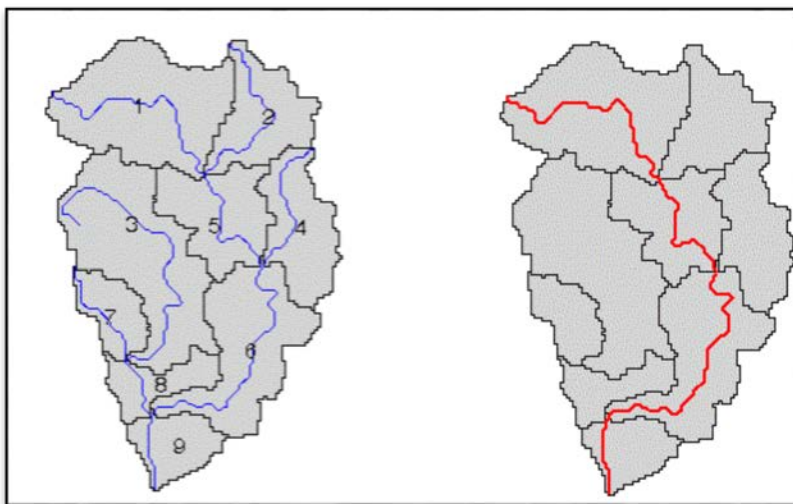
$$T_c = \frac{\left[ \frac{L_1}{V_1} + \frac{L_2}{V_2} + \frac{L_3}{V_3} + \frac{L_n}{V_n} \right]}{60} \quad 402-3$$

for minutes (or divide by 360 rather than 60 if  $T_c$  in hours is required)

where:

$T_c$	=	Time of Concentration, minutes (or hours depending on method)
$V_1$	=	average flow velocity in the uppermost reach of the watercourse, ft/s
$L_1$	=	length of the uppermost reach of the watercourse, ft
$V_2, V_3 \dots V_n$	=	average flow velocities in subsequent reaches progressing downstream, ft/s
$L_2, L_3 \dots L_n$	=	lengths of subsequent reaches progressing downstream, ft

$T_c$  is the time required for runoff to travel from the hydraulically most distant point in the watershed to the outlet. The hydraulically most distant point is the point with the longest travel time to the watershed outlet, and not necessarily the point with the longest flow distance to the outlet, see **Figure 402-13**.



**Figure 402-13 Longest Travel Time Illustration in Basin**

Time of Concentration ( $T_c$ ) is generally applied only to surface runoff and may be computed using many different methods.  $T_c$  will vary depending upon slope and character of the watershed and the flow path. In hydrograph analysis,  $T_c$  is the time from the end of excess rainfall to the point on the falling limb of the dimensionless unit hydrograph (point of inflection) where the recession curve begins, see **Figure 402-12**.

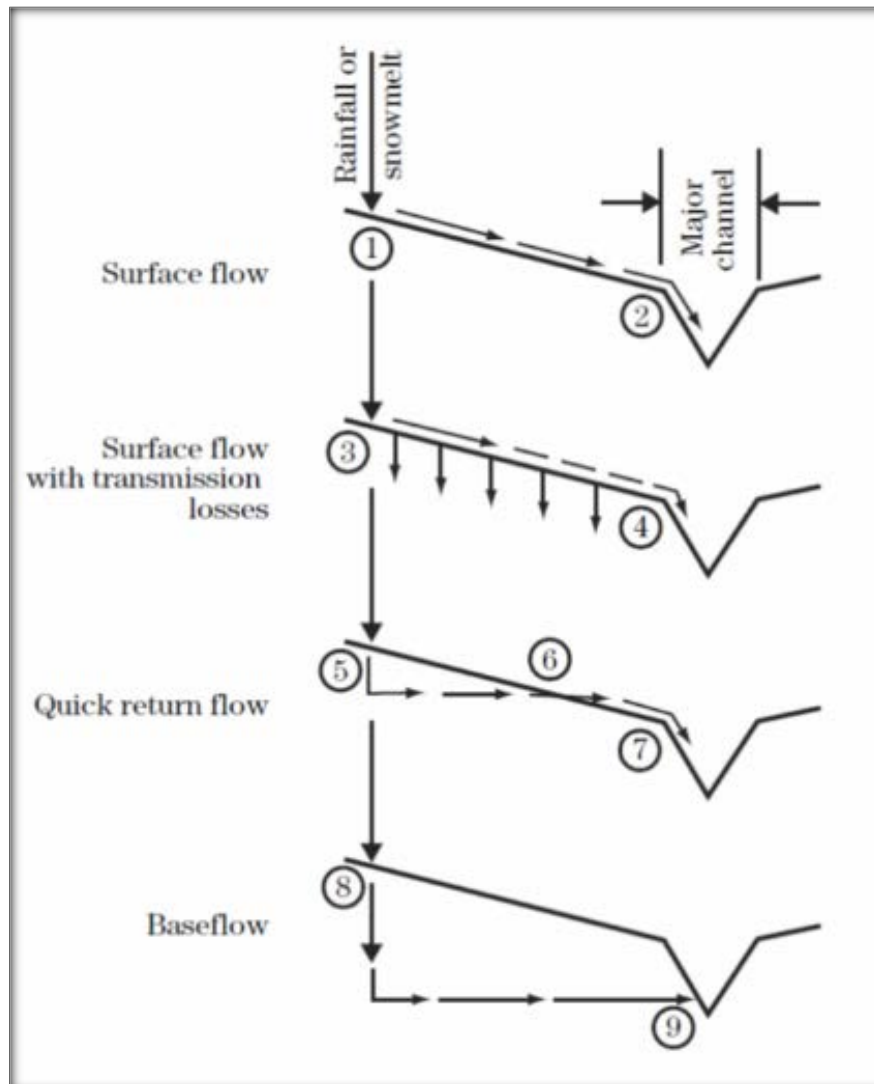
$T_c$  can be estimated using one of the methods listed in **Table 402-6**, depending on the application and circumstances. In cases where only a peak discharge and/or hydrograph are desired at the watershed outlet and watershed characteristics are fairly homogenous, the watershed may be treated as a single basin. However, if land use, Hydrologic Soil Group, slope, or other watershed characteristics are not homogeneous throughout the watershed, or the basin is large enough that the assumption of one rainfall amount is not appropriate, then divide the watershed into smaller subbasins, which requires a  $T_c$  estimation for each subbasin. Hydrographs are then developed for each subbasin and routed appropriately to a point of reference using the methods described in **Section 405.11**.

Note: Peak rates of runoff are extremely sensitive to small changes in  $T_c$ . For this reason, it is very important that the physical processes and hydraulic principles involved are very well understood and that procedures used to estimate the  $T_c$  are valid and uniformly applied.

Rainfall over a watershed (that reaches the ground) will generally follow one of four potential paths:

- Some rain will be intercepted by vegetation and evaporate into the atmosphere
- Some rain will fall onto the ground surface and evaporate
- Some rain will infiltrate into the soil
- Some rain will run directly off from the ground surface

Depending on total storm rainfall and a variety of other factors, a portion of the stormwater runoff will drain to the stream system. There are four types of flow that may occur singly or in combination throughout the watershed as presented in **Figure 402-14**.



Source: NRCS, 2010, "Part 630 Hydrology, National Engineering Handbook, Chapter 15 Time of Concentration", Figure 15-1, p. 15-2.

<http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=27002.wba>

**Figure 402-14 Types of Flow**

### **Relation between Lag, Time to Peak, and Time of Concentration**

Lag Time ( $L$ ), Time to Peak ( $T_p$ ), and Time of Concentration ( $T_c$ ) are often misunderstood. When these terms are encountered in the documents referenced in this manual, it is important to understand each of them and their relationships to one another. The following is offered to assist in that understanding.

Researchers (Mockus 1961; Simas 1996) found that **Figure 402-12** graphically portrays the relationship between average natural watershed conditions and an approximately uniform distribution of runoff.

$$L = 0.6 \times T_c$$

**402-4**

(NRCS, 2010, “Part 630 Hydrology, National Engineering Handbook, Chapter 15 Time of Concentration”, Eq. 15-3, p. 15-3)

<http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=27002.wba>

where:

L	=	Lag, hr
T <sub>c</sub>	=	Time of Concentration, hr

When runoff is not uniformly distributed due to significant differences in slope, drainage patterns, soils cover, and land use in a watershed, the watershed should be subdivided into subbasins with nearly uniform runoff characteristics so that **Equation 402-4** can be applied to each subbasin.

Four methods to calculate T<sub>c</sub> presented in this manual are:

- The Upland Method
- The Kirpich Equation
- Kerby Equation
- The Kerby-Kirpich Method
- The Iterative Method within the Stream Hydraulic Method

## **402.9 Time of Concentration**

### **402.9.1 The Upland Method**

The Upland Method (also known as the Velocity Method) is used to estimate travel times for overland flow and shallow concentrated flow conditions. The Upland Method is used for the ungullied portion of the primary watercourse when the overland flow length is 300 feet or less.

The Upland Method was originally developed by the Soil Conservation Service (SCS), which is now the Natural Resource Conservation Service (NRCS). The Upland Method is described in Chapter 15 Time of Concentration of “Part 630 Hydrology, National Engineering Handbook” (NRCS, 2010). Note that in the current (2010) version of Chapter 15, the NRCS has renamed the “Upland Method” to the “Velocity Method.” However, many documents still refer to it as the “Upland Method” and, therefore, the name “Upland Method” is used in this Drainage Design Manual.

The Upland Method is limited to use in watersheds that are less than 2,000 acres in size, or to the upper reaches of larger watersheds. For NMDOT projects the Upland Method may be used for computing the Time of Concentration when using the Rational Formula Method or the Simplified Peak Discharge Method on a largely un-gullied watershed. A watershed is considered un-gullied when 10% or less of the most hydraulically remote flow path exhibits gullyng.



Water moves through a watershed as sheet flow, shallow concentrated flow, open channel flow, or some combination of these. The type of flow that occurs is a function of the conveyance system and is best determined by field inspection.

Travel time ( $T_t$ ) is the ratio of flow length to flow velocity:

$$T_t = \frac{L}{3600 \times V} \quad 402-5$$

(NRCS, 2010, "Part 630 Hydrology, National Engineering Handbook, Chapter 15 Time of Concentration", Eq. 15-1, p. 15-2)

<http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=27002.wba>

where:

$T_t$	=	travel time, hr
$L$	=	flow length, ft
$V$	=	average velocity, ft/s
3600	=	conversion factor from seconds to hours

Time of Concentration ( $T_c$ ), is the sum of Travel Time ( $T_t$ ) values for the various consecutive flow segments:

$$T_c = T_1 + T_2 + T_3 \dots T_n \quad 402-6$$

(NRCS, 2010, "Part 630 Hydrology, National Engineering Handbook, Chapter 15 Time of Concentration", Eq. 15-7, p. 15-6)

<http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=27002.wba>

where:

$T_c$	=	Time of Concentration, hr
$T_n$	=	number of flow segments

## Sheet Flow

At the top to the watershed, sheet flow is generally the predominant flow regime. Sheet flow is defined as flow over plane surfaces. Sheet flow usually occurs in the headwaters of a stream near the ridgeline that defines the watershed boundary. Typically, sheet flow occurs for no more than 100 to 300 feet before transitioning to shallow concentrated flow (Merkel, 2001).

A simplified version of the Manning's Kinematic Equation may be used to compute travel time for sheet flow. This simplified form of the Kinematic Equation presented here was developed by (Welle and Woodward, 1986) after studying the impact of various parameters on the estimates.

$$T_t = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5} S^{0.4}} \quad 402-7$$

(NRCS, 2010, "Part 630 Hydrology, National Engineering Handbook, Chapter 15 Time of Concentration", Eq. 15-8, p. 15-6)

<http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=27002.wba>

where:

$T_t$	=	travel time, hr
$n$	=	Manning's roughness coefficient ( <b>Table 402-7</b> )
$l$	=	sheet flow length, ft
$P_2$	=	2-year, 24-hour rainfall, in.
$S$	=	slope of land surface, ft/ft

This simplification is based on the following assumptions:

- Shallow steady uniform flow
- Constant rainfall excess intensity (that part of a rain available for runoff) both temporally and spatially
- 2-year, 24-hour rainfall assuming standard NRCS rainfall intensity-duration relations apply (Types I, II, and III)
- Minor effect of infiltration on travel time

For sheet flow, the roughness coefficient includes the effects of roughness and the effects of raindrop impact including drag over the surface; obstacles such as litter, crop row ridges, and rocks; and erosion and sediment transport. These “ $n$ ” values are only applicable for flow depths of approximately 0.1 foot or less, where sheet flow occurs. **Table 402-7** gives roughness coefficient values for sheet flow for various surface conditions.

**Table 402-7 Roughness Coefficients (Manning's "n") for Sheet Flow**

Source: NRCS, 2010, "Part 630 Hydrology, National Engineering Handbook, Chapter 15 Time of Concentration", Table 15-1, p. 15-6.

<http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=27002.wba>

Surface description	"n" <sup>1/</sup>
Smooth surfaces (concrete, asphalt, gravel, or bare soil).....	0.011
Fallow (no residue) .....	0.05
Cultivated soils:0.	
Residue cover ≤20% .....	0.06
Residue cover >20% .....	0.17
Grass:	
Short grass prairie .....	0.15
Dense grasses <sup>2/</sup> .....	0.24
Bermuda grass .....	0.41
Range (natural).....	0.13
Woods: <sup>3/</sup>	
Light underbrush .....	0.40
Dense underbrush.....	0.80

<sup>1/</sup> The "n" values are a composite of information compiled by Engman (1986).  
<sup>2/</sup> Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures.  
<sup>3/</sup> When selecting "n", consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.

It is important to note that there are many locations in New Mexico where there is little or no runoff resulting from a 2-year storm and that due to the combination of high desert climate and soils in the upper portions of many watersheds, there is no evidence of gully formation for distances far exceeding 100 to 300 feet. However, the maximum sheet flow length used for NMDOT hydrologic analyses should not exceed 300 feet, except when a greater length can be justified by onsite inspection of the upper watershed or through inspection of high resolution aerial photography.

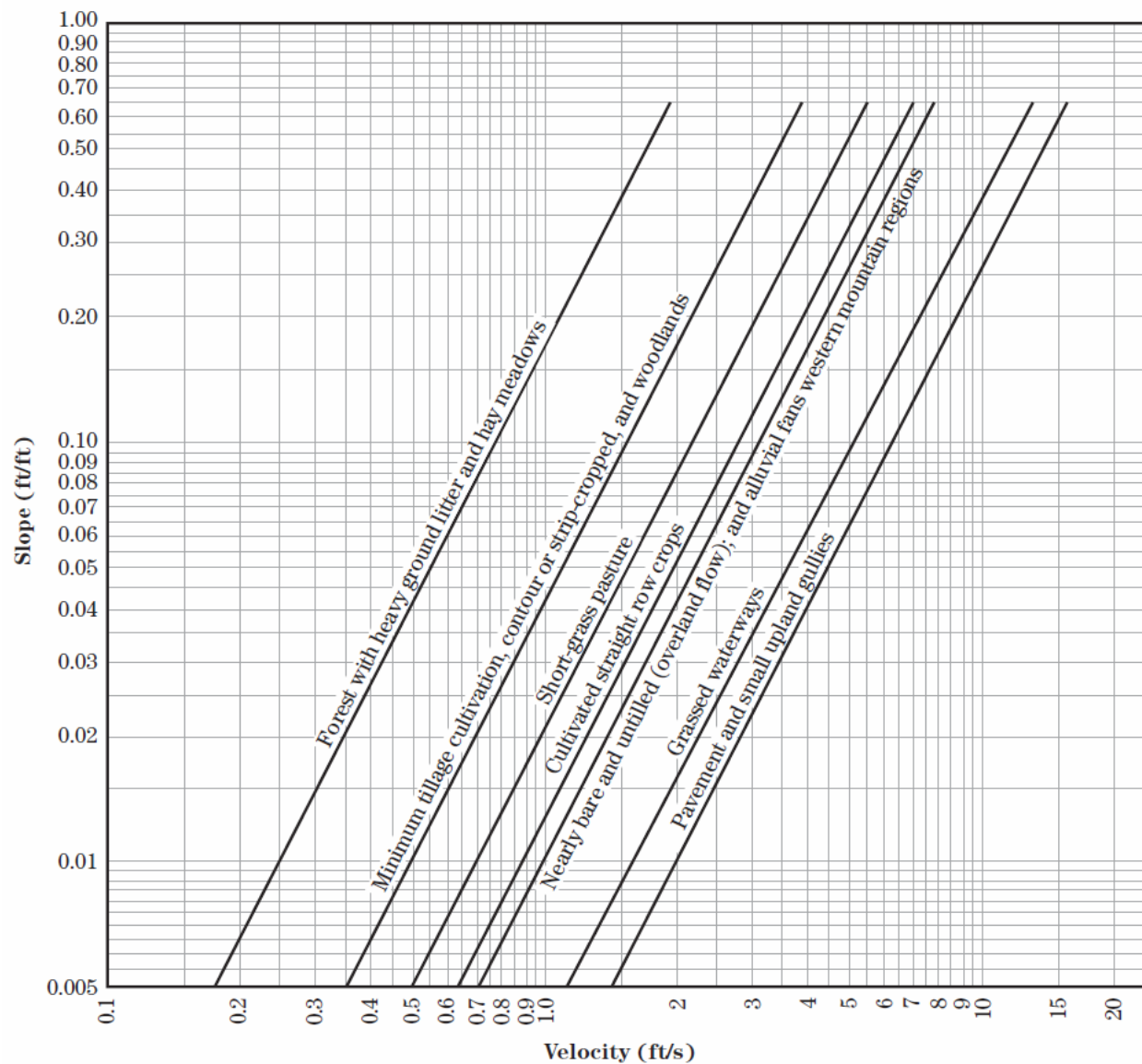
Overland flow continues until the volume of water is sufficient to create a shallow concentrated flow regime. In erosive soil formations with limited ground cover, the length of overland flow may be so short that it is negligible. Given the slope of the land and some knowledge of the ground

cover conditions, once the most hydraulically remote flow path is determined, the overland flow length can be determined.

For NMDOT projects, shallow concentrated flow is assumed to occur from the end of overland flow to the bottom of a watershed where there is little or no gullying (10% or less). Where gullying is evident in the majority of the watershed (by field inspection, aerial photography or by a blue line shown on the USGS quadrangle topographic map), the Time of Concentration should be computed by the Kirpich Equation for the entire watershed. When the Simplified Peak Discharge Method is being used for NMDOT projects, the Upland Method may be used for the un-gullied portion of the watercourse, in combination with the Kirpich Equation for the gullied sections of the watercourse. For watersheds with more than 30% of the uplands or with little or no gullying (valley areas), the Kerby-Kirpich Method should be used. The NMDOT Drainage Design Bureau can be contacted to obtain a copy of a spreadsheet to determine  $T_c$  using these methods. Note that the Engineer/Consultant is responsible for understanding the use of, and the accuracy of the results from this spreadsheet.

### Shallow Concentrated Flow

After approximately 100 to 300 feet, sheet flow usually becomes shallow concentrated flow collecting in swales, small rills, and gullies. Shallow concentrated flow is assumed not to have a well-defined channel and has flow depths of 0.1 to 0.5 feet. It is assumed that shallow concentrated flow can be represented by one of seven flow types. **Figure 402-15** presents curves as Velocity versus Slope for Shallow Concentrated Flow and these curves were used to develop the information in **Table 402-8**. To estimate shallow concentrated flow travel time, velocities are developed using **Figure 402-15**, in which average velocity is a function of watercourse slope and type of channel (Kent, 1973). For slopes less than 0.005 feet per foot, the equations in **Table 402-8** may be used. After estimating average velocity using **Figure 402-15**, use **Equation 402-5** to estimate travel time for the shallow concentrated flow segment.



Source: NRCS, 2010, "Part 630 Hydrology, National Engineering Handbook, Chapter 15 Time of Concentration", Figure 15-4, p. 15-8.

<http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=27002.wba>

**Figure 402-15 Velocity Versus Slope for Shallow Concentrated Flow**

**Table 402-8 Equations and Assumptions Developed from Figure 402-15**

Source: NRCS, 2010, "Part 630 Hydrology, National Engineering Handbook, Chapter 15 Time of Concentration", Table 15-3, p.15-8.

<http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=27002.wba>

Flow type	Depth (ft)	Manning's <i>n</i>	Velocity equation (ft/s)
Pavement and small upland gullies	0.2	0.025	$V = 20.328(s)^{0.5}$
Grassed waterways	0.4	0.050	$V = 16.135(s)^{0.5}$
Nearly bare and untilled (overland flow); and alluvial fans in western mountain regions	0.2	0.051	$V = 9.965(s)^{0.5}$
Cultivated straight row crops	0.2	0.058	$V = 8.762(s)^{0.5}$
Short-grass pasture	0.2	0.073	$V = 6.962(s)^{0.5}$
Minimum tillage cultivation, contour or strip-cropped, and woodlands	0.2	0.101	$V = 5.032(s)^{0.5}$
Forest with heavy ground litter and hay meadows	0.2	0.202	$V = 2.516(s)^{0.5}$

For that portion of the flow path that is channel flow, use Manning's Equation (**Equation 402-10**) to calculate the velocity. The approach outlined in **Section 402.9.5** should be followed to determine the average velocity for the channel reaches.

Once the reach lengths and flow velocities for each defined reach along the flow path have been calculated as described above, the  $T_c$  for each of the segments are added together to find the total  $T_c$ .

### 402.9.2 Time of Concentration by the Kirpich Equation

The Kirpich Equation should be used in watersheds when gullying (including manmade conveyances in fully urbanized watersheds such as curb and gutter, storm drains and channels) is evident in more than 10% of the primary watercourse. Gullying can be assumed if a blue line appears on the watercourse shown on the USGS quadrangle topographic map or is apparent from field investigation or from inspection of aerial photography. The Kirpich Equation is given as:

$$T_c = 0.0078 \times L^{0.77} \times S^{-0.385} \quad \text{402-8}$$

(TxDOT, July 2016, "Hydraulic Design Manual", Eq. 4-15, p. 4-39)

<http://onlinemanuals.txdot.gov/txdotmanuals/hyd/index.htm>

where:

$T_c$	=	Time of Concentration, minutes
$L$	=	maximum length of water travel, ft
$S$	=	surface slope, given by $H/L$ , ft/ft
$H$	=	difference in elevation between the most hydraulically remote point in the drainage basin and the outlet, ft

In small watersheds where the slope is flat, and the flow path of the hydraulically longest flow path is dominated by overland flow greater than 300 feet, the Kerby Equation should be considered for the overland flow portion and Kirpich Equation for the channelized portion.

In gullied (and in fully urbanized) basins, the Kirpich Equation should generally be used for the entire drainage basin. The exception to this rule occurs when the Simplified Peak Discharge Method is being used on NMDOT projects or when the watercourse has a mixture of gullied and un-gullied sections. In these situations, mixing of Time of Concentration methods is allowed and is called the Kerby-Kirpich Method as described in **Section 402.9.4**.

### 402.9.3 Time of Concentration by the Kerby Equation

For small watersheds where overland flow and overland flow length are an important component of overall travel time, the Kerby Equation can be used. The Kerby Equation is:

$$T_{OV} = K (L \times N)^{0.467} \times S^{-0.235} \quad 402-9$$

(TxDOT, July 2016, "Hydraulic Design Manual", Eq. 4-14, p. 4-37)

<http://onlinemanuals.txdot.gov/txdotmanuals/hyd/index.htm>

where:

$T_{OV}$	=	overland flow Time of Concentration, minutes
$K$	=	a unit conversion coefficient, in which $K = 0.828$
$L$	=	the overland-flow length, feet
$N$	=	a dimensionless retardance coefficient
$S$	=	the dimensionless slope of terrain conveying the overland flow

In the development of the Kerby Equation, the length of overland flow was as much as 1,200 feet. This length is considered an upper limit, and in practice, shorter values generally are expected. The dimensionless retardance coefficient used is similar in concept to the well-known Manning's roughness coefficient; however, for a given type of surface, the retardance coefficient for overland flow will be considerably larger than for open-channel flow. Typical values for the retardance coefficient are listed in **Table 402-9**. Roussel et al., 2005, recommends that the user should not interpolate the retardance coefficients in **Table 402-9**. If it is determined that a low slope condition or a transitional slope condition exists, the user should consider using an adjusted slope in calculating the Time of Concentration.

**Table 402-9 Kerby Equation Retardance Coefficient Values**

Source: TxDOT, July 2016, "Hydraulic Design Manual", Table 4-5, p. 4-38.

<http://onlinemanuals.txdot.gov/txdotmanuals/hyd/index.htm>

Generalized Terrain Description	Dimensionless Retardance Coefficient (N)
Pavement	0.02
Smooth, bare, packed soil	0.10
Poor grass, cultivated row crops, or moderately rough packed surfaces	0.20
Pasture, average grass	0.40
Deciduous forest	0.60
Dense grass, coniferous forest, or deciduous forest with deep litter	0.80

#### **402.9.4 The Kerby-Kirpich Method**

The Upland Method is used for the ungullied portion of the primary watercourse when the overland flow length is 300 feet or less. The Kerby Equation should be used for the ungullied portions when the overland flow length is greater than 300 feet. The Kirpich Equation is used for the gullied portion of the watercourse, including those drained by manmade conveyances such as curb and gutter, storm drains and channels. The  $T_c$  result from each equation are added to obtain the watershed total  $T_c$ , thus the name "Kerby-Kirpich" Method.

#### **402.9.5 The Iterative Method Within the Stream Hydraulic Method**

The Iterative Method within the Stream Hydraulic Method is used when calculating peak discharges by the Unit Hydrograph Method in a watercourse where a defined stream channel is evident in the field or aerial photography (or a blue line, solid or broken, on a quadrangle topo map) and is the dominant runoff conveyance in the watershed. The Iterative Method within the Stream Hydraulic Method is applicable principally on larger basins where the longest flow path is dominated by channel flow, but that are small enough not to warrant subdividing the basin, or in basins where gullying is evident all the way to the top of the basin.

The engineer must measure or estimate the hydraulic properties of the stream channel. The total watercourse must be divided into channel reaches which are hydraulically similar within themselves. Often, hydraulically similar reaches will have similar slopes. Dramatic slope changes should be apparent from both topography and channel shape. Field reconnaissance measurements of the stream channel are suggested; however, sometimes direct measurements are not possible. The engineer must determine the slope, channel cross section, and an appropriate hydraulic roughness coefficient for each channel reach using the best information available within the limits of access, time, and budgets (topographic maps, aerial photography,



etc.). Average slope is often determined from the topographic mapping. Channel cross sections should be measured in the field whenever possible, but scalable aerial photography may provide sufficient information to assess channel cross section characteristics.

Roughness coefficients of the waterway should be based on actual observations of the watercourse or of accessible nearby watercourses which are believed to be similar. If the reach is inaccessible, and if there is good quality aerial photography available it may provide adequate information for this purpose.

Time of Concentration (Tc) by Iterative Method within the Stream Hydraulic Method is simply the travel time (Tt) in the stream channel. Channel flow velocities can be estimated from normal depth calculations for the watercourse. In addition to the average flow velocity, engineers should compute the Froude number (Fr) of the flow. If the Fr number of the flow exceeds a value of 1.3, the engineer should verify that supercritical flow conditions can be sustained. For most earth lined channels, the velocity calculation should be recomputed using a larger effective Manning's roughness coefficient "n" until the Froude number has a value less than 1.3. Note that most upland arroyos flow very close to critical depth (Fr=1) and in most cases, normal depth and critical depth are very close to the same depth and velocity.

Velocity (V) is determined from Manning's Equation:

$$V = \frac{1.486}{n} R^{0.667} S^{0.5} \quad 402-10$$

where:

V	=	velocity, ft/s
n	=	Manning's roughness coefficient
R	=	hydraulic radius (area/wetted perimeter), ft
S	=	slope of the energy grade line (assumed to be the same as the channel slope) ft/ft

Froude number (Fr) is calculated by the following equation:

$$Fr = \frac{V}{(g d)^{0.5}} \quad 402-11$$

where:

Fr	=	Froude number
V	=	velocity, ft/s
g	=	gravitational acceleration, 32.2 ft/s <sup>2</sup>
d	=	hydraulic depth (flow cross sectional area/top width of flow), ft

In order to solve Manning's Equation for velocity (V), calculate or estimate the hydraulic radius (R). If the flow depth or flow rate is known, then R may be found directly. However, the usual situation is that neither flow depth nor flow rate are known without first computing the Tc and an initial discharge. Three procedures are provided below for solving this problem.

## Simplified Flow Estimating Procedure

### Wide Shallow Channels

Use this method for channels where the flow depth is relatively shallow compared to the flow width. When this is true, the hydraulic radius ( $R$ ) converges toward depth ( $d$ ). The use of  $R=d$  is acceptable for NMDOT projects where the stream channel is relatively wide, and the flow is shallow. Larger arroyo systems in alluvial terrain often satisfy this criterion.

### Moderate and Narrow Width Channels

Use this method for all other channels. Estimate the flow depth from high water mark evidence or other available data. For most ephemeral stream channels, the 25-year to 100-year storm flow depths may be in the range of 1 to 3 ft. Where a channel has obvious channel banks in the 1 to 3 ft height range, use the "bank full" depth. For most ephemeral streams use the bank full depth of the low flow channel. If the evidence suggests a flow depth greater than the height of an incised channel bank, use the physical evidence depth but compute the flow velocity based on water in the channel only (no overbank flow considered). Use the flow depth and channel cross section geometry to estimate  $R$ . For estimated flow depths deeper than 3 to 5 ft, the engineer should consider using the iterative procedure described below.

### Iterative Procedure

For some channel flow conditions, the simplified procedures described above may not be adequate. In these cases, the iterative procedure described here must be followed. First, the peak rate of runoff from the watershed is estimated. A beginning estimate may be obtained using experience and judgment or by using the USGS regional regression equations for New Mexico (see **Section 407** of this Manual.) The flow rate for the velocity calculation is assumed to be two-thirds of the peak rate. Average channel velocity is calculated from **Equation 402-10** using the other hydraulic parameters of the channel. The average channel velocity for each reach is then used to determine the total  $T_c$  for the watershed. After the peak discharge from the watershed is computed, reassess the flow rate used to compute an average channel velocity. If the assumed peak discharge is within 10% of the calculated peak discharge, the computed average channel velocity and resulting  $T_c$  should be reasonably accurate. Often a second iteration is required using two-thirds of the computed peak flow to compute a new average channel velocity. This iterative procedure should be continued until the assumed peak discharge rate is within 10% of the computed peak discharge rate. Appendix 6 contains **Example Problem 6-5** that demonstrates this Method. Note: use of a computer program to calculate normal depth will greatly expedite this iterative procedure.

## **402.10 Channel and Floodplain Characteristics**

Stream channels, floodplains, and reservoirs can have a significant impact on the delivery of water to any location along a stream network. Flood routing impacts the magnitude of the peak discharge, the time of the peak discharge, depth and extent of flooding, and environmental factors such as stream bank erosion, floodplain scour, sediment transport, and deposition.

The size, shape, and configuration of the channel and floodplain of a stream system are a reflection of the hydrologic processes within the watershed that created the stream system. A channel/floodplain system that is part of a high runoff producing watershed will look dramatically

different than one that regularly produces little runoff. The process of both developing the hydrologic parameters needed to perform hydrologic analyses and the qualitative review of the results should include an assessment of the resulting channel/floodplain system.

The Time of Concentration (Tc) calculation is one of the most critical input parameters to any deterministic (as opposed to probabilistic) hydrologic analysis. Tc in a large watershed is determined largely on the hydraulics of the channel and floodplain system while in smaller watersheds, sheet flow and shallow concentrated flow may dominate.

Hydraulic parameters and qualities such as slope, cross section, bed form, Manning's roughness coefficient "n", rating curves, sediment size, sediment volumes, vegetation type and densities, are all related to the watershed's response to rainfall and the climate in which the watershed is located. Experience and judgment are required to assess the relative importance and impacts of each of these parameters. This experience is gained by always beginning with a qualitative assessment of the channel/floodplain system. Then developing hydrologic and hydraulic data, assumptions and calculations, and then checking the analysis results to verify that they are reasonable given the characteristics of the channel/floodplain system.

#### **402.11 Sediment Bulking**

Flood flows from high-intensity rainfall events on bare or mostly bare soils and flows within ephemeral sand bottom arroyos often contain significant amounts of sediment. When using one of the deterministic modeling approaches (but not Regional Regression Equations or streams with gage records) in this manual, it should be recognized that the resulting peak discharge and runoff volume are clean or clear water values, and therefore do not include the flow bulking that results from sediment.

##### Conveyance Structures

If the water conveyance structure (culvert, concrete box culvert, or bridge) has 120% or more of the required design capacity above the clear water discharge to meet NMDOT hydraulic criteria, then no further bulking factor analyses is required. However, if the conveyance structure does not meet the 120% criterion, see **Table 205-1**, then a more rigorous bulking factor analysis must be performed, or upsize the conveyance structure.

##### Detention and Retention Ponds

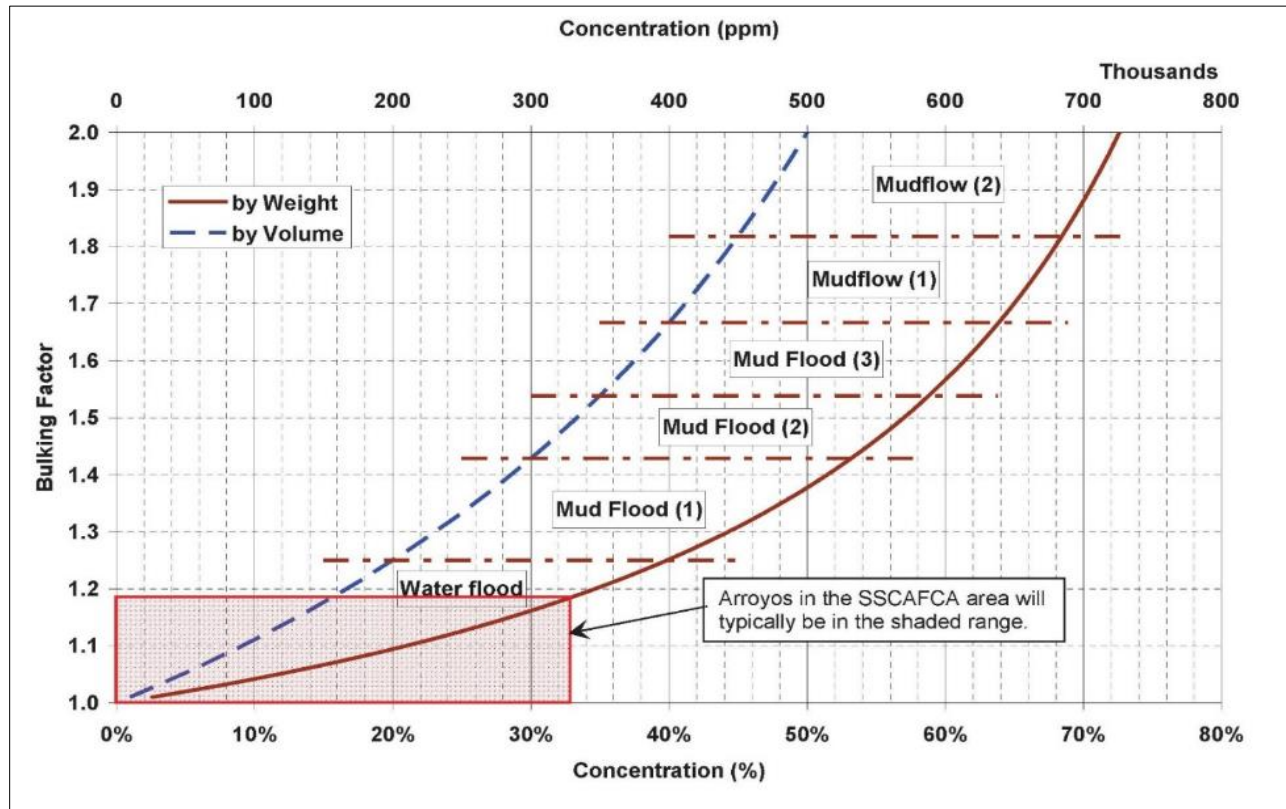
For the hydrologic analyses required for pond design, clear water storm runoff hydrographs must account for sediment by application of sediment bulking factors. The information presented in this Section combined with the pond design requirements presented in **Section 207** must be addressed during pond design.

#### **402.11.1 SCAFCA Sediment and Erosion Guide**

The information in this Section was excerpted from a document titled "Sediment and Erosion Design Guide", November 2008, developed for the Southern Sandoval County Arroyo Flood Control Authority (SCAFCA), prepared by Mussetter Engineering, Inc.

<http://sscafca.org/sediment-and-erosion-design-guide/>

**Figure 402-16** provides a guide to a range of possible sediment bulking factors in relation to sediment concentration for sand arroyos in the Sandoval County area. These figures and the supporting text of the Sediment and Erosion Guide will assist in estimating sediment bulking factors in arroyos outside the Sandoval County area (qualitatively at least).



Source: SSCAFCA, November 2008, "Sediment and Erosion Design Guide", Figure 3.8, p. 3.24.  
[http://sscafca.org/development/documents/sediment\\_design\\_guide/Sediment%20Design%20Guide%202012-30-08.pdf](http://sscafca.org/development/documents/sediment_design_guide/Sediment%20Design%20Guide%202012-30-08.pdf)

**Figure 402-16 Relationship between Total Sediment Concentration and Bulking Factor**

### Bulking Factors for the SSCAFCA Area

Discharges estimated using standard rainfall-runoff procedures typically do not account for the presence of sediment in the flow. At high sediment loads, the total volume of the water/sediment mixture, and thus, the peak design discharges, can be substantially higher than the corresponding clear-water values. The following relation provides a means of computing a bulking factor ( $B_f$ ) which is a factor applied to adjust (increase) the clear-water discharges for the presence of the transported sediment, if the sediment load is known:

$$B_f = \frac{Q + Q_{S_{\text{total}}}}{Q} = \frac{1}{1 - \frac{C_s / 10^6}{S_g - (C_s / 10^6)(S_g - 1)}}$$

**402-12**

(SSCAFCA, November 2008, “Sediment and Erosion Guide”, Eq. 3.25, p. 3.23)

[http://sscafca.org/development/documents/sediment\\_design\\_guide/Sediment%20Design%20Guide%2012-30-08.pdf](http://sscafca.org/development/documents/sediment_design_guide/Sediment%20Design%20Guide%2012-30-08.pdf)

where:

$B_f$	=	bulking factor
$Q$	=	clear-water discharge, cfs
$Q_{s\text{ total}}$	=	total sediment load (i.e., combination of bed material and wash load), cfs
$C_s$	=	total sediment concentration by weight, ppm and
$S_g$	=	specific gravity of the sediment

This relationship indicates that the bulked discharge for a water/sediment mixture at the upper limit of concentrations for water floods (200,000 ppm by volume or 410,000 ppm by weight) would be about 25 percent greater than the clear water discharge (i.e., a bulking factor of 1.25) (**Figure 402-16**).

Because specific knowledge of the sediment load is often not available, conservative estimates of the bulking factor that can be applied to a range of potential design discharges were made by applying the MPM-Woo procedure for a typical rectangular cross section with width-depth ratio ( $F_D$ ) at the dominant discharge ( $Q_D$ ) of 40, assuming critical flow conditions and a range of median ( $D_{50}$ ) particle sizes. Dominant discharge is defined in **Figure 402-17**, and a method for estimating its magnitude is provided in the text box that follows. Note that the figure enclosed within the text box is difficult to read as is the original document (SSCAFCA, 2008).

Chapter 3 of this guide provides guidance in relating bulking factors to median ( $D_{50}$ ) bed material size for the following recurrence interval floods: 2-, 5-, 10-, 25-, 50- and 100-year, based on a range of dominant discharge values.  $D_{50}$  is defined as the sediment size for which 50% of the sample is finer by weight.

### Annual Sediment Yield and Dominant Discharge

The **dominant** (or effective) discharge is defined as the increment of discharge that carries the most sediment over a long period of time (Wolman and Miller, 1960; Andrews, 1980; Biedenharn et al., 2000). In perennial, self-adjusted streams, the dominant discharge is often assumed to be same as the bankfull discharge because this represents the long-term condition to which the channel has adjusted, and it is also often assumed to be equivalent to about the mean annual flood peak. Care must be taken in making these assumptions, however, because the dominant, bankfull and mean annual flood peak discharges can be quite different, even in perennial, self-adjusted stream. For ephemeral streams, the dominant discharge tends to be associated with larger, less frequent flood peaks than in perennial streams, due to the absence of sustained flows and the flashy nature of the storm hydrographs. For design purposes, the dominant discharge for lightly developed watersheds in the SSCAFCA jurisdictional area will typically be in the range of the 5- to 10-year peak discharge. In more highly developed watersheds, the frequency of the dominant discharge is typically less because runoff (and sediment transport) associated with the more frequent storms tends to increase dramatically. As a result, the frequency of the dominant discharge is typically in the range of the 3- to 5-year flood peak.

### A quantitative method for estimating $Q_D$ in arroyos

If bed-material transport rating curves and storm hydrographs are available, the dominant discharge can be estimated as the peak of the storm event that will produce a bed-material sediment yield equal to the mean annual bed-material sediment yield. The mean annual sediment yield can be estimated by integrating the sediment yield frequency curve (Chang, 1988):

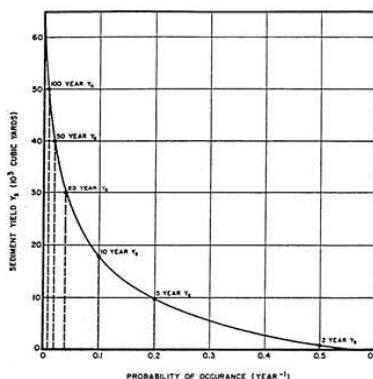
$$Y_{sm} = \int_0^1 Y_s dP_F \quad (3.26)$$

where  $Y_s$  is the individual storm sediment yield and  $P_F$  is the probability of occurrence of that flood in one year. The product  $Y_s \cdot P_F$  represents the contribution of a particular flood to the long-term mean annual yield. For practical purposes, the integration can be accomplished for a series of discrete storm events using the trapezoidal rule. Using the 2-, 5-, 10-, 25-, 50-, and 100-year events, for example, the mean annual sediment yield is approximated by the following relationship:

$$Y_{sm} = 0.015 Y_{s100} + 0.015 Y_{s50} + 0.04 Y_{s25} + 0.08 Y_{s10} + 0.2 Y_{s5} + 0.4 Y_{s2} \quad (3.27)$$

If only the 2-, 10- and 100-year events are used, the following relationship is obtained:

$$Y_{sm} = 0.055 Y_{s100} + 0.245 Y_{s10} + 0.45 Y_{s2} \quad (3.28)$$

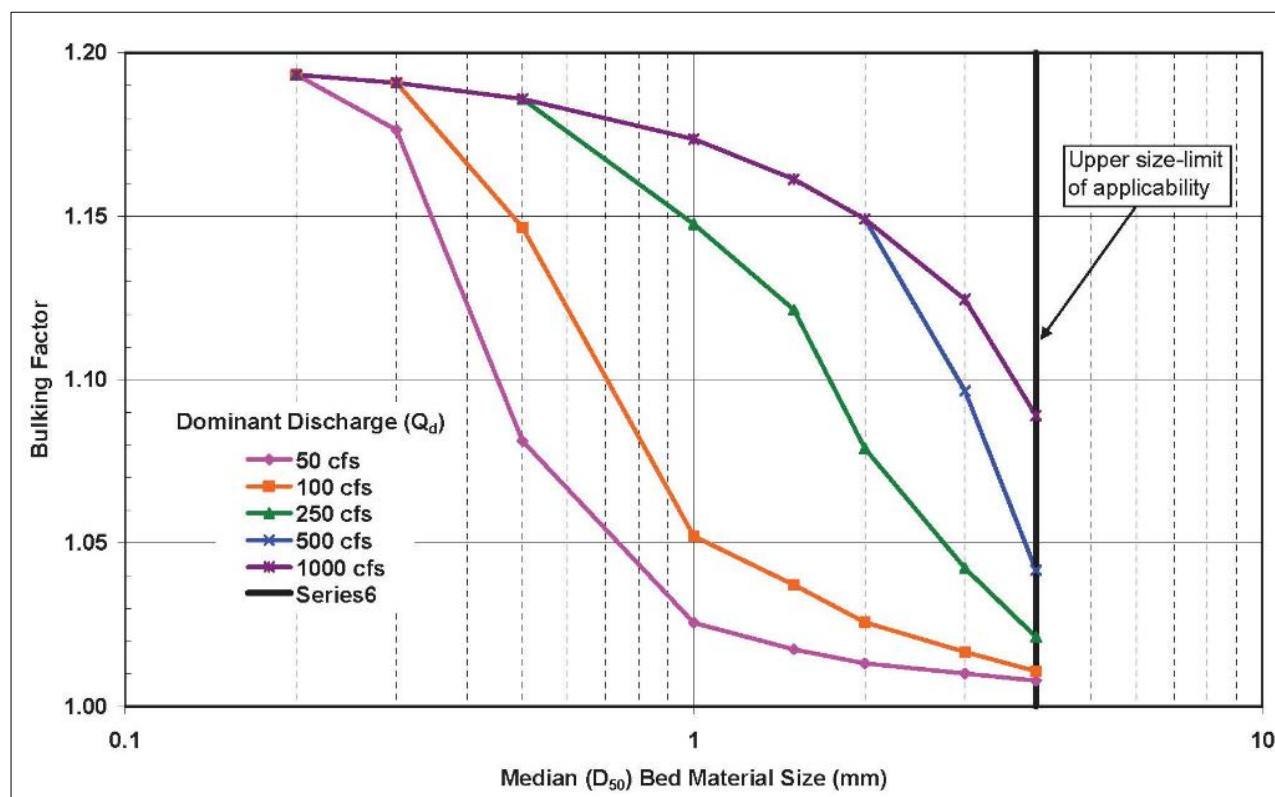


Source: SSCAFCA, November 2008, "Sediment and Erosion Design Guide", p. 3.28.  
[http://sscafca.org/development/documents/sediment\\_design\\_guide/Sediment%20Design%20Guide%2012-30-08.pdf](http://sscafca.org/development/documents/sediment_design_guide/Sediment%20Design%20Guide%2012-30-08.pdf)

**Figure 402-17 Annual Sediment Yield and Dominant Discharge**



The assumed width-depth ratio ( $F_D$ ) of 40 is based on data from a variety of existing, naturally adjusted arroyos (Leopold and Miller, 1956; Harvey et al., 1985). The assumption of critical flow is based on the observation that average Froude numbers ( $F_r$ ) in stable sand-bed streams rarely exceed 0.7 to 1.0 (Richardson, personal communication) at high discharges. It should also be noted that current FEMA procedures for evaluating hydraulic conditions on alluvial fans is based on the assumption of critical flow ( $F_r = 1$ ). Based on analysis of a wide range of arroyos in the greater Rio Rancho and Albuquerque area, the dominant discharge typically has a recurrence interval in the range of 5 to 10 years under relatively undeveloped conditions and decreases to 3 to 5 years under highly developed conditions due, primarily, to the increase in runoff during frequently occurring storms. The peak discharge associated with other recurrence interval flows was estimated using average ratios for conditions in the greater Rio Rancho and Albuquerque area. The 100-year peak discharge, for example, averages about five times the dominant discharge. Bulking factors estimated using the above assumptions for the 100-year peak are shown in **Figure 402-18** for channels with dominant discharge ranging from 50 to 1,000 cfs and median ( $D_{50}$ ) bed-material sizes ranging from 0.5 to 4 mm. As shown in that figure, the bulking factors range from about 1.01 for small arroyos ( $W_d < 50$  cfs) with relatively coarse bed material ( $D_{50} = 4$  mm) to a maximum of 1.19 for larger channels ( $Q_D > 500$  cfs) and relatively fine bed material ( $D_{50} \leq 0.5$  mm). Estimated bulking factors for other recurrence interval events for the median bed-material sizes are provided in **Figure 402-19**.



Source: SSCAFCA, November 2008, "Sediment and Erosion Design Guide", Figure 3.9, p. 3.25.  
[http://sscafca.org/development/documents/sediment\\_design\\_guide/Sediment%20Design%20Guide%202012-30-08.pdf](http://sscafca.org/development/documents/sediment_design_guide/Sediment%20Design%20Guide%202012-30-08.pdf)

**Figure 402-18 Bulking Factors for the 100-year Peak Discharge for Natural Channels**

Table 3.6. Estimated sediment bulking factors for arroyos in the SSCAFCA jurisdictional area.					
Recurrence Interval (yrs)	Dominant Discharge (cfs)				
	50	100	250	500	1,000
$D_{50}$ (mm) = 0.5 mm					
2	1.01	1.01	1.01	1.01	1.02
5	1.02	1.02	1.05	1.08	1.14
10	1.03	1.05	1.10	1.19	1.19
25	1.05	1.09	1.19	1.19	1.19
50	1.07	1.12	1.19	1.19	1.19
100	1.08	1.15	1.19	1.19	1.19
$D_{50}$ (mm) = 1.0 mm					
2	1.01	1.01	1.01	1.01	1.01
5	1.01	1.01	1.01	1.03	1.05
10	1.01	1.01	1.03	1.07	1.16
25	1.02	1.03	1.08	1.17	1.17
50	1.02	1.04	1.12	1.17	1.17
100	1.03	1.05	1.15	1.17	1.17
$D_{50}$ (mm) = 1.5 mm					
2	1.01	1.01	1.01	1.01	1.01
5	1.01	1.01	1.01	1.02	1.04
10	1.01	1.01	1.02	1.05	1.13
25	1.01	1.02	1.06	1.14	1.16
50	1.01	1.03	1.09	1.16	1.16
100	1.02	1.04	1.12	1.16	1.16
$D_{50}$ (mm) = 2.0 mm					
2	1.01	1.01	1.01	1.01	1.01
5	1.01	1.01	1.01	1.01	1.03
10	1.01	1.01	1.02	1.04	1.08
25	1.01	1.01	1.04	1.09	1.15
50	1.01	1.02	1.06	1.15	1.15
100	1.01	1.03	1.08	1.15	1.15
$D_{50}$ (mm) = 3.0 mm					
2	1.01	1.01	1.01	1.01	1.01
5	1.01	1.01	1.01	1.01	1.02
10	1.01	1.01	1.01	1.02	1.04
25	1.01	1.01	1.02	1.05	1.11
50	1.01	1.01	1.03	1.07	1.12
100	1.01	1.02	1.04	1.10	1.12
$D_{50}$ (mm) = 4.0 mm					
2	1.01	1.01	1.01	1.01	1.01
5	1.01	1.01	1.01	1.01	1.01
10	1.01	1.01	1.01	1.02	1.03
25	1.01	1.01	1.02	1.03	1.06
50	1.01	1.01	1.02	1.04	1.10
100	1.01	1.01	1.03	1.06	1.10

Source: SSCAFCA, November 2008, "Sediment and Erosion Design Guide", Table 3.6, p. 3.26.  
[http://sscafca.org/development/documents/sediment\\_design\\_guide/Sediment%20Design%20Guide%2012-30-08.pdf](http://sscafca.org/development/documents/sediment_design_guide/Sediment%20Design%20Guide%2012-30-08.pdf)

**Figure 402-19 Estimated Bulking Factors**

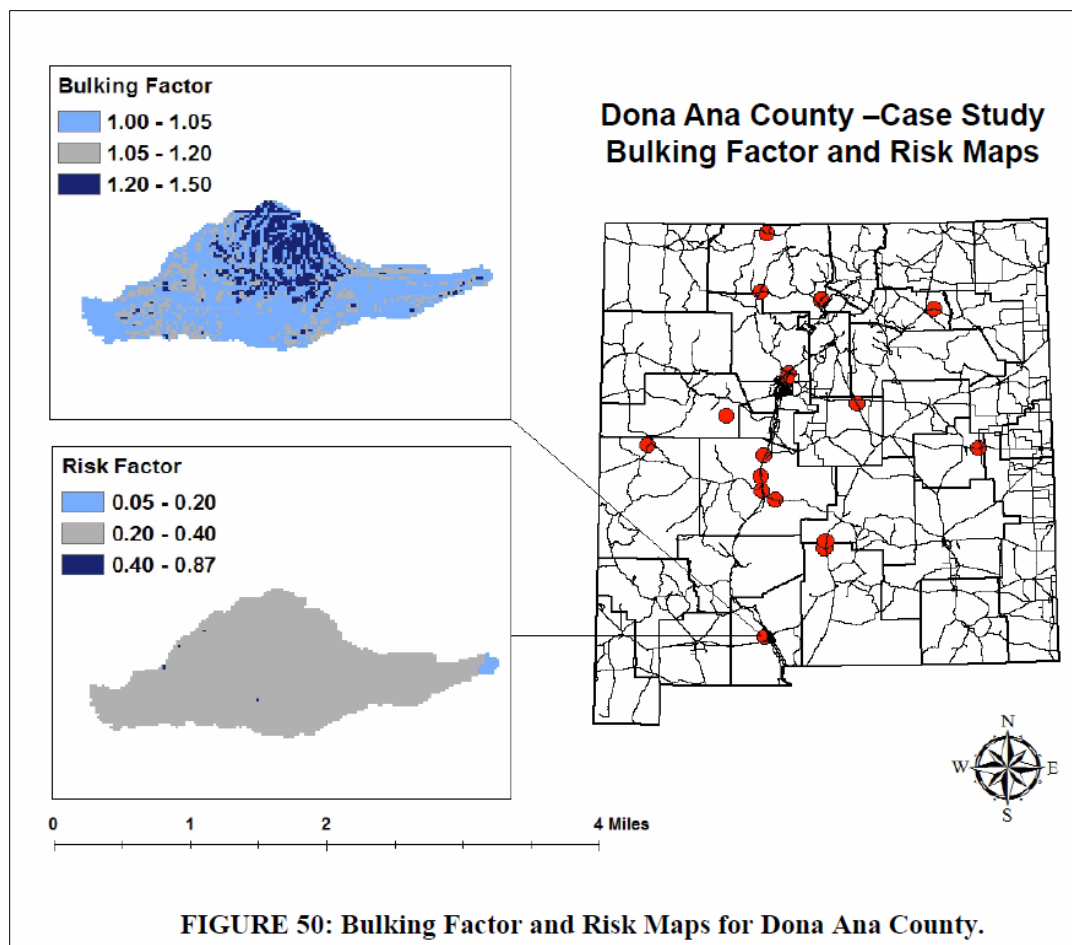


### 402.11.2 New Mexico Institute of Mining and Technology

The NMDOT previously contracted with New Mexico Institute of Mining and Technology (NMIMT) to study the sediment bulking issue in New Mexico streams and arroyos. The resulting study report “Development of Watercourse Aggradation/Degradation Risk Index for New Mexico,” May 2013, may be acquired from the NMDOT website at:

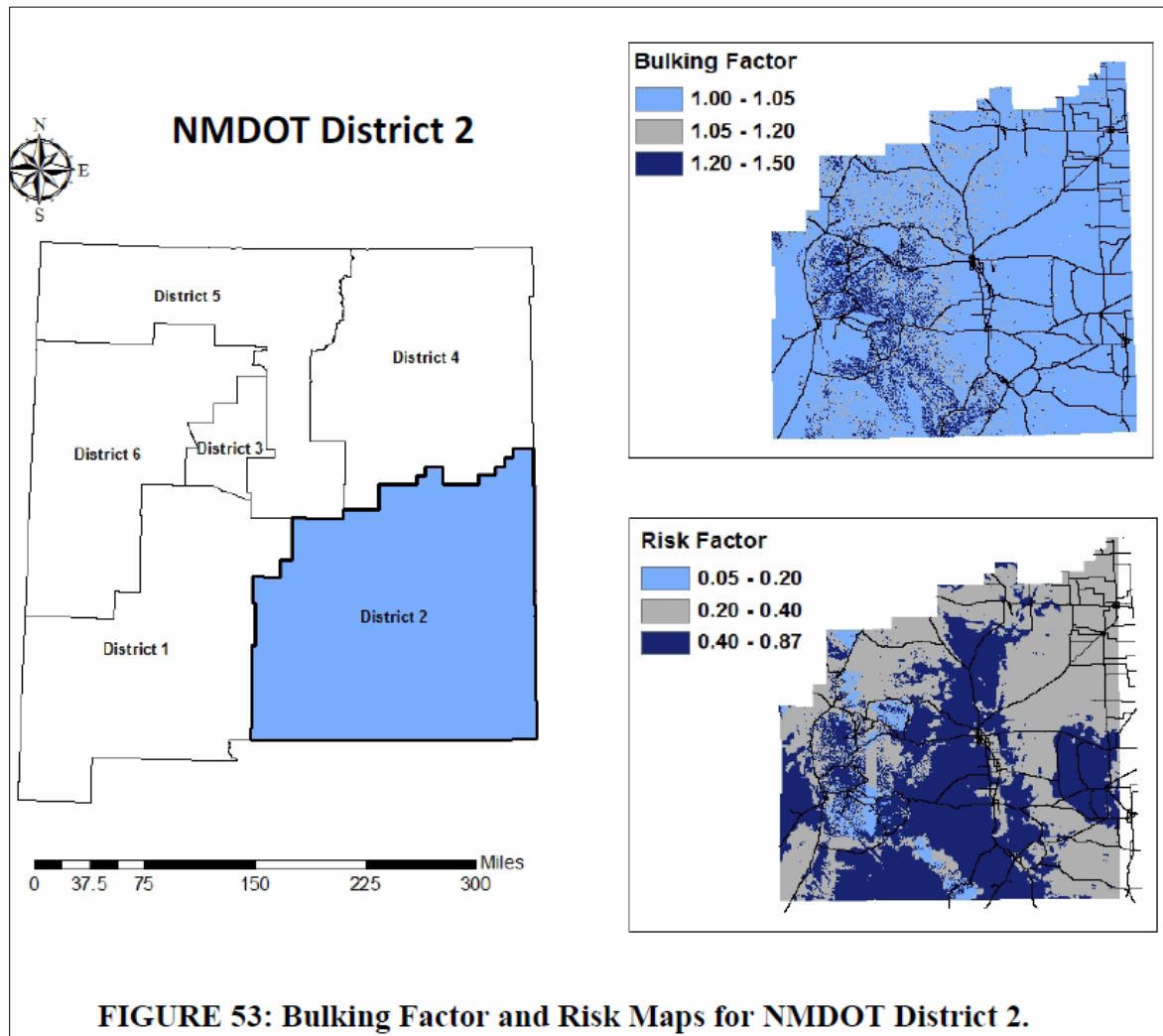
[http://dot.state.nm.us/content/dam/nmdot/Research/NM10DSN-01\\_Final\\_Report\\_Aggradation\\_Risk\\_with\\_Impl.pdf](http://dot.state.nm.us/content/dam/nmdot/Research/NM10DSN-01_Final_Report_Aggradation_Risk_with_Impl.pdf)

The NMIMT report provides estimates for sediment bulking factors and risk maps for selected New Mexico Watersheds and for each of the NMDOT Maintenance Districts. **Figure 402-20** and **Figure 402-21** are examples of the maps found in this report. The NMIMT figures illustrate bulking factors up to 1.50 for some areas. Note that a sediment bulking factor greater than about 1.25 would be considered mud flow based on the reference presented in the previous Section.



Source: New Mexico Institute of Mining and Technology, May 2013,  
Development of Watercourse Aggradation/Degradation Risk Index for New Mexico

**Figure 402-20 Bulking and Risk Map Example**



Source: New Mexico Institute of Mining and Technology, May 2013,  
Development of Watercourse Aggradation/Degradation Risk Index for New Mexico

#### Figure 402-21 District Bulking Factor and Risk Map Example

#### 402.11.3 Guidance on Sediment Bulking Factor Selection

Sediment bulking factor selection is subjective and is driven by the basin land use type and condition, and also by the drainage conveyance system type and condition. General guidance, questions and items to consider that contribute or not, to bulking factor selection follow.

- Is the basin 100% urbanized without any exposed soil areas or landscape areas that will general sediment? If so, this would imply a bulking factor of 1.0 (no sediment load) from the basin surface. However, then the drainage conveyance system must also be evaluated.
- If the basin is 100% urbanized, does the drainage conveyance system consist of only storm drains and hard lined channels, or are there also unlined watercourses? A system that is totally lined would imply that no sediment bulking factor would be required (factor

of 1.0). However, if the urbanized basin contains unlined areas and unlined channels, a sediment bulking factor would be required.

- Mountain forest basins in good condition, with rock channels will generally contribute very minor sediment loads. However, if the land has been overgrazed, damaged by logging operations, damaged by recreational vehicular traffic and related activities, or burned by fire, the sediment yield to the watercourse must be considered and will obviously increase the sediment bulking factor compared to a healthy forest.
- Rangeland basins in good condition will contribute minor sediment loads, and rangelands generally outfall to natural unlined watercourses. The composition of the watercourse must be considered (clays, sands, gravels, cobbles, boulders). A bulking factor will be required for rangeland basins and the magnitude of the factor will depend on the basin and watercourse conditions. However, if the land has been overgrazed, damaged by logging operations, damaged by recreational vehicular traffic and related activities, or burned by fire, excess sediment yield to the watercourse must be considered and will obviously increase the sediment bulking factor compared to a healthy rangeland.

#### **402.12 Rain on Snow**

Snowmelt runoff is a major component of the hydrologic cycle in some parts of New Mexico and can be an important consideration for design flood analysis. Heavy rainfall on snow can result in runoff events that are significantly larger than would otherwise result from either the rainfall event or snowmelt event alone. Consult the Drainage Design Bureau when the drainage analysis is in a watershed with the potential for significant snow accumulations. The NRCS provides good guidance in “Part 630 Hydrology, National Engineering Handbook”, Chapter 11 Snowmelt” and in “Chapter 18, Selected Statistical Methods”.

#### **402.13 Fire Related Impacts**

Increased risk of severe wildfires has become increasingly frequent in New Mexico and the Western U.S. and are currently an area of intense study by a variety of Federal and State agencies. Much literature has been produced in recent years due to the number, size, and severity of wildfires in the west in general and in and around New Mexico specifically. While at this time no dependable analysis tools are available for estimating the runoff from a severely burned watershed, it is clear that severe wildfires in a watershed can result in flood flows that are orders of magnitude higher than would have been expected prior to the fire. While it may be unfeasible to design a highway crossing for a flood that is 10 to 100 times larger than would have resulted from the standard design storm, consideration should be given with respect to the potential flood risk after a severe wildfire. NRCS and the U.S. Forest Service are expected to produce planning, analysis, and design documents in the near future addressing this issue. The hope is that these tools will assist in planning for and defending against large post-fire flood events. Consult with the Drainage Design Bureau for guidance when simulating burned watersheds.

In the interim, Ventura County in California has conducted studies, and developed guidance for estimating the impacts of flood flows after a severe wildfire. The study is titled “Sediment/Debris Bulking Factors and Post-Fire Hydrology for Ventura County, Final Report – June 2011”. (A hotlink is not available.)

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## 403 Rational Formula Method

Hydrologic analyses performed on small (<160 acre) watersheds will normally be performed using the Rational Formula Method. The Rational Formula Method is a widely and long accepted procedure worldwide for estimating peak rates of runoff from small watersheds. The Rational Formula Method may be used on NMDOT projects for roadway drainage facilities and small drainage structures as described in **Section 401 (Figure 401-1 and Figure 401-2)** of this manual. The standard form of the Rational Formula Equation in English units is:

$$Q = C i A$$

**403-1**

where:

Q	=	the peak rate of runoff, cfs
C	=	Runoff Coefficient
i	=	the rainfall intensity, in./hr
A	=	the watershed or drainage area, acres

The units in the Rational Formula do not yield peak discharge in cubic feet per second (cfs) directly, but rather are in acre-inches/hour. However, the conversion from acre-inches/hour to cfs is 1.008 which is commonly neglected because it does not introduce a significant error. The Rational Formula has several assumptions implicit to the method, including:

- The rainfall intensity is uniform for a duration equal to or greater than  $T_c$
- Peak flow occurs when the entire watershed is contributing runoff
- The frequency of the resulting peak discharge is equal to the frequency of the rainfall event.
- Both the Runoff Coefficient (C) and the rainfall intensity (i) vary with the return period (both tend to increase as return period increases). Therefore, both must be determined separately for each design storm frequency.
- The Runoff Coefficient (C) is dependent on the Hydrologic Soil Group (HSG) and the vegetative cover or in the case of developed watersheds, the percentage of impervious cover. HSGs are divided into four soil groups and are described in **Section 402.4**.

Limitations for using the Rational Formula Method on NMDOT projects include the following:

- The total drainage area should not exceed 160 acres
- Land use, slope, and soils are fairly consistent throughout the watershed
- There are no diversions, detention basins, pump stations, or other structures in the watershed which would require the routing of a flood hydrograph
- The Time of Concentration ( $T_c$ ) does not exceed one hour
- Runoff volumes may not be computed with the Rational Formula Method or Modified Rational Formula Method (not included in this Drainage Design Manual)

### 403.1 Time of Concentration ( $T_c$ ) for Use in the Rational Formula Method

The assumptions within the Rational Formula Method are that the rainfall intensity is uniform for a duration equal to or greater than  $T_c$  and that the entire watershed is contributing runoff when the peak occurs. Therefore, in order to determine the appropriate rainfall intensity “i” for the

watershed, the  $T_c$  must be determined. For NMDOT projects,  $T_c$  shall be calculated using the Kirpich Equation or Upland Method depending on specific circumstances.

The Upland Method was originally developed by the Soil Conservation Service (SCS), which is now the Natural Resources Conservation Service (NRCS). The Upland Method is described in Chapter 15 Time of Concentration of “Part 630 Hydrology, National Engineering Handbook” (NRCS, 2010). Note that in the current (2010) version of Chapter 15, the NRCS has renamed the “Upland Method” to the “Velocity Method.” However, many documents still refer to it as the “Upland Method” and, therefore, the name “Upland Method” is used in this Drainage Design Manual.

The Upland Method is used to estimate travel times for overland flow and shallow concentrated flow conditions. The Upland Method is limited to use in watersheds less than 2000 acres in size, or to the upper reaches of larger watersheds. For NMDOT projects, the Upland Method may be used for computing the  $T_c$  when using the Rational Formula Method or the Simplified Peak Discharge Method on an **un-gullied** watershed. The use of Upland Method is described in **Section 402.9.1**.

When using the Rational Formula, the Kirpich Equation should be used in watersheds **when gullyng is evident in more than 10% of the primary watercourse**. Gullyng can be assumed if a blue line appears on the watercourse shown on the USGS quadrangle topographic map or is apparent from field reconnaissance or from inspection of aerial photography. The Kirpich Equation is given as:

$$T_c = 0.0078 L^{0.77} S^{-0.385} \quad 403-2$$

(TxDOT, July 2016, “Hydraulic Design Manual,” Eq. 4-15, p. 4-39)

<http://onlinemanuals.txdot.gov/txdotmanuals/hyd/index.htm>

where:

$T_c$	=	Time of Concentration, minutes
$L$	=	maximum length of water travel, ft
$S$	=	surface slope, given by $H/L$ , ft/ft
$H$	=	difference in elevation between the most hydraulically remote point in the drainage basin and the outlet, ft

In small watersheds where the slope is very flat, and the flow path of the hydraulically longest flow path is dominated by overland flow (> 300 ft), the Kerby Equation should be considered for the overland flow portion and Kirpich Equation for the channelized portion.

For small watersheds where overland flow is an important component of overall travel time, the Kerby Equation can be used. The Kerby Equation is:

$$T_{OV} = K (L N)^{0.467} S^{-0.235} \quad 403-3$$

(TxDOT, July 2016, “Hydraulic Design Manual,” Eq. 4-14, p. 4-37)

<http://onlinemanuals.txdot.gov/txdotmanuals/hyd/index.htm>



where:

$T_{ov}$	=	overland flow Time of Concentration, minutes
$K$	=	$K = 0.828$ , a unit conversion factor
$L$	=	the overland-flow length, ft
$N$	=	a dimensionless retardance coefficient
$S$	=	the dimensionless slope of terrain conveying the overland flow

In the development of the Kerby Equation, the length of overland flow was as much as 1,200 feet. Hence, this length is considered an upper limit, and in practice, shorter values generally are expected. The dimensionless retardance coefficient used is similar in concept to the well-known Manning's roughness coefficient; however, for a given type of surface, the retardance coefficient for overland flow will be considerably larger than for open-channel flow. Typical values for the retardance coefficient are listed in **Table 402-9**. Roussel et al. (2005), recommends that the user should not interpolate the retardance coefficients shown in **Table 402-9**. If it is determined that a low slope condition or a transitional slope condition exists, the user should consider using an adjusted slope in calculating the  $T_c$ .

### Time of Concentration with the Kerby-Kirpich Method

When the Kirpich Equation result and the Kerby Equation result are combined, it is referred to as the Kerby-Kirpich Method. The watershed should be divided between the channelized reach and the overland flow reach and the travel time across each reach calculated and combined to compute the total  $T_c$ .

- If the calculations (with either Kirpich Equation or with the Kerby Equation) yield a  $T_c$  less than 10 minutes, use 10 minutes
- If the resulting  $T_c$  is greater than 1 hour, do not use the Rational Formula Method, select another hydrologic analysis method

## 403.2 Rainfall

When developing Intensity-Duration-Frequency (IDF) curves and Depth-Duration (DD) values for Rational Formula Method from NOAA Precipitation Frequency Data Server (PFDS), the following approach is provided to develop the IDF curves, from which the rainfall intensity “ $i$ ” is derived for the design frequency storm required.

1. Go to NOAA Precipitation Frequency Data Server (PFDS)  
[http://hdsc.nws.noaa.gov/hdsc/pfds/pfds\\_map\\_cont.html?bkmrk=nm](http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=nm)
  - a. Click on New Mexico on the Map
  - b. Data Description – use defaults
  - c. Get Location Options
    - i. Use navigation tools to either:
      1. Enter latitude and longitude or
      2. Select Station or
      3. Selection Location on map
  - d. Data Description
    - i. Data Type: Select “precipitation intensity”

- ii. Units: Select “*English*”
  - iii. Time series type: Select “***partial duration***”
- e. Scroll down to Depth-Duration-Frequency table below map
- f. Scroll to bottom of table and in the “Estimates from the table in csv format” box select “***precipitation frequency estimates***”
- g. Open in MS Excel and do a “save as” to your workspace as a .txt file
- h. Open .txt file (it should open in Excel)
- i. Insert Chart into the Excel spreadsheet (see **Table 403-1** example spreadsheet below)
  - i. Insert a column adjacent to the durations and fill in with time values  
(Excel doesn't recognize “5-min” as a value)
  - ii. Select X Y Scatter Chart Type
  - iii. Select Data with duration (in minutes) on the x axis, intensity (in./hr) on the y axis for each frequency (1-year, 2-year, 5-year, 10-year, 25-year, 50-year, 100-year) as needed for project analyses. (See **Table 403-1**)
- j. Format x axis to allow reading duration in 1 minute increments and y axis to read intensity in 0.1 in./hour increments. (See **Figure 403-1**)
- k. Read rainfall intensity that matches basin Tc for the storm frequency required.
- l. **Minimum Tc = 10 minutes for this purpose!**

**Table 403-1 NOAA Data Server Sample IDF Spreadsheet-Lemitar NM**

Point precipitation frequency estimates (inches/hour)

NOAA Atlas 14 Volume 1 Version 5

Data type: Precipitation intensity

Time series type: Partial duration

Project area: Southwest

Location name: Lemitar, New Mexico, US\*

Station Name: -

Latitude: 34.1580°

Longitude: -106.9181°

Elevation: 4712 ft\*

\* source: Google Maps

**PRECIPITATION FREQUENCY ESTIMATES**

by

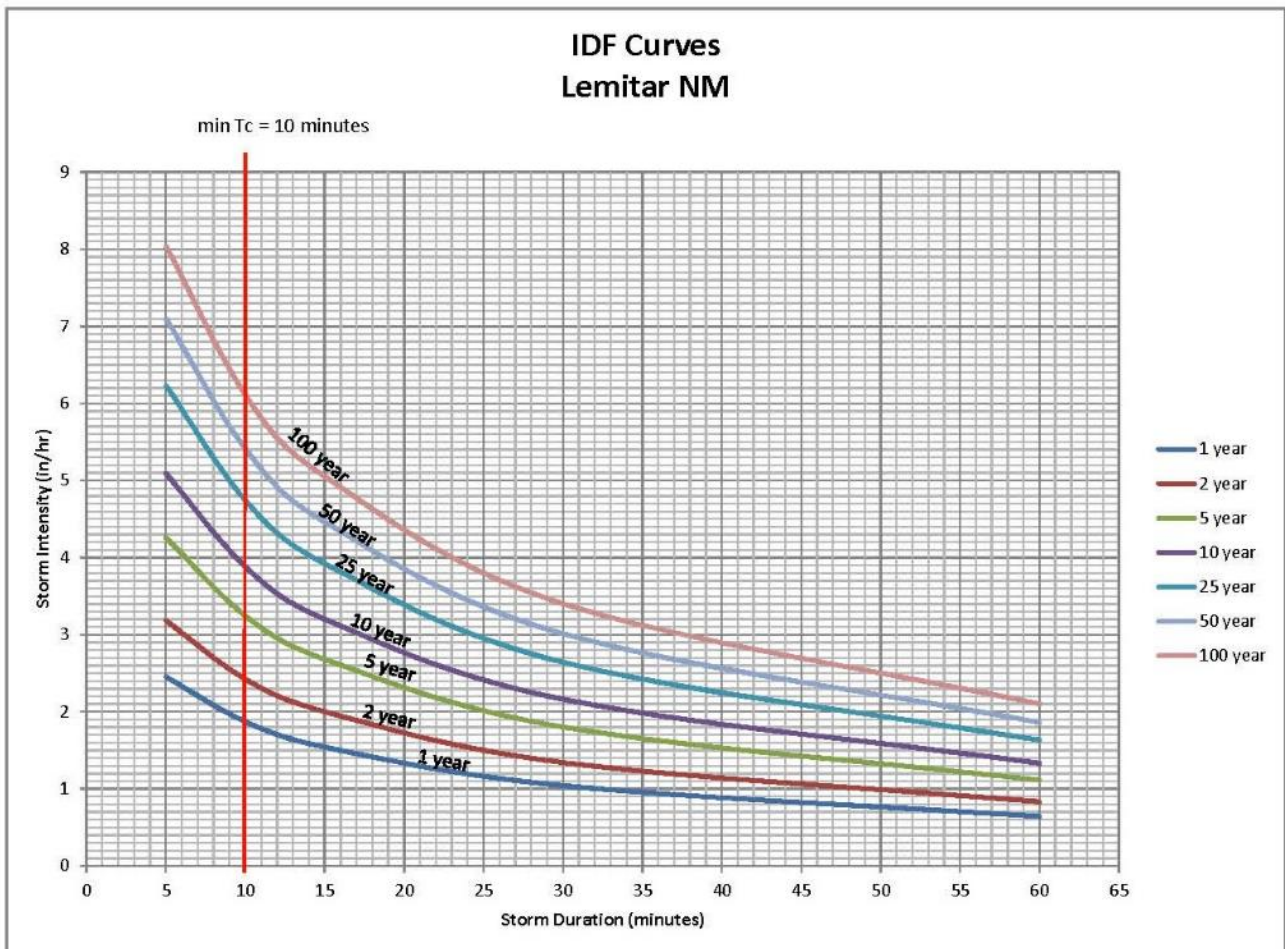
duration

for ARI:

	1	2	5	10	25	50	100	200	500	1000 years
5-min:	5	2.45	3.18	4.26	5.09	6.23	7.1	8.04	9	10.31 11.35
10-min:	10	1.87	2.42	3.24	3.88	4.74	5.41	6.11	6.85	7.84 8.64
15-min:	15	1.54	2	2.68	3.2	3.92	4.46	5.05	5.66	6.48 7.14
30-min:	30	1.04	1.34	1.8	2.16	2.64	3.01	3.4	3.81	4.36 4.81
60-min:	60	0.64	0.83	1.11	1.33	1.63	1.86	2.1	2.36	2.7 2.98
2-hr:		0.37	0.48	0.64	0.76	0.95	1.11	1.29	1.49	1.8 2.06
3-hr:		0.27	0.34	0.45	0.54	0.67	0.78	0.9	1.04	1.25 1.43
6-hr:		0.16	0.2	0.25	0.3	0.36	0.42	0.48	0.55	0.66 0.75
12-hr:		0.08	0.11	0.13	0.16	0.19	0.22	0.25	0.29	0.34 0.38
24-hr:		0.05	0.06	0.08	0.09	0.11	0.12	0.14	0.16	0.18 0.2
2-day:		0.03	0.03	0.04	0.05	0.06	0.07	0.07	0.08	0.1 0.11
3-day:		0.02	0.02	0.03	0.03	0.04	0.05	0.05	0.06	0.07 0.08
4-day:		0.02	0.02	0.02	0.03	0.03	0.04	0.04	0.05	0.05 0.06
7-day:		0.01	0.01	0.02	0.02	0.02	0.02	0.03	0.03	0.03 0.04
10-day:		0.01	0.01	0.01	0.01	0.02	0.02	0.02	0.02	0.03 0.03
20-day:		0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02 0.02
30-day:		0	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01 0.01
45-day:		0	0	0	0.01	0.01	0.01	0.01	0.01	0.01 0.01
60-day:		0	0	0	0	0.01	0.01	0.01	0.01	0.01 0.01

Date/time (GMT): Fri Nov 13 22:14:03 2015

pyRunTime: 0.127875804901



**Figure 403-1 IDF Curves from NOAA Data Server-Lemitar, NM**

To produce the Depth-Duration 1-hour precipitation values for use in determining the Rational Formula Runoff Coefficient “C”, return to the NOAA Data Server for the same location as for the IDF Curve development (see **Table 403-2** from NOAA Data Server)

[http://hdsc.nws.noaa.gov/hdsc/pfds/pfds\\_map\\_cont.html?bkmrk=nm](http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=nm)

**Table 403-2 Depth-Duration-Frequency Table from NOAA Data Server**

[http://hdsc.nws.noaa.gov/hdsc/pfds/pfds\\_map\\_cont.html?bkmrk=nm](http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=nm)

Point precipitation frequency estimates (inches)										
NOAA Atlas 14 Volume 1 Version 5										
Data type: Precipitation depth										
Time series type: Partial duration										
Project area: Southwest										
Location name: Lemitar, New Mexico, US*										
Station Name: -										
Latitude: 34.1584°										
Longitude: -106.9189°										
Elevation: 4713 ft*										
* source: Google Maps										
PRECIPITATION FREQUENCY ESTIMATES										
by duration	1	2	5	10	25	50	100	200	500	1000 years
5-min:	0.2	0.27	0.35	0.42	0.52	0.59	0.67	0.75	0.86	0.95
10-min:	0.31	0.4	0.54	0.65	0.79	0.9	1.02	1.14	1.31	1.44
15-min:	0.39	0.5	0.67	0.8	0.98	1.12	1.26	1.41	1.62	1.78
30-min:	0.52	0.67	0.9	1.08	1.32	1.5	1.7	1.9	2.18	2.4
60-min:	0.64	0.83	1.11	1.33	1.63	1.86	2.1	2.36	2.7	2.98
2-hr:	0.75	0.96	1.27	1.52	1.9	2.22	2.58	2.98	3.59	4.13
3-hr:	0.81	1.03	1.35	1.61	2	2.33	2.7	3.12	3.75	4.3
6-hr:	0.93	1.18	1.51	1.78	2.18	2.52	2.9	3.31	3.93	4.48
12-hr:	1.01	1.28	1.63	1.91	2.31	2.65	3.03	3.44	4.06	4.59
24-hr:	1.16	1.45	1.82	2.12	2.55	2.9	3.29	3.72	4.35	4.88
2-day:	1.27	1.59	1.98	2.3	2.76	3.13	3.54	3.99	4.64	5.2
3-day:	1.36	1.7	2.12	2.46	2.94	3.34	3.78	4.25	4.95	5.55
4-day:	1.45	1.81	2.25	2.61	3.12	3.55	4.01	4.51	5.25	5.89
7-day:	1.67	2.08	2.57	2.96	3.52	3.97	4.46	4.99	5.77	6.41
10-day:	1.84	2.3	2.84	3.29	3.91	4.41	4.96	5.56	6.43	7.17
20-day:	2.33	2.9	3.54	4.03	4.71	5.25	5.81	6.39	7.2	7.89
30-day:	2.81	3.5	4.23	4.78	5.53	6.11	6.7	7.3	8.12	8.81
45-day:	3.41	4.23	5.08	5.7	6.51	7.12	7.71	8.29	9.11	9.78
60-day:	3.9	4.84	5.8	6.52	7.44	8.13	8.81	9.47	10.33	10.98
Date/time (GMT): Mon Nov 16 19:12:46 2015										
pyRunTime: 0.126244068146										

#### Procedure:

1. Data Description
  - a. Data Type: Select "**precipitation depth**"
  - b. Units: Select "**english**"
  - c. Time series type: Select "**partial duration**"
2. Scroll down to Depth-Duration-Frequency table below map
3. Scroll to bottom of table and in the "Estimates from the table in csv format" box select "**precipitation frequency estimates**"
4. Open in MS Excel and do a "save as" to your workspace as a .txt file
5. Open .txt file (it should open in Excel) **Table 403-2**
6. Read point rainfall value for 1-hour design storm

### 403.3 Rational Formula Runoff Coefficient "C"

The Rational Formula Runoff Coefficient, "C" should be selected from **Figure 403-2** to **Figure 403-7** depending on the ground cover, Hydrologic Soil Group, type of development, and 1-hour rainfall depth for the design return period. The Runoff Coefficient "C" figures are adopted from

the Arizona DOT Drainage Design Manual due to the similarities in climate, soils, vegetation and terrain between Arizona and New Mexico.

[http://www.azdot.gov/docs/default-source/roadway-engineering-library/2014\\_adot\\_hydrology\\_manual.pdf?sfvrsn=16](http://www.azdot.gov/docs/default-source/roadway-engineering-library/2014_adot_hydrology_manual.pdf?sfvrsn=16)

Hydrologic Soil Groups are defined in **Section 402.4**. **Figure 403-2** to **Figure 403-7** show how “C” varies with 1-hour rainfall depth. This is because “C” is a function of infiltration and other hydrologic abstractions, relating the peak discharge to the theoretical peak discharge produced by 100% runoff.

Engineers are encouraged to review the supporting information provided in the Arizona manual before using these figures in order to familiarize themselves with their limitations and assumptions. When land use or other factors vary significantly throughout the watershed, an area weighted “C” value should be used. The weighted “C” value is computed by the equation:

$$\text{Weighted C} = \frac{C_1 A_1 + C_2 A_2 + C_3 A_3 \dots}{\sum A} \quad \mathbf{403-4}$$

(Arizona Department of Transportation, 2014, “Highway Drainage Design Manual, Volume 2, Hydrology, Second Edition”, Eq. 2.5, p. 2-7)

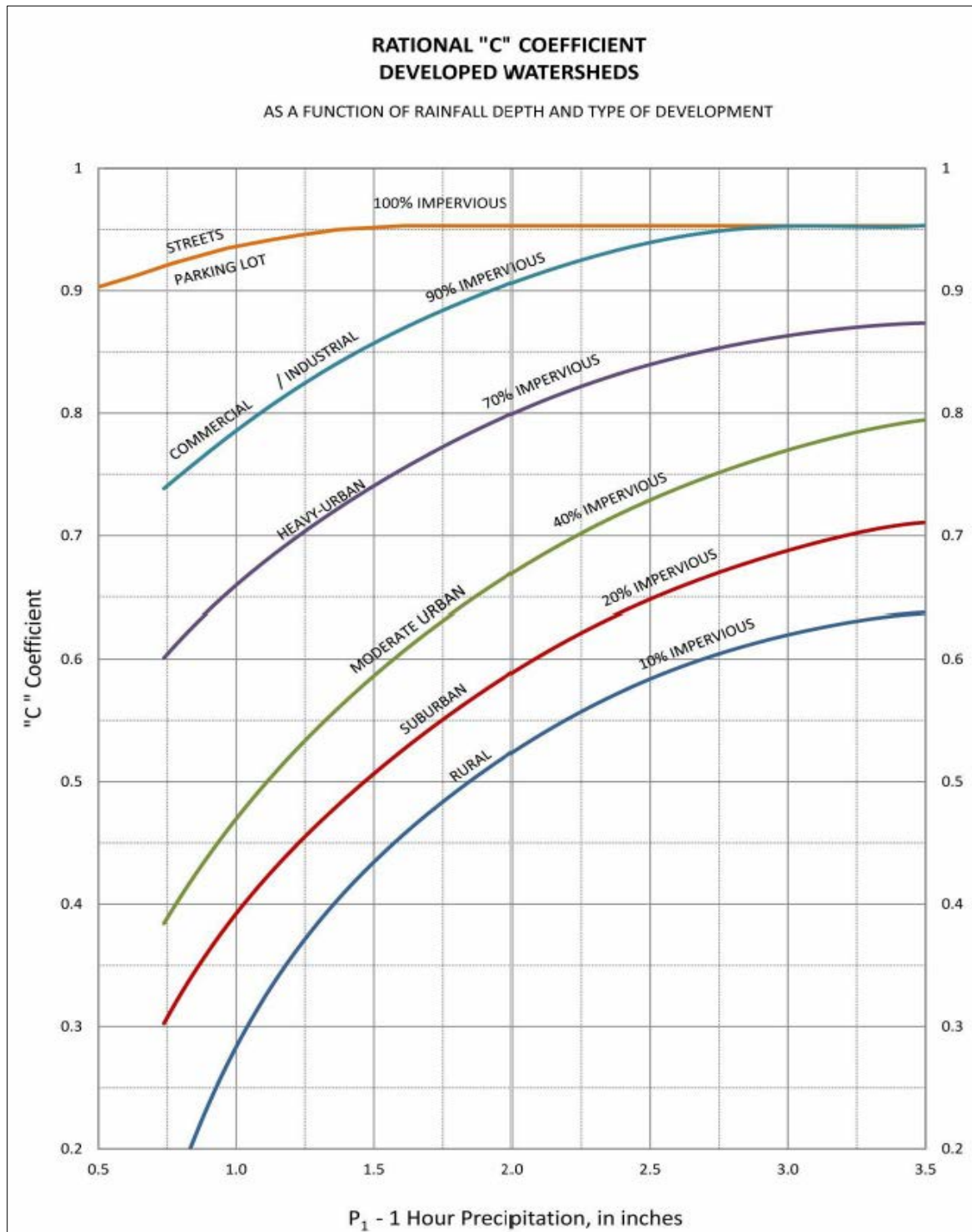
[http://www.azdot.gov/docs/default-source/roadway-engineering-library/2014\\_adot\\_hydrology\\_manual.pdf?sfvrsn=16](http://www.azdot.gov/docs/default-source/roadway-engineering-library/2014_adot_hydrology_manual.pdf?sfvrsn=16)

where:

C1	=	“C” Runoff Coefficient for subbasin(s) 1, etc.
A1	=	area of subbasin(s) 1, etc., acres
ΣA	=	total basin area, acres

The designer should select the appropriate **Figure 403-2** to **Figure 403-7**, depending on the watershed location (desert, upland range, mountain or urban) and the predominant vegetation type (cactus, brush, grasses, juniper, pine). Enter the appropriate Figure with the design 1-hour rainfall depth. Move vertically up through the Figure until the appropriate curve is found, then move horizontally to find the design “C” value. The appropriate curve is selected based on the Hydrologic Soil Group (HSG) and the percent ground cover of the vegetation or percent imperviousness. When a value falls between two curves, interpolate linearly between the two nearest curves to the required percentage of cover or imperviousness.

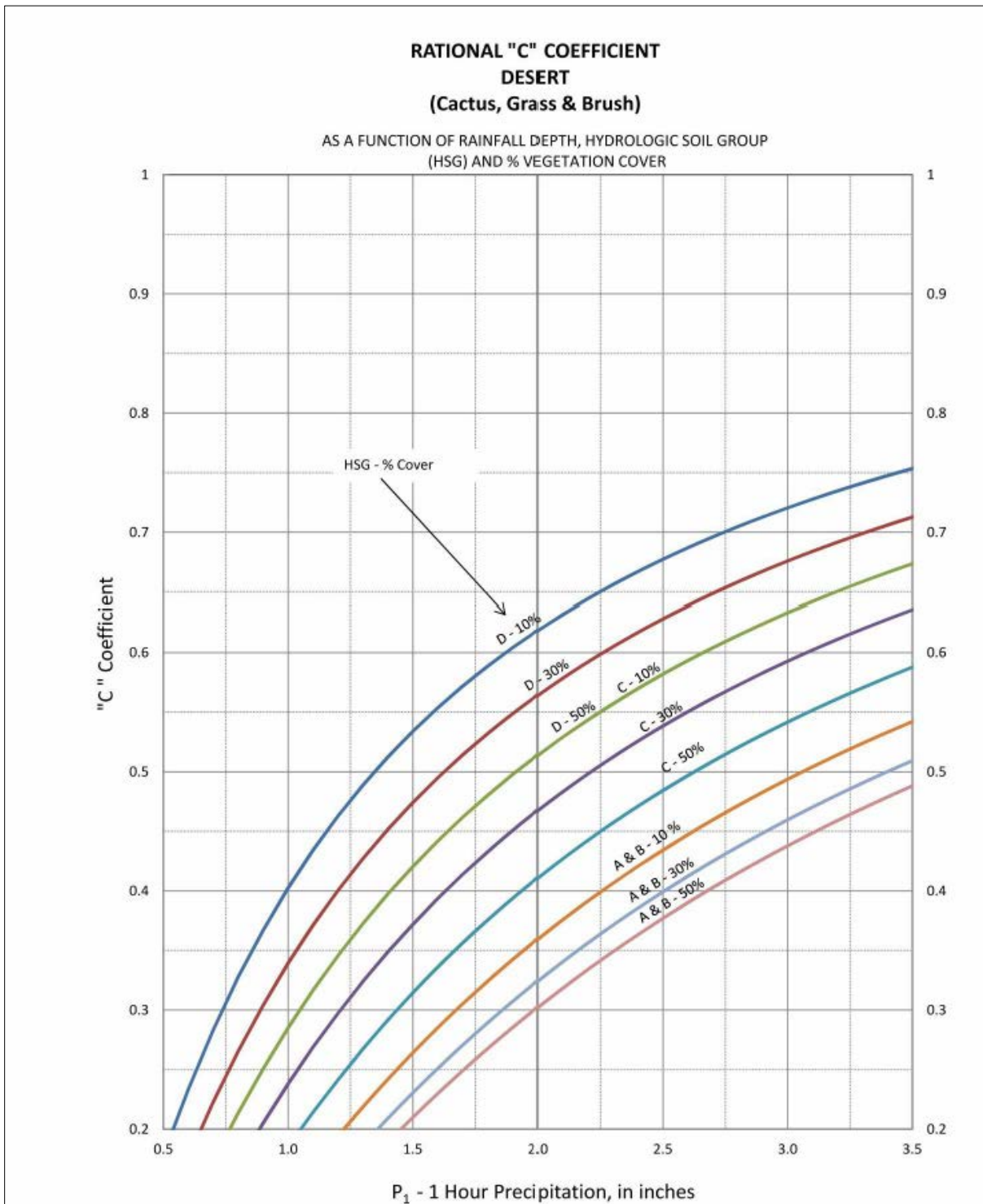




Source: Arizona Department of Transportation, 2014, "Highway Drainage Design Manual, Volume 2, Hydrology, Second Edition", Figure 2-1, p. 2-8.

[http://www.azdot.gov/docs/default-source/roadway-engineering-library/2014\\_azdot\\_hydrology\\_manual.pdf?sfvrsn=16](http://www.azdot.gov/docs/default-source/roadway-engineering-library/2014_azdot_hydrology_manual.pdf?sfvrsn=16)

**Figure 403-2 Rational "C" Coefficient Developed Watersheds**

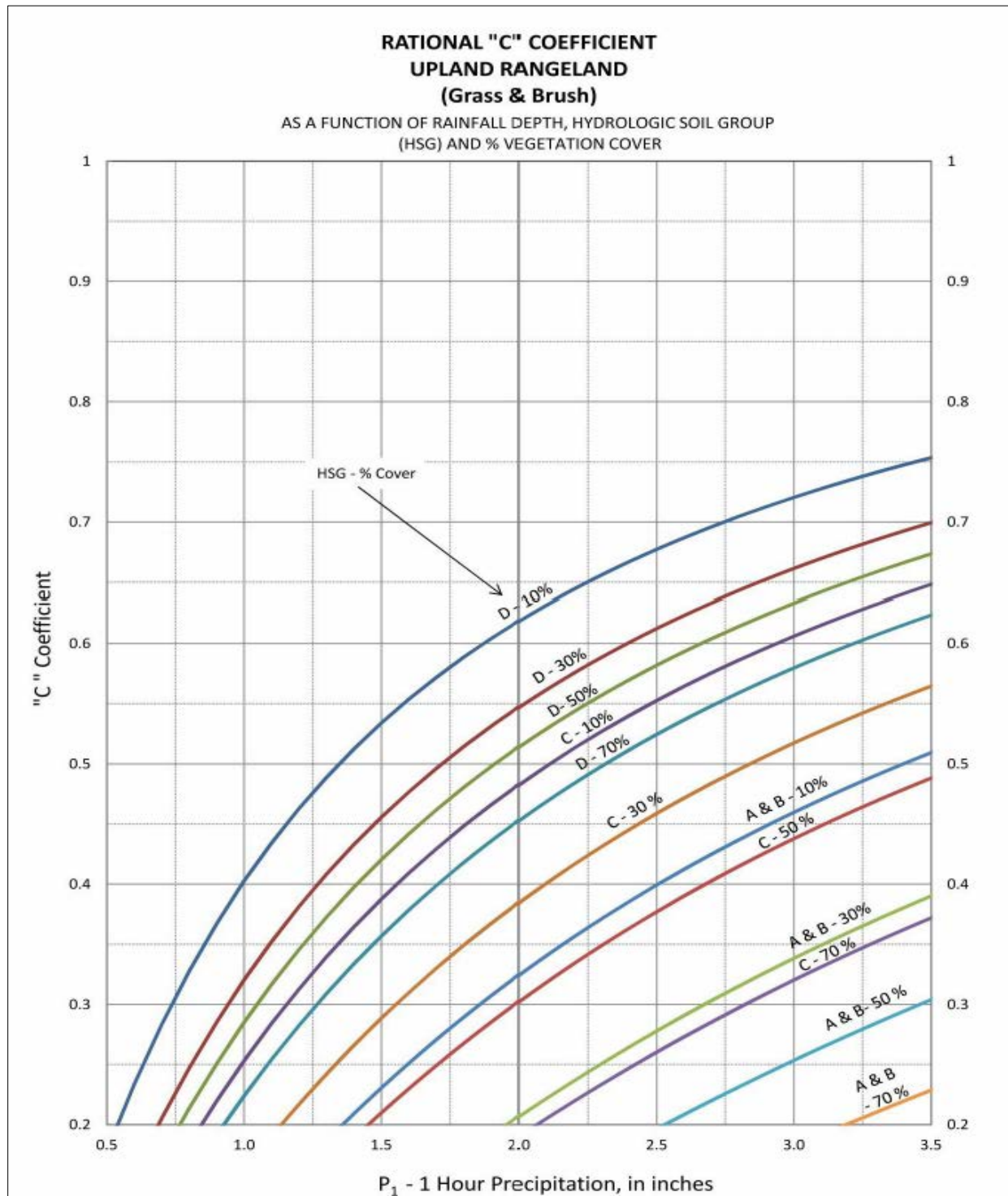


Source: Arizona Department of Transportation, 2014, "Highway Drainage Design Manual, Volume 2, Hydrology, Second Edition", Figure 2-2, p. 2-9.

[http://www.azdot.gov/docs/default-source/roadway-engineering-library/2014\\_adot\\_hydrology\\_manual.pdf?sfvrsn=16](http://www.azdot.gov/docs/default-source/roadway-engineering-library/2014_adot_hydrology_manual.pdf?sfvrsn=16)

**Figure 403-3 Rational "C" Coefficient Desert (Cactus, Grass & Brush)**

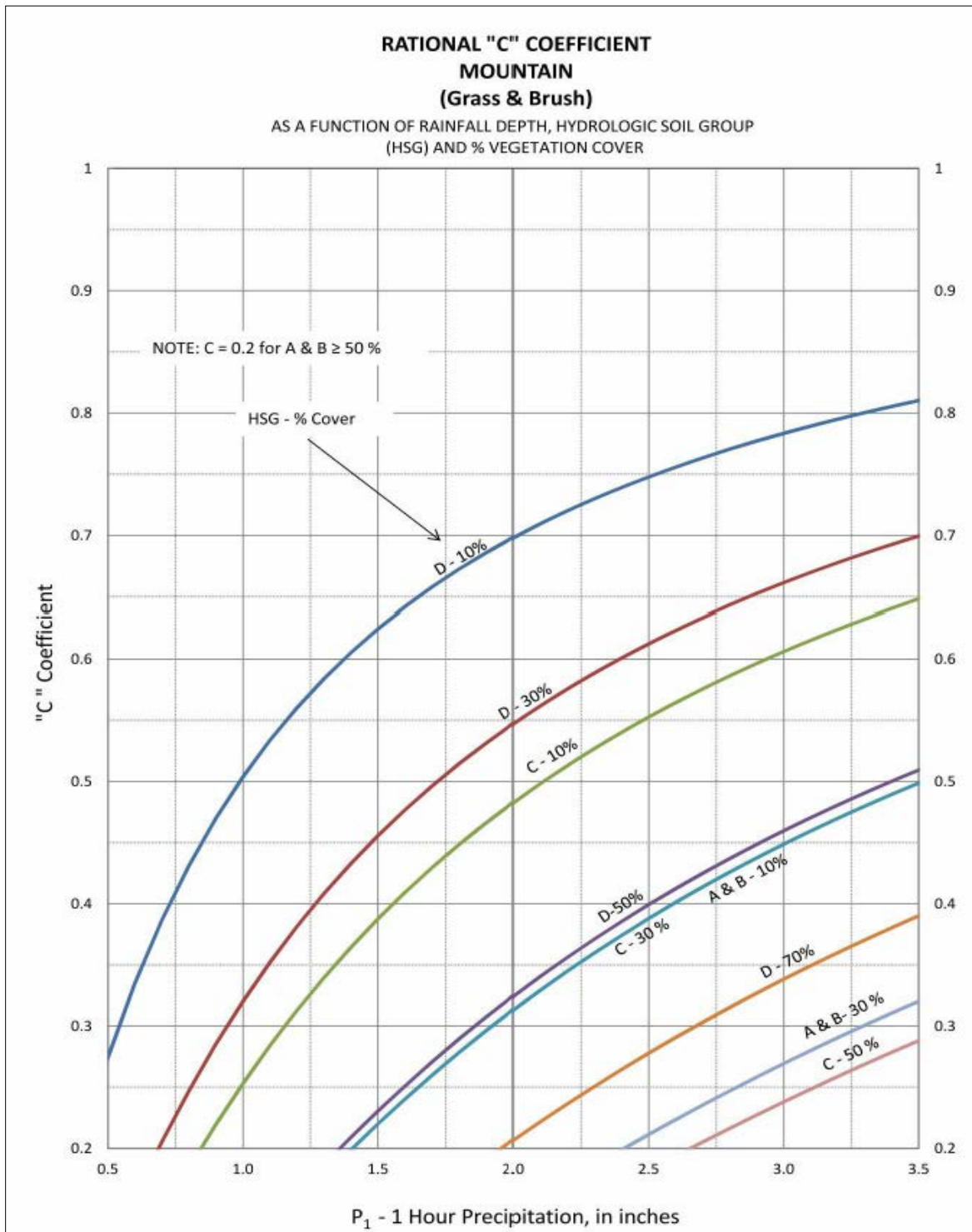




Source: Arizona Department of Transportation, 2014, "Highway Drainage Design Manual, Volume 2, Hydrology, Second Edition", Figure 2-3, p. 2-10.

[http://www.azdot.gov/docs/default-source/roadway-engineering-library/2014\\_adot\\_hydrology\\_manual.pdf?sfvrsn=16](http://www.azdot.gov/docs/default-source/roadway-engineering-library/2014_adot_hydrology_manual.pdf?sfvrsn=16)

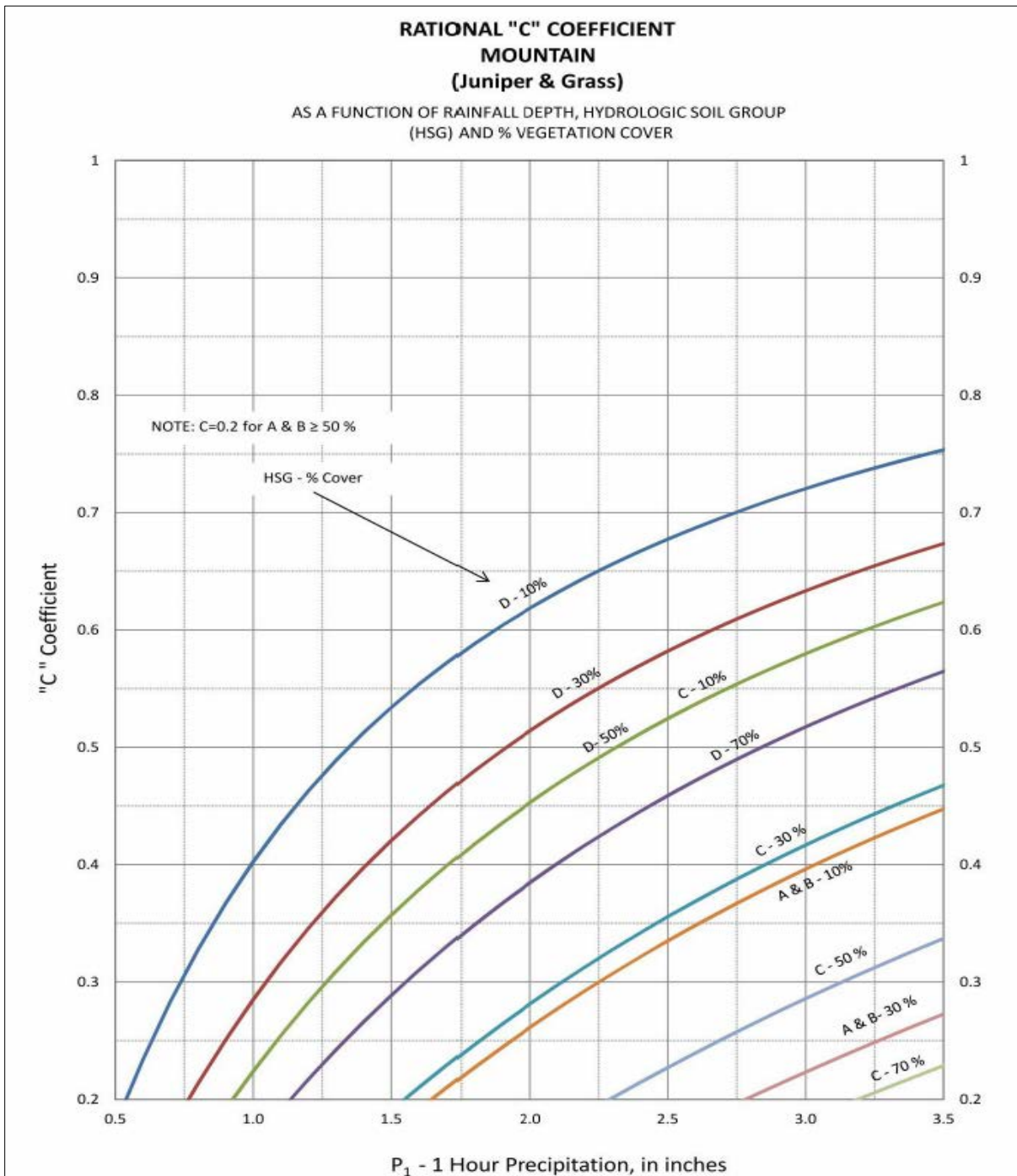
**Figure 403-4 Rational "C" Coefficient Upland Rangeland (Grass & Brush)**



Source: Arizona Department of Transportation, 2014, "Highway Drainage Design Manual, Volume 2, Hydrology, Second Edition", Figure 2-4, p. 2-11.

[http://www.azdot.gov/docs/default-source/roadway-engineering-library/2014\\_adot\\_hydrology\\_manual.pdf?sfvrsn=16](http://www.azdot.gov/docs/default-source/roadway-engineering-library/2014_adot_hydrology_manual.pdf?sfvrsn=16)

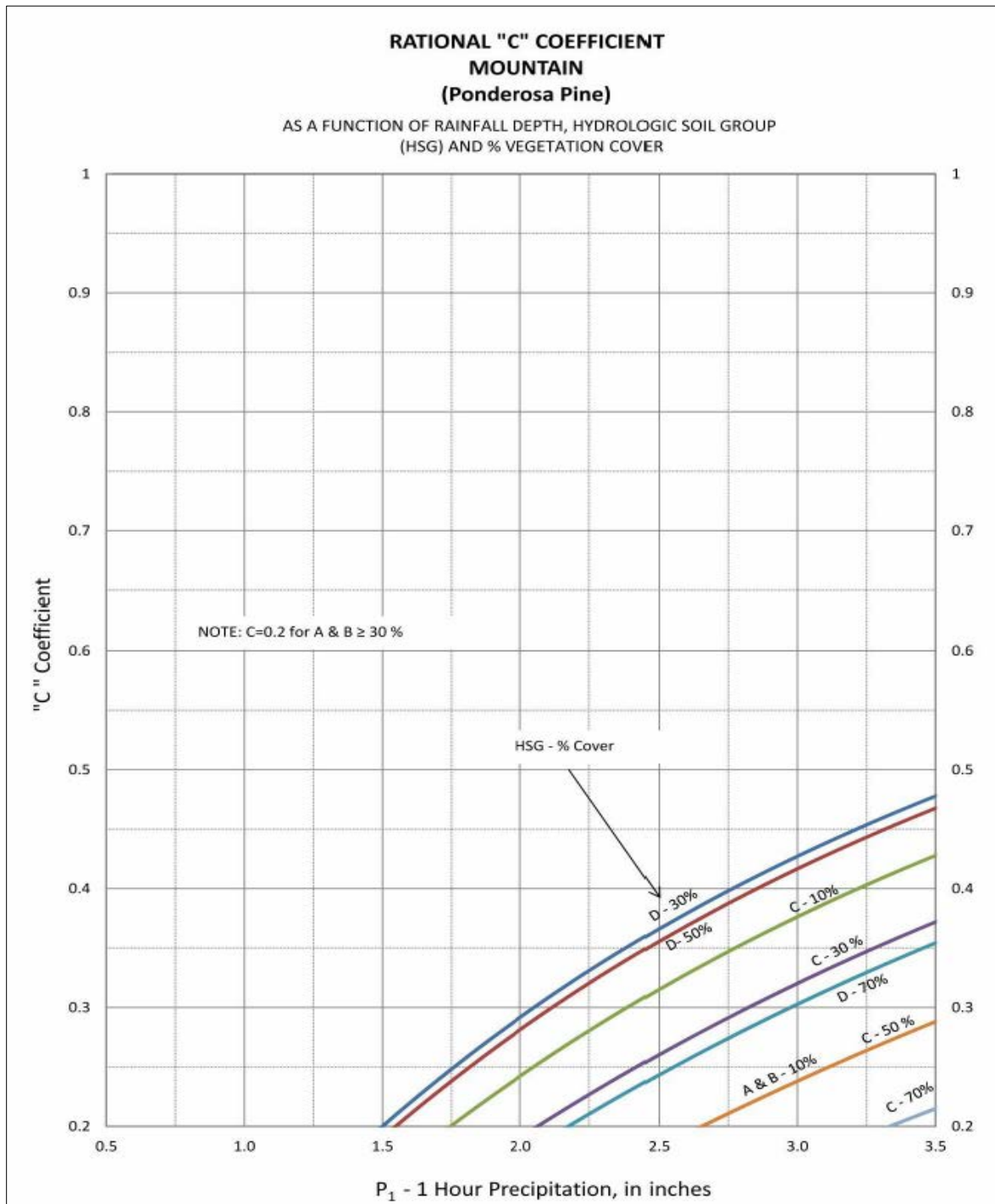
**Figure 403-5 Rational "C" Coefficient Mountain (Grass and Brush)**



Source: Arizona Department of Transportation, 2014, "Highway Drainage Design Manual, Volume 2, Hydrology, Second Edition", Figure 2-5, p. 2-12.

[http://www.azdot.gov/docs/default-source/roadway-engineering-library/2014\\_adot\\_hydrology\\_manual.pdf?sfvrsn=16](http://www.azdot.gov/docs/default-source/roadway-engineering-library/2014_adot_hydrology_manual.pdf?sfvrsn=16)

**Figure 403-6 Rational "C" Coefficient Mountain (Pinion, Juniper & Grass)**



Source: Arizona Department of Transportation, 2014, "Highway Drainage Design Manual, Volume 2, Hydrology, Second Edition", Figure 2-6, p. 2-13.

[http://www.azdot.gov/docs/default-source/roadway-engineering-library/2014\\_adot\\_hydrology\\_manual.pdf?sfvrsn=16](http://www.azdot.gov/docs/default-source/roadway-engineering-library/2014_adot_hydrology_manual.pdf?sfvrsn=16)

**Figure 403-7 Rational "C" Coefficient Mountain (Ponderosa)**

Appendix 6 contains **Example Problem 6-1** and **Example Problem 6-2**.

**Example Problem 6-1** and is a smaller site (34 acres) with 55% imperviousness located in central New Mexico. **Example Problem 6-2** is larger site (80 acres) with a more natural basin the demonstrates an area weighted Runoff Coefficient “C” calculation.

#### 403.4 References

Arizona Department of Transportation, 2014, “Highway Drainage Design Manual, Volume 2, Hydrology, Second Edition”.

[http://www.azdot.gov/docs/default-source/roadway-engineering-library/2014\\_adot\\_hydrology\\_manual.pdf?sfvrsn=16](http://www.azdot.gov/docs/default-source/roadway-engineering-library/2014_adot_hydrology_manual.pdf?sfvrsn=16)

NOAA Hydrometeorological Design Studies Center Precipitation Frequency Data Server. (PFDS).

<http://hdsc.nws.noaa.gov/hdsc/pfds/>

NRCS, “Part 630 Hydrology, National Engineering Handbook”. Note that various Chapters have different dates.

[https://www.nrcs.usda.gov/wps/portal/nrcs/detailfull/national/water/manage/hydrology/?cid=stelp\\_rdb1043063](https://www.nrcs.usda.gov/wps/portal/nrcs/detailfull/national/water/manage/hydrology/?cid=stelp_rdb1043063)

NRCS, 2010, “Part 630 Hydrology, National Engineering Handbook, Chapter 15 Time of Concentration”.

<http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=27002.wba>

Roussel, M.C., Asquith, W. H., Thompson, D. B., Cleveland, T. G., and Fang, X., 2005, “Summary of Dimensionless Texas Hyetographs and Distribution of Storm Depth, Developed for Texas Department of Transportation Research Project 0-4194”, U.S. Geological Survey, Austin, TX. (TxDOT 0-4194-4).

<http://library.ctr.utexas.edu/digitized/texasarchive/phase1/4194-4-TxDOT.pdf>

TxDOT, July 2016, “Hydraulic Design Manual,” Chapter 4, Section 11.

<http://onlinemanuals.txdot.gov/txdotmanuals/hyd/index.htm>



## 404 NRCS Simplified Peak Discharge Method

### 404.1 General

The NRCS Simplified Peak Discharge Method estimates the peak rate of runoff and runoff volume from small to medium size watersheds ( $\leq 10$  square miles). This method was developed by the Soil Conservation Service (now the NRCS) for use in New Mexico, and was originally developed in October 1973. This document was revised in 1985 titled "Peak Rates of Discharge for Small Watersheds, Chapter 2, Engineering Field Manual for Conservation Practices", SCS, February 1985. **APPENDIX 5** contains a copy that document. In April 2014, Supplemental Notice No. NM-36 was developed as a modification to the 1985 document. NM-36 only prescribed to replace the previous document (1985) rainfall data with NOAA Atlas 14 rainfall data.

The original Chapter 2 method (SCS, 1973) included unit peak discharge curves for different rainfall distributions, varying from 45% to 85% of the rainfall occurring in the peak hour.

After analysis of stream gage data, the 1985 update included only one peak discharge curve, representing a variable rainfall distribution depending on the  $T_c$  of the watershed. This curve is shown in **Figure 404-1**. Therefore, a separate estimate of rainfall distribution is not required to use this method. The analysis of gage data also showed that the method overestimated peak discharges at elevations above 7500 ft. Drainage structures above this elevation should be evaluated by the Unit Hydrograph Method (**Section 405**). The completion of the "Simplified Peak Discharge Method Worksheet" (**Figure 404-2**) is required when using this method. The NOAA Atlas 14 references and links are provided here.

NOAA, Rev. ed. 2011, "Atlas 14, Precipitation-Frequency Atlas of the United States Volume 1 Version 5.0".

[http://www.nws.noaa.gov/oh/hdsc/PF\\_documents/Atlas14\\_Volume1.pdf](http://www.nws.noaa.gov/oh/hdsc/PF_documents/Atlas14_Volume1.pdf)

Precipitation Frequency Data Server (PFDS):

[http://hdsc.nws.noaa.gov/hdsc/pfds/pfds\\_map\\_cont.html?bkmrk=nm](http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=nm)

The use of the PFDS is preferred due to the accuracy with which point rainfall amounts may be determined using the digital map based tools.

Infiltration and other losses are estimated using the NRCS Curve Number (CN) methodology. Input parameters are consistent with those used in the NRCS Unit Hydrograph Method. The Simplified Peak Discharge Method is limited for NMDOT use to single basins less than 10 square miles in area and should not be used when  $T_c$  exceeds 10.0 hours. When  $T_c$  is less than 10 minutes, use 10 minutes. This method may be used on NMDOT projects for those conditions identified in **Section 401** (**Figure 401-1** and **Figure 401-2**) of this manual. This method should not be used for watersheds with perennial streamflow. In the case of perennial streams, use the method described in **Section 406** if a stream gage exists, or the method described in **Section 405**, and include base flow.

The NMDOT Drainage Design Bureau can be contacted to obtain a copy of a spreadsheet used to calculate flows via the SCS/NRCS Simplified Peak Discharge Method. Note that the

Engineer/Consultant is responsible for understanding the use of, and the accuracy of the results from this spreadsheet.

#### **404.2 Limitations**

The NRCS Simplified Peak Discharge Method limitations are as follows:

- Do not use on watersheds larger than 10 square miles
- Do not use when more than 30% of the drainage area is urban
- Do not use when more than 30% of the watershed is above 7500 feet in elevation
- Do not use a  $T_c$  of less than 10 minutes (0.16 hours) or greater than 10 hours
- Do not use on watersheds with perennial streams
- Do not use on areas impacted by significant snowmelt or recently impacted by severe wildfire

#### **404.3 Factors Affecting Runoff**

Precipitation is the source of runoff from small watersheds. The soils and vegetation of the watershed affect the amount of precipitation that runs off. Mechanical treatment on a watershed, along with its topography and shape, also affect the rate at which water runs off. Runoff Curve Numbers (CNs) represent the combined effect of soil, vegetative cover, and conservation practices in runoff determinations. Transmission or channel losses in sand and gravel bed channels can also significantly affect the volume and peak discharge arriving at the point of interest in a watershed.

NRCS, 2007, Part 630 National Engineering Handbook, Chapter 19, Transmission Losses, provides guidance for calculating the impacts of these losses on the flood hydrograph. If the engineer believes that transmission losses have a significant impact on flows in the basin, the analysis should not be performed using the Simplified Peak Discharge Method, but rather the Unit Hydrograph Method in HEC-HMS (**Section 405**).

#### **404.4 Precipitation**

The highest rates of runoff from small watersheds are usually caused by intense rainfall. The intensity of rainfall affects the rate of runoff more than it does the volume of runoff. Intense rainstorms that produce high rates of runoff in small watersheds usually do not extend over a large area. The same intense rainstorm that causes flooding in a small tributary is not likely to be the one that will cause major flooding in a main watercourse that drains many square miles. Data from recording rain gages were studied to determine an appropriate rainfall distribution for New Mexico. Generally, New Mexico has more intense, shorter duration rainfalls than other parts of the U.S.

The melting of accumulated snow in the mountains may result in a greater volume of runoff, but usually at a lesser rate than runoff caused by rainfall. The melting of a winter's snow accumulation over a large area may cause major flooding along rivers.

The Simplified Peak Discharge Method requires the 24-hour total precipitation depth, and the method is applicable to the 100-yr storm and all more frequent recurrence interval storms.

Obtain the 24-hour rainfall depth directly from the NOAA Precipitation Frequency Data Server (PFDS) as described in **Section 403.2**. For NMDOT projects, there is no reduction factor for partial series versus annual series applied to 2-year, 5-year, and 10-year rainfall depths. This represents a slight departure from the original NRCS Method (NRCS, 1985-2014) and adds a small percentage of safety factor for the more frequent return period events.

The time distribution of rainfall is built into the Simplified Peak Discharge Method. This statewide rainfall distribution varies from 45% to over 85% of the 24-hour rainfall occurring in the peak hour of the storm as the Time of Concentration ( $T_c$ ) varies from 10 minutes to 10 hours.

For NMDOT drainage design, find the 24-hour rainfall depth from the NOAA Precipitation Frequency Data Server for the centroid of each watershed.

#### **404.5 Antecedent Runoff Condition**

The amount of precipitation occurring in the five days preceding the storm in question is an indication of the Antecedent Runoff Condition (ARC) of the soil. The CNs in **Table 402-2** to **Table 402-5** are for an average ARC II. Watersheds in New Mexico most often meet an ARC I or ARC II condition. NRCS has over 60 years of experience in the sizing of flood control dams around New Mexico using ARC II as the design condition. Experience has shown that the use of ARC II is conservative in that as it has been extremely rare for the emergency spillway on one of their dams to flow (a majority of these dams were designed for the 25-year or 50-year flood event). ARC III provides a very conservative assumption and generates significantly larger peak discharges and runoff volumes than ARC II for the same Curve Number and is typically not the case for most watersheds in New Mexico. Therefore, **use ARC II for NMDOT projects.**

#### **404.6 Hydrologic Soil Groups**

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

Information regarding the soils in a watershed has been surveyed by NRCS and other agencies for almost the entire country including the State of New Mexico. This information is generally available from the NRCS by consulting the Natural Resources Conservation Service's (NRCS) Field Office Technical Guide; or the Web Soil Survey website.

<http://websoilsurvey.nrcs.usda.gov/>

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



For an expanded discussion and instructions on soils and their effects on runoff, see **Sections 402.4, 402.5, and 402.6**. See also **Example Problem 6-7** located in Appendix 6 for a technical paper titled “Hatch Site 6 Runoff Methods Revisited” as an example of an approach for searching more deeply into predicted runoff results.

#### 404.7 Vegetative Cover

Vegetation affects runoff in several ways including the following:

- The foliage and its litter maintain the soil's infiltration potential by preventing the sealing of the soil surface from raindrop impact
- Foliage retains some of the raindrops, increasing their chance of being evaporated
- Some of the moisture is intercepted on the plant and withheld from the initial period of runoff
- Vegetation transpires soil moisture leaving a greater void in the soil to be filled
- Vegetation, including its ground litter, forms numerous barriers along the path of the water flowing over the surface of the land (this lengthens the travel time and increases opportunity for infiltration)

The following information can be used as a guide in determining the vegetative cover conditions for range sites. Grass cover is evaluated on plant basal area while trees and shrubs are evaluated using canopy cover. Litter can be an effective cover and should be considered.

**Cover Condition Class**

Condition	Vegetative Cover
Poor	Less than 30% ground cover
Fair	About 30% to 70% ground cover
Good	More than 70% ground cover

Refer to NRCS NEH Part 630, (EFH) Amend. IA50, Nov. 2007 “Hydrologic Soil-Cover Complexes”.

[https://www.nrcs.usda.gov/Internet/FSE\\_DOCUMENTS/nrcs142p2\\_022388.pdf](https://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/nrcs142p2_022388.pdf)

For a more complete guide to determining the percentage of vegetative cover, see “Sampling Vegetation Attributes” Interagency Technical Reference 1996 (Rev. ed. 1997 and 1999) at:

[http://www.nrcs.usda.gov/Internet/FSE\\_DOCUMENTS/stelprdb1044175.pdf](http://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/stelprdb1044175.pdf)

For a more detailed discussion and instructions on determining the appropriate Cover Conditions see **Sections 402.5 and 402.6** and the example Soil Cover Complex photographs presented in **APPENDIX 4**.

#### 404.8 Conservation Practices

Conservation practices, in general, reduce sheet erosion and thereby maintain an open structure of the soil surface. Soil and water conservation practices are control measures

consisting of managerial, vegetative, and structural practices to reduce the loss of soil and water. The application of conservation practices across a watershed reduces the volume of runoff, but the effect diminishes rapidly with increased storm magnitude. Some types of these practices are discussed below. Visit the NRCS website for more detailed information regarding conservation practices.

[https://www.nrcs.usda.gov/wps/portal/nrcs/detailfull/national/technical/cp/ncps/?cid=nrcs143\\_026849](https://www.nrcs.usda.gov/wps/portal/nrcs/detailfull/national/technical/cp/ncps/?cid=nrcs143_026849)

Crop residue tilled into the soil and the residual root system from grasses that have been in the crop rotations produce a condition favoring greater infiltration and water storage in the soil profile. The effect of conservation tillage on reducing runoff ranges from slight to substantial.

Contouring and terracing reduce sheet erosion and increase the amount of rainfall withheld from runoff by the small reservoirs they form. Land areas in which level terraces have been constructed may be excluded from the drainage area above downstream measures if they store the design depth of runoff. Gradient terraces increase the distance water must travel and thereby increase the Time of Concentration. This, in turn, reduces the peak rate of discharge.

Watershed slopes affect the rate of runoff and the peak discharge rate at downstream points. Slopes have a smaller effect on the volume of runoff than conservation practices such as contouring and terracing.

Small depressions may trap an initial amount of rain, thus reducing the amount of expected runoff. Where ponding or swampy areas occur in the watershed, a considerable amount of surface runoff may be retained in temporary storage. NRCS Small Watershed Hydrology WinTR-55 User Guide, 2009 contains a procedure to adjust the peak discharge for ponded areas.

[http://www.nrcs.usda.gov/Internet/FSE\\_DOCUMENTS/stelprdb1042897.pdf](http://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/stelprdb1042897.pdf)

#### **404.9 Runoff Curve Number (CN)**

The NRCS Runoff Curve Number (CN) is a lumped watershed parameter and most often serves as a proxy for all losses to precipitation from the time it hits the ground surface until it reaches the point of interest in a hydrologic analysis. As such, it should not be interpreted as a point infiltration value but rather as representing all losses (capture, infiltration, transmission, evaporation, etc.) unless separate calculations will be made for ponding and transmission losses.

**Sections 402.5** and **402.6** contain important and useful excerpts from NRCS, June 1986, TR-55, Urban Hydrology for Small Watersheds, Chapter 2, which provides a complete and clear explanation of the CN, its determination, and its use in hydrologic analyses.

[www.wcc.nrcs.usda.gov/ftpref/wntsc/H&H/other/TR55documentation.pdf](http://www.wcc.nrcs.usda.gov/ftpref/wntsc/H&H/other/TR55documentation.pdf)

#### **404.10 Time of Concentration**

Calculate the Time of Concentration ( $T_c$ ) for use in the Simplified Peak Discharge Method using the Upland Method for un-gullied watersheds and the upper, un-gullied portions of somewhat gullied watersheds. Use the Kirpich Equation for the gullied portions of the watershed and for watersheds that are almost entirely gullied. Follow the guidance in **Section 402.8**.

## 404.11 Peak Discharge Application Procedure

### Step 1 – Gather input data.

Use the Simplified Peak Discharge Method worksheet **Figure 404-2**. Establish the appropriate Design Frequency Flood(s) for analysis (**Section 200**).

- Measure the drainage area, (A), in acres
- Compute the Time of Concentration, (T<sub>c</sub>), in hours (**Sections 402.8 and 402.9**)
- Determine the appropriate Runoff Curve Number, CN, for the drainage basin (**Sections 402.5 and 402.6**)
- Obtain the 24-hour rainfall depth, P<sub>24</sub>, in inches, for the appropriate design frequency, from NOAA Atlas 14 or online from the NOAA PFDS

### Step 2 – Determine the unit peak discharge, q<sub>u</sub>, for the watershed.

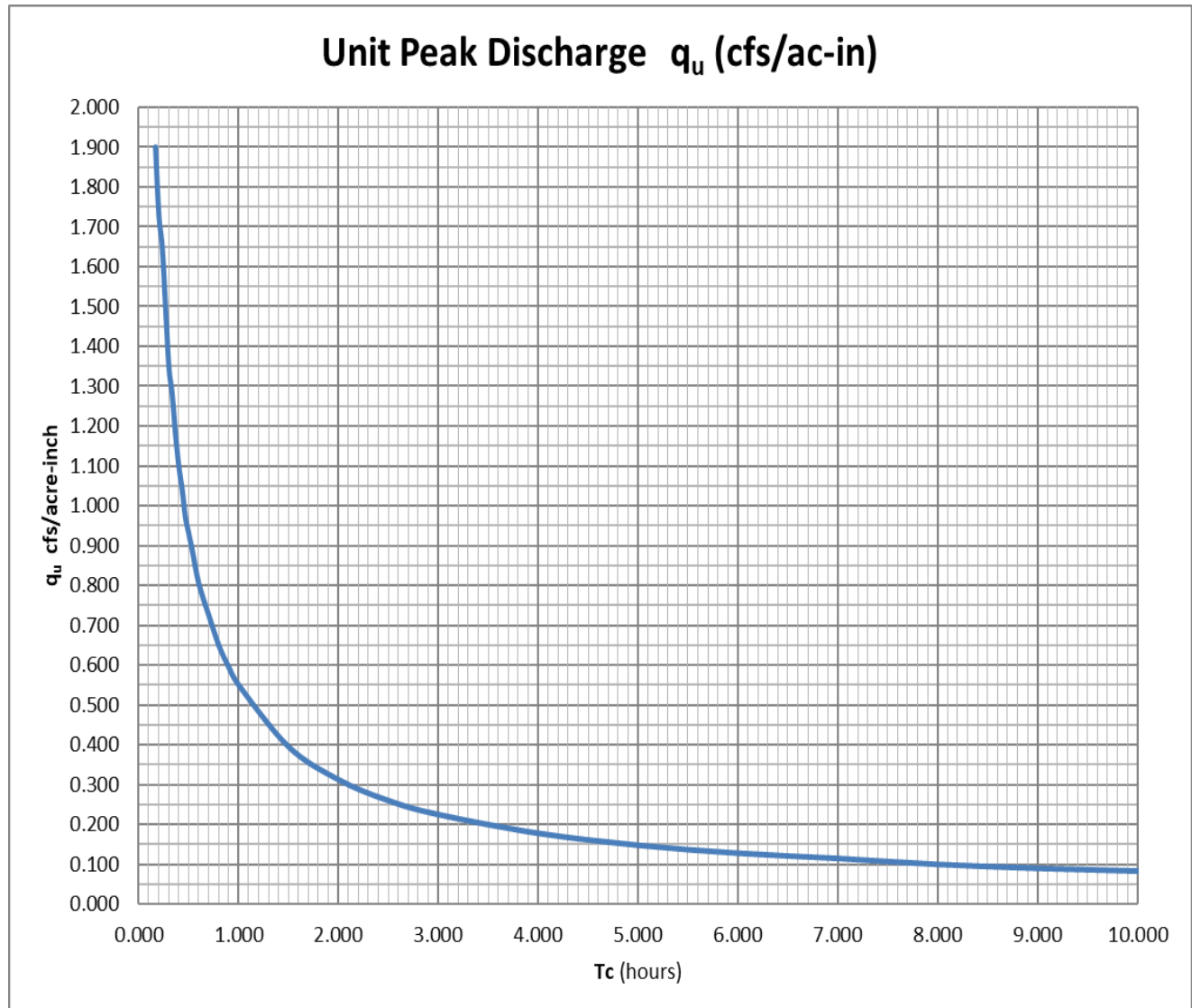
The unit peak discharge, q<sub>u</sub>, in cfs/ac-in. can be read from **Table 404-1** or **Figure 404-1**, given the T<sub>c</sub>.

**Table 404-1 Unit Peak Discharge Table for NRCS Simplified Peak Discharge Method**

Source: Soil Conservation Service, 1973, revised by Luther McDougal, and Calvin Jackson, 1973, updated by Larry Goertz, February 1985, updated by Roger Ford, 2014, "Peak Rates of Discharge for Small Watersheds, Chapter 2, Engineering Field Manual for Conservation Practices".

(Not available on-line – see **APPENDIX 5**).

<b>Tc</b>		<b>qu</b>	<b>Tc</b>		<b>qu</b>
hours	min	cfs/ac-in	hours	min	cfs/ac-in
0.167	10.000	1.900	1.500	90	0.395
0.200	12.000	1.730	2.000	120	0.313
0.233	14.000	1.650	2.500	150	0.260
0.267	16.000	1.500	3.000	180	0.225
0.300	18.000	1.350	3.500	210	0.202
0.333	20.000	1.280	4.000	240	0.178
0.367	22.000	1.180	4.500	270	0.163
0.400	24.000	1.100	5.000	300	0.148
0.433	26.000	1.040	5.500	330	0.138
0.467	28.000	0.970	6.000	360	0.128
0.500	30.000	0.930	6.500	390	0.122
0.533	32.000	0.890	7.000	420	0.115
0.567	34.000	0.848	7.500	450	0.108
0.600	36.000	0.805	8.000	480	0.100
0.633	38.000	0.778	8.500	510	0.095
0.667	40.000	0.752	9.000	540	0.090
0.700	42.000	0.725	9.500	570	0.087
0.733	44.000	0.688	10.000	600	0.083
0.800	48.000	0.650			
0.867	52.000	0.623			
0.900	54.000	0.595			
1.000	60.000	0.550			



Source: Soil Conservation Service, 1973, revised by Luther McDougal, and Calvin Jackson, 1973, updated by Larry Goertz, February 1985, updated by Roger Ford, 2014, "Peak Rates of Discharge for Small Watersheds, Chapter 2, Engineering Field Manual for Conservation Practices".

(Not available on-line – see **APPENDIX 5**).

**Figure 404-1 Unit Peak Discharge for NRCS Simplified Peak Discharge Method**

If not using **Figure 404-1**, then read the unit peak discharge ( $q_u$ ) value from **Table 404-1**.

Calculate the direct runoff depth ( $Q$ ) from the watershed. The direct runoff is expressed as an average depth of runoff ( $Q$ ) over the entire watershed, in inches. The direct runoff may be read from **Figure 402-8** using the 24-hour rainfall depth ( $P$ ) in inches, and the Runoff Curve Number, CN.

The direct runoff depth (Q) may also be calculated from the following equation:

$$Q = \frac{\left[ P - \left( \frac{200}{CN} \right) + 2 \right]^2}{P + \left( \frac{800}{CN} \right) - 8} \quad 404-1$$

(Soil Conservation Service, 1973, revised by Luther McDougal, and Calvin Jackson, 1973, updated by Larry Goertz, February 1985, updated by Roger Ford, 2014, "Peak Rates of Discharge for Small Watersheds, Chapter 2, Engineering Field Manual for Conservation Practices")

where:

Q	=	direct runoff, inches
P	=	rainfall depth, inches
CN	=	Runoff Curve Number

Note that this method was developed based the 24-hour rainfall duration (P), with the maximum return period of 100-years, and is also applicable for more frequent return periods. The direct runoff depth (Q) may sometimes be shown as  $Q_d$ , to indicate depth, and to distinguish this term from the letter Q, which is also used often to designate discharge in cubic feet per second (cfs).

### Step 3 – Compute the peak discharge

Compute the peak discharge ( $Q_p$ ) from the watershed by the following equation:

$$Q_p = A Q q_u \quad 404-2$$

where:

$Q_p$	=	peak discharge, cfs
A	=	drainage area, acres
Q	=	direct runoff, inches
$q_u$	=	unit peak discharge, cfs/acre-inch

Step 4 – Compute the runoff volume, if required.

The runoff volume ( $Q_v$ ) is obtained by the equation:

$$Q_v = (Q \ A) / 12$$

**404-3**

where:

Q	=	direct runoff, inches
$Q_v$	=	runoff volume from the watershed, ac-ft
A	=	drainage area, acres

Step 5 – Estimate Transmission Losses

Transmission losses shall not be applied when using the Simplified Peak Discharge Method except for water quality and sediment transport related applications. For small frequent rainfall events and water quality analyses, transmission losses can be significant and should be considered. For sediment transport analyses, transmission losses should be considered to avoid over estimation of sediment transport rates.

**Simplified Peak Discharge Method Worksheet**

Structure Location: MP: \_\_\_\_\_ County: \_\_\_\_\_  
 District: \_\_\_\_\_

Structure Description: \_\_\_\_\_

Drainage Area:  $A =$  \_\_\_\_\_ acres, \_\_\_\_\_  $\text{mi}^2$

Elevation at Centroid of Watershed: Elev = \_\_\_\_\_ ft \*

Location of Centroid: Lat: \_\_\_\_\_ Long: \_\_\_\_\_

Time of Concentration:  $T_c =$  \_\_\_\_\_ hours

Method: ☐ Upland ☐ Kirpich ☐ Mixed

Weighted Runoff Curve Number: CN = \_\_\_\_\_

Method: ☐ Area ☐ Runoff

Unit Peak Discharge (from Figure 404-1):  $q_u =$  \_\_\_\_\_ cfs/ac-in

<u>Design Frequency Flood</u>	_____ - year	_____ - year
24-hour Rainfall Depth (NOAA PFDS): $P_{24}$	_____ in.	$P_{24} =$ _____ in.
Direct Runoff (Figure 402-8):	$Q_d =$ _____ in.	$Q_d =$ _____ in.
Peak Discharge, $Q_p = A \cdot Q_d \cdot q_u$ :	$Q_p =$ _____ cfs	$Q_p =$ _____ cfs
Discharge per acre	_____ cfs/ac	_____ cfs/ac
Runoff Volume, $Q_v = A \cdot Q_d / 12$ :	$Q_v =$ _____ ac-ft	$Q_v =$ _____ ac-ft

Project Location: \_\_\_\_\_

CN#: \_\_\_\_\_

Date: \_\_\_\_\_

Computed By: \_\_\_\_\_

Checked By: \_\_\_\_\_

\* If elevation is greater than 7500 ft, use NRCS Unit Hydrograph method

Source: Soil Conservation Service, 1973, revised by Luther McDougal, and Calvin Jackson, 1973, updated by Larry Goertz, February 1985, updated by Roger Ford, 2014, "Peak Rates of Discharge for Small Watersheds, Chapter 2, Engineering Field Manual for Conservation Practices".

**Figure 404-2 Simplified Peak Discharge Method Worksheet**

Appendix 6 contains two example Simplified Peak Discharge Method problems. **Example Problem 6-3** is for a mid-size basin (7.6 sq mi) and **Example Problem 6-4** is for a small basin (1.07 sq mi).



## 404.12 References

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[http://www.nrcs.usda.gov/Internet/FSE\\_DOCUMENTS/nrcs142p2\\_011485.pdf](http://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/nrcs142p2_011485.pdf)

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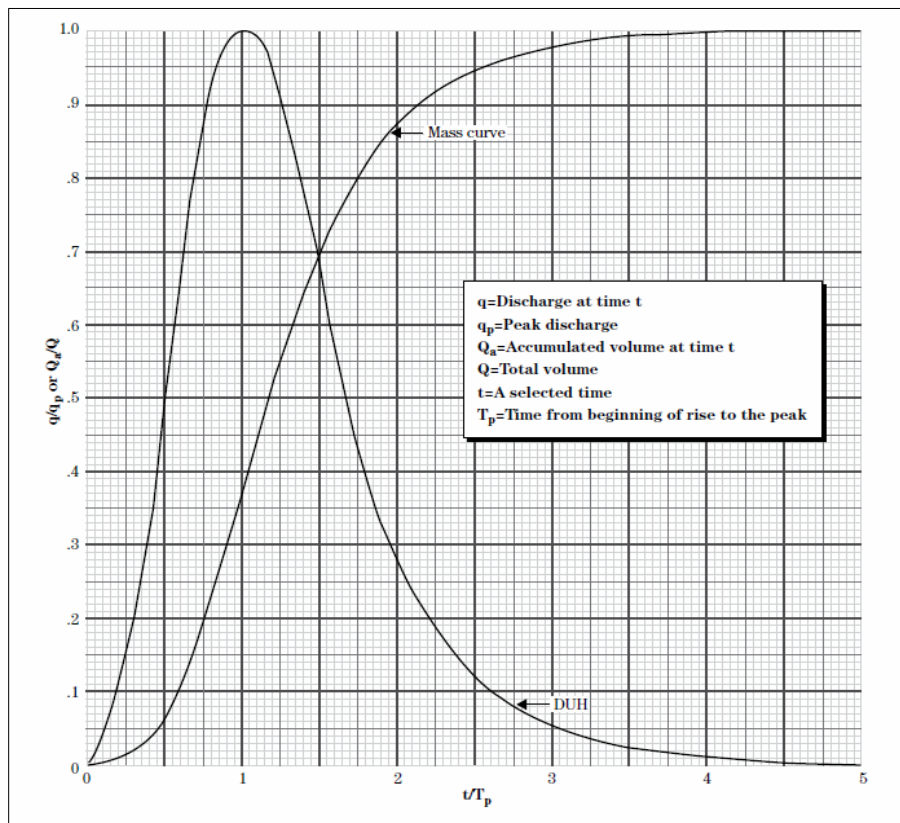
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Soil Conservation Service, 1973, revised by Luther McDougal, and Calvin Jackson, 1973, updated by Larry Goertz, February 1985, updated by Roger Ford, 2014, “Peak Rates of Discharge for Small Watersheds, Chapter 2, Engineering Field Manual for Conservation Practices”. (Not available on-line see **APPENDIX 5**).

## 405 NRCS (SCS) Unit Hydrograph Method within HEC-HMS

While there are multiple computer programs that can be used to develop a hydrograph, the NRCS Synthetic Unit Hydrograph Method has been selected for use on NMDOT projects in order to simplify reviews and to improve consistency. This method shall be used for watersheds over 10 square miles, or which have centroids above 7500 feet and whenever peak discharge calculations involve multiple subbasins and complex hydraulics within and among subbasins. The method should also be used whenever the analysis includes flood routing through detention facilities, pump stations, or long conveyance facilities. Synthetic unit hydrographs can be used to model drainage basins with or without base flow.

A hydrograph is a plot of discharge versus time. Synthetic unit hydrograph methods are used to adjust the shape of a generalized hydrograph to a particular drainage basin, usually at an ungaged site. A unit hydrograph is defined as the direct runoff hydrograph resulting from a rainfall event which has a specific temporal and spatial distribution, and which generates a unit depth of rainfall. The area beneath the unit hydrograph curve is equal to the volume of direct runoff from one inch of excess rainfall over the entire drainage basin or subbasin. **Figure 405-1** shows a dimensionless unit hydrograph and its associated cumulative mass curve.



Source: NRCS, 2007, "Part 630 Hydrology, National Engineering Handbook", Chapter 16, Hydrographs, Figure 16-1, p. 16-3.

<http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=17755.wba>

**Figure 405-1 Dimensionless Unit Hydrograph and Mass Curve**

The NRCS Unit Hydrograph was developed through the analysis of a large number of natural (measured) unit hydrographs from a broad cross section of geographic locations and hydrologic regions around the continental United States.

Computer models are the preferred approach for application of the SCS (now NRCS) Synthetic Unit Hydrograph Method. These computation methods make creation, addition, and routing of multiple hydrographs a relatively easy task.

There are commercially available software programs such as WMS and AutoDesk that perform hydrologic modeling. However, the NMDOT model of choice for large and/or complex watersheds and anytime a hydrograph is needed, is the U.S. Army Corps of Engineers (USACE) program HEC-HMS. Appendix 6 contains **Example Problem 6-6** that presents an example of a HEC-HMS problem.

The program, the User's Manual, the Technical Reference Manual, the Application Guide and sample models are available as free downloads from the USACE Hydrologic Engineering Center at:

<http://www.hec.usace.army.mil/software/hec-hms/>

HEC-HMS version 4.2.1 (latest version at the time of the publication of this manual) is capable of performing a wide variety of hydrologic analyses. With the GIS companion product (HEC-GeoHMS) data collection, basin delineation and rainfall input parameters have been simplified and made reproducible.

Basic data for HEC-HMS is standard to nearly all hydrologic analyses models as follows:

- Drainage basin area
- Time of Concentration
- Rainfall/Runoff algorithm (in this case Runoff Curve Number)
- Total rainfall depth
- Rainfall temporal distribution
- Conveyance system hydraulic data

Detailed instructions for the construction of a HEC-HMS model are not included in this manual since they are extensive and well presented in the HEC-HMS User's and Technical Reference Manuals. HEC-HMS has been updated several times since its introduction, and its capabilities are modified and expanded with each version. Also, since the use of the most current version is recommended, the inclusion of detailed usage instructions which are subject to change in this manual is not practical.

There are some basic requirements for use of a hydrologic computer model on a NMDOT project.

- Use of a computer model other than HEC-HMS must be approved by the NMDOT Drainage Design Bureau prior to its use.
- **The rainfall distribution used must be the 25% frequency** produced by HEC-HMS from rainfall data from NOAA Atlas 14 or the NOAA Precipitation Frequency Data Server for the specific flood frequency and watershed under investigation, unless otherwise authorized by the Drainage Design Bureau (see **Section 405.3** for further explanation).
- Tc must be computed using the Iterative Method within the Stream Hydraulic Method, and/or the Upland Method as appropriate. The use of the Kirpich Equation is appropriate for checking the results from **Section 402.9.5**. Refer to **Table 402-6** for guidance on

selection of a Time of Concentration method. Complete input files, routing diagrams, and summary output files must be included (in an appendix) in every drainage report, as well as the HEC-HMS Method worksheet (see **Figure 405-9**).

- When hydrograph routing is required, the Muskingum-Cunge Method is preferred for use with the NRCS Unit Hydrograph procedure. On occasion, special circumstances may warrant the use of one of the other routing methods available within HEC-HMS. Consult with the Drainage Design Bureau before using an alternative method.

## **405.1 Basin Delineation**

Regardless of the hydrologic analysis method selected including HEC-HMS, the area of a drainage basin and its subbasins are always required. Basic to all hydrologic methods is the assumption that the basin or subbasin can be reasonably characterized by one set of hydrologic characteristics (soils, slope, rainfall, vegetative cover, and land use). The further the basin and subbasin characteristics diverge from this assumption, the less accurate and reproducible the results will be. Good “rules of thumb” regarding basin and subbasin sizing are that the length of a basin or subbasin delineation should not exceed 4 times its width and that no subbasin should be more than 10 times larger than the smallest subbasin.

**Section 402.2** contains a more detailed description of the hydrologic factors that should be considered when delineating basins and subbasins. Also refer to the discussion in **Section 405.9** regarding minimum  $T_c$  and model computation interval as they relate to basin size and modeling.

## **405.2 Rainfall Volume**

The rainfall depths for the design frequency storm are to be found at the NOAA Precipitation Frequency Data Server for the centroid of the watershed being studied (using the Partial Duration Series). In very large watersheds, the use of different rainfall volumes for portions of the watershed may be appropriate (e.g. mountain faces might differ from the alluvial plains below). Rainfall depths for specific durations (i.e. 5 minute, 15 minute, 60 minute, etc.) are also provided. These values are inputs to HEC-HMS for development of the 25% design rainfall temporal distribution used in the NRCS Unit Hydrograph Method.

## **405.3 Rainfall Temporal Distribution**

Proper application of this method requires use of a 24-hour rainfall event with the peak precipitation rate occurring at 6 hours. Rainfall data for the NRCS Unit Hydrograph Method consists of point precipitation depths for various durations up to and including the 24-hour point depth, and also requires a rainfall distribution. Point precipitation depths for the design return period may be obtained directly from NOAA Atlas 14 or the NOAA Precipitation Frequency Data Server.

Previously, the rainfall distribution prescribed for use on NMDOT projects with the NRCS (SCS) Unit Hydrograph Method was called the Modified NOAA-SCS rainfall distribution. This Modified NOAA-SCS rainfall distribution was a combination of the peak rainfall intensity defined by NOAA, with an NRCS Type II-a storm rearrangement. HEC-HMS does not have a built in NRCS Type II-a storm distribution.

However, the 25% frequency storm distribution available within HEC-HMS is a very close approximation and is prescribed for NMDOT hydrologic analyses wherever a rainfall distribution is required. Given that NOAA Atlas 14 has a greatly expanded database compared to the data available to the U.S. Weather Bureau at the time the Type II-a distribution was developed, the 25% distribution available in the HEC-HMS program should produce more accurate results throughout New Mexico.

For NMDOT drainage design projects, apply the 25% frequency storm distribution. The HEC-HMS User's Manual describes the method for creating model rainfall distributions. **Figure 405-2** and **Figure 405-3** are provided for additional guidance.

## Precipitation Frequency Data Server

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NOAA Atlas 14, Volume 1, Version 5  
 Location name: Mountainair, New Mexico, US\*  
 Latitude: 34.3000°, Longitude: -106.1000°  
 Elevation: 6500 ft\*  
 \* source: Google Maps



## POINT PRECIPITATION FREQUENCY ESTIMATES

Sanja Perica, Sarah Dietz, Sarah Heim, Lillian Hiner, Kazungu Maitaria, Deborah Martin, Sandra Pavlovic, Ishani Roy, Carl Trypaluk, Dale Unruh, Fenglin Yan, Michael Yekta, Tan Zhao, Geoffrey Bonnin, Daniel Brewer, Li-Chuan Chen, Tye Parzybok, John Yarchon

NOAA, National Weather Service, Silver Spring, Maryland

[PF tabular](#) | [PF graphical](#) | [Maps & arials](#)

## PF tabular

PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches) <sup>1</sup>										
Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.246 (0.216–0.280)	0.318 (0.280–0.362)	0.422 (0.371–0.482)	0.502 (0.440–0.574)	0.608 (0.534–0.696)	0.689 (0.606–0.793)	0.775 (0.681–0.897)	0.862 (0.759–1.00)	0.979 (0.863–1.15)	1.07 (0.947–1.28)
10-min	0.375 (0.329–0.427)	0.484 (0.425–0.551)	0.643 (0.564–0.734)	0.764 (0.669–0.874)	0.925 (0.812–1.06)	1.05 (0.922–1.21)	1.18 (1.04–1.37)	1.31 (1.16–1.53)	1.49 (1.31–1.76)	1.63 (1.44–1.94)
15-min	0.464 (0.408–0.529)	0.599 (0.527–0.683)	0.797 (0.700–0.910)	0.947 (0.829–1.08)	1.15 (1.01–1.31)	1.30 (1.14–1.50)	1.46 (1.28–1.69)	1.63 (1.43–1.89)	1.85 (1.63–2.18)	2.02 (1.79–2.41)
30-min	0.625 (0.549–0.713)	0.807 (0.710–0.920)	1.07 (0.942–1.23)	1.28 (1.12–1.46)	1.54 (1.36–1.77)	1.75 (1.54–2.02)	1.97 (1.73–2.28)	2.19 (1.93–2.55)	2.49 (2.19–2.93)	2.73 (2.41–3.24)
60-min	0.774 (0.680–0.882)	0.999 (0.878–1.14)	1.33 (1.17–1.52)	1.58 (1.38–1.81)	1.91 (1.68–2.19)	2.17 (1.90–2.49)	2.44 (2.14–2.82)	2.71 (2.39–3.16)	3.08 (2.72–3.63)	3.37 (2.98–4.01)
2-hr	0.907 (0.799–1.04)	1.16 (1.02–1.33)	1.53 (1.35–1.75)	1.82 (1.60–2.07)	2.23 (1.95–2.54)	2.57 (2.23–2.91)	2.93 (2.52–3.32)	3.31 (2.83–3.75)	3.86 (3.26–4.37)	4.32 (3.61–4.90)
3-hr	0.956 (0.848–1.09)	1.21 (1.08–1.39)	1.58 (1.39–1.81)	1.88 (1.65–2.14)	2.30 (2.00–2.61)	2.64 (2.28–2.99)	3.00 (2.58–3.41)	3.40 (2.90–3.85)	3.96 (3.34–4.50)	4.43 (3.70–5.04)
6-hr	1.08 (0.961–1.23)	1.37 (1.22–1.56)	1.75 (1.55–1.99)	2.06 (1.82–2.34)	2.49 (2.19–2.82)	2.84 (2.47–3.21)	3.21 (2.78–3.63)	3.61 (3.10–4.08)	4.16 (3.54–4.71)	4.62 (3.89–5.24)
12-hr	1.23 (1.09–1.39)	1.55 (1.37–1.75)	1.96 (1.74–2.22)	2.29 (2.02–2.59)	2.75 (2.42–3.11)	3.12 (2.73–3.52)	3.51 (3.05–3.96)	3.92 (3.39–4.43)	4.50 (3.84–5.09)	4.97 (4.21–5.63)
24-hr	1.42 (1.31–1.55)	1.78 (1.65–1.94)	2.23 (2.06–2.44)	2.59 (2.38–2.83)	3.08 (2.82–3.36)	3.47 (3.16–3.78)	3.87 (3.51–4.22)	4.28 (3.87–4.68)	4.85 (4.34–5.32)	5.29 (4.71–5.83)
2-day	1.58 (1.45–1.72)	1.98 (1.82–2.16)	2.47 (2.27–2.69)	2.86 (2.63–3.13)	3.40 (3.10–3.71)	3.83 (3.48–4.18)	4.27 (3.87–4.67)	4.73 (4.25–5.18)	5.35 (4.76–5.89)	5.85 (5.16–6.45)
3-day	1.72 (1.58–1.87)	2.15 (1.98–2.34)	2.68 (2.47–2.92)	3.11 (2.86–3.38)	3.69 (3.38–4.02)	4.15 (3.78–4.52)	4.63 (4.20–5.04)	5.12 (4.62–5.59)	5.79 (5.17–6.35)	6.33 (5.60–6.96)
4-day	1.85 (1.71–2.01)	2.32 (2.14–2.53)	2.89 (2.67–3.15)	3.35 (3.09–3.64)	3.98 (3.65–4.32)	4.48 (4.09–4.86)	4.98 (4.54–5.42)	5.51 (4.98–6.00)	6.23 (5.58–6.81)	6.80 (6.04–7.46)
7-day	2.25 (2.08–2.43)	2.81 (2.59–3.05)	3.48 (3.21–3.78)	4.02 (3.70–4.35)	4.74 (4.35–5.14)	5.30 (4.84–5.75)	5.87 (5.35–6.37)	6.45 (5.84–7.02)	7.24 (6.50–7.90)	7.86 (7.00–8.60)
10-day	2.53 (2.34–2.75)	3.16 (2.92–3.44)	3.94 (3.63–4.29)	4.54 (4.18–4.95)	5.38 (4.92–5.86)	6.03 (5.49–6.57)	6.69 (6.08–7.31)	7.37 (6.66–8.08)	8.30 (7.43–9.12)	9.02 (8.01–9.97)
20-day	3.36 (3.12–3.62)	4.19 (3.89–4.53)	5.15 (4.78–5.57)	5.88 (5.44–6.36)	6.83 (6.30–7.39)	7.55 (6.93–8.17)	8.26 (7.56–8.95)	8.96 (8.16–9.73)	9.88 (8.95–10.8)	10.6 (9.52–11.6)
30-day	4.06 (3.79–4.35)	5.06 (4.72–5.44)	6.17 (5.74–6.63)	6.99 (6.50–7.52)	8.04 (7.46–8.65)	8.82 (8.16–9.50)	9.58 (8.84–10.3)	10.3 (9.49–11.2)	11.3 (10.3–12.2)	12.0 (10.9–13.0)
45-day	5.11 (4.78–5.47)	6.36 (5.94–6.81)	7.67 (7.16–8.22)	8.63 (8.04–9.26)	9.83 (9.13–10.6)	10.7 (9.91–11.5)	11.5 (10.6–12.4)	12.3 (11.3–13.3)	13.3 (12.2–14.4)	14.1 (12.8–15.3)
60-day	6.00 (5.59–6.42)	7.47 (6.97–8.02)	9.00 (8.39–9.66)	10.1 (9.41–10.8)	11.5 (10.7–12.3)	12.4 (11.5–13.4)	13.4 (12.4–14.4)	14.2 (13.1–15.4)	15.3 (14.1–16.6)	16.1 (14.7–17.5)

<sup>1</sup> Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

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Figure 405-2 Sample NOAA Precipitation Frequency Data Sever Output



The screenshot shows the HEC-HMS software interface. The 'Components' pane on the left lists the project structure: NMDOT, Basin Models, Basin 1, Meteorologic Models, Met 1, and Frequency Storm (selected). The main window displays the 'Frequency Storm' input parameters for 'Met 1'. The parameters are as follows:

- Met Name: Met 1
- Probability: Other
- Input Type: Partial Duration
- Output Type: Annual Duration
- Intensity Duration: 5 Minutes
- Storm Duration: 1 Day
- Intensity Position: 25 Percent
- Storm Area (MI2): 15
- Curve: Uniform For All Subbasins

Below the parameters is a table showing the 'Partial-Duration Depth (IN)' for various 'Duration' values. The '15 Minutes' row is highlighted in blue.

Duration	Partial-Duration Depth (IN)
5 Minutes	0.689
15 Minutes	1.30
1 Hour	2.17
2 Hours	2.57
3 Hours	2.64
6 Hours	2.84
12 Hours	3.12
1 Day	3.47

**Figure 405-3 Sample HEC-HMS Precipitation Input Table**

#### **405.4 Soils Data**

The NMDOT requires that hydrologic modeling within HEC-HMS utilize the NRCS Runoff Curve Number (CN) Method for determining a watershed's response to rainfall. Soils data (Hydrologic Soils Group) is integral to determining the CN.

The texture, composition and density of soils have a direct impact on the amount, and rate at which rainfall becomes runoff and, therefore, the soil type is a critical piece of information in the development of rainfall/runoff calculations. In general, soils are classified as sandy, silty, loamy or clayey. Of course, in nature, there can be an infinite number of combinations of these characteristics. The NRCS has divided the extremely wide range of soil textures by their hydrologic (runoff producing) characteristics into four Hydrologic Soils Groups: Type A, B, C, and D with: Type A being generally sandy soils and low runoff producers and Type D being

clayey soils and high producers of runoff for a given rainfall volume. See **Section 402.4** for a more detailed description of soil classifications and their impact on the CN. Soils data are available for almost all of New Mexico from the NRCS Web Soil Survey at:

<http://websoilsurvey.nrcs.usda.gov/>.

### **405.5 Hydrologic Soil Cover Complexes**

A combination of a Hydrologic Soil Group (soil), land use and treatment class (cover) is a hydrologic soil-cover complex. A range of Runoff Curve Numbers (CNs) based on the combination of soil texture and cover has been developed by the NRCS from empirical data and is published by NRCS in their National Engineering Handbook, Chapter 9 as well in multiple other locations. **Section 402.5** contains a detailed description of the accepted process for determining appropriate soil cover complexes for use on NMDOT projects.

### **405.6 Runoff Curve Number**

The NRCS Runoff Curve Number (CN) is a lumped watershed parameter and most often serves as a proxy for all losses from the beginning of precipitation until runoff reaches the point of interest in a hydrologic analysis. As such, it should not be interpreted as a point infiltration value but rather as representing all losses (initial abstraction, infiltration, transmission, evaporation, etc.) unless separate calculations will be made for ponding and transmission losses. **Section 402.6** contains a detailed description of the methods prescribed for determining the CN for NMDOT projects.

### **405.7 Other Land Use Effects**

HEC-HMS has the ability to simulate the effects of directly connected impervious areas, ponds, dams, storm drains, and pump stations on the runoff hydrograph. The HEC-HMS User's Manual and the Technical Reference Manual should be consulted for the details regarding input data, limitations and capabilities of the software. Any NMDOT project that contains these elements and requires analyses of their impacts should utilize HEC-HMS unless approved by the Drainage Design Bureau.

Note that when modeling heavily urban basins, if the engineer inputs percentage impervious directly into the model, HEC-HMS assumes a CN=100 and produces 100% runoff from that area. Impervious areas should be classified as CN=98. Do not use the percentage impervious option in HEC-HMS.

### **405.8 Time of Concentration and Basin Lag**

Time of Concentration ( $T_c$ ), is defined as the time required for runoff to travel from the hydraulically most remote part of the watershed to the point of interest. The determination of  $T_c$  is one of the most important and sensitive drainage basin modeling needs when calculating the peak rate of runoff and hydrographs in HEC-HMS.  $T_c$  is a simplified proxy for the hydrologic response to precipitation by a watershed (capturing the interrelated effects of size, shape, and slope). The  $T_c$  for a watershed or subbasin has the most dramatic effect on the shape of the runoff hydrograph of any parameter. An accurate estimate of a watershed's  $T_c$  is therefore



crucial to every type of hydrologic modeling. **Section 402.8** contains a detailed discussion and outlines the various methods approved to calculate and check  $T_c$  for a subbasin.

In the SCS (NRCS) Unit Hydrograph Method, basin lag (Lag or  $t_{lag}$ ) is defined as the time between the center of mass of excess rainfall and the peak of the unit hydrograph as:

$$\text{Lag} = 0.6 \times T_c \quad \text{405-1}$$

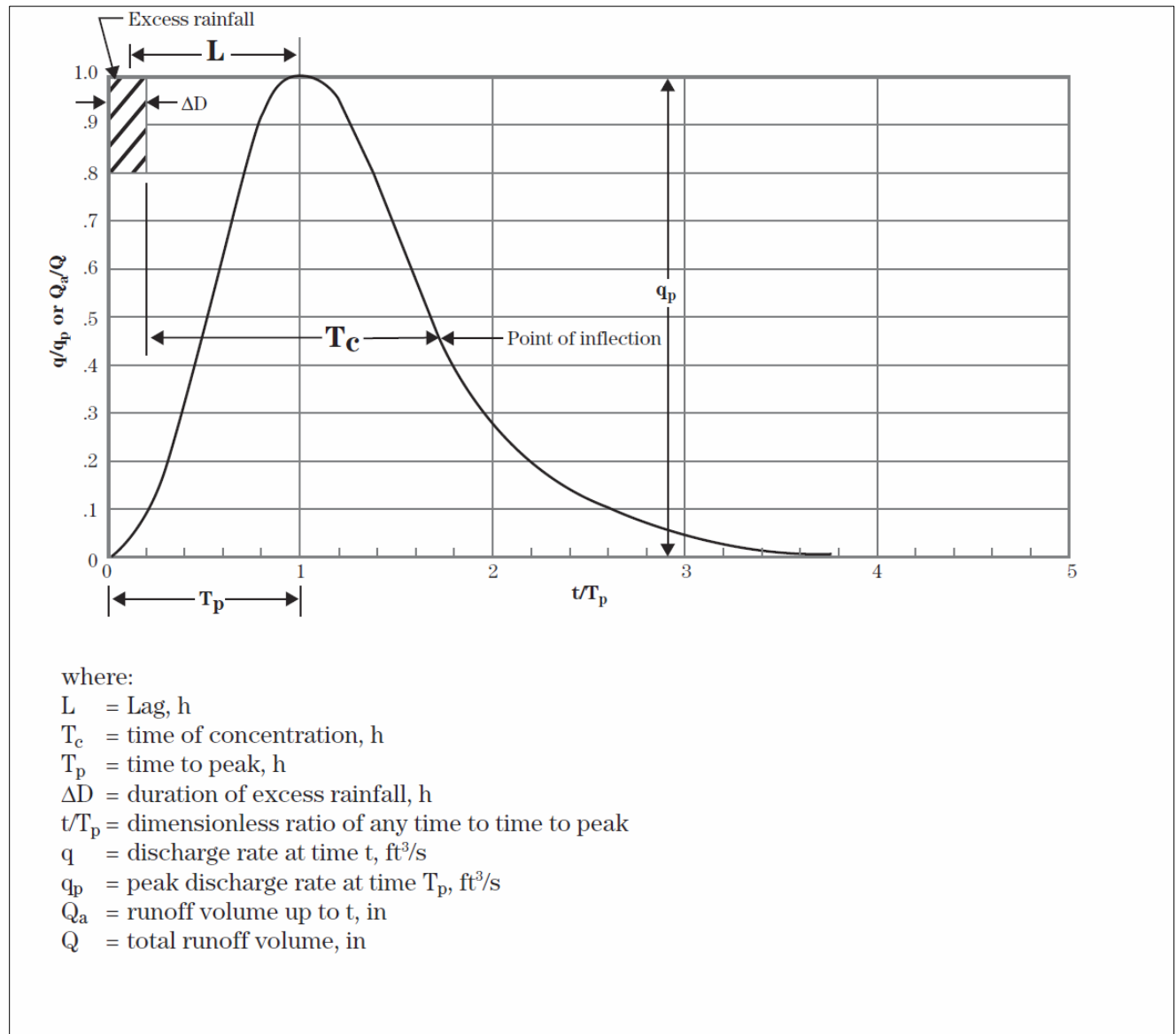
(NRCS, 2010, "Part 630 Hydrology, National Engineering Handbook, Chapter 15 Time of Concentration", Eq. 15-3, p. 15-3)

<https://directives.sc.egov.usda.gov/27002.wbas>

where:

Lag	=	the time between the center of mass of excess runoff and the hydrograph peak, hr
$T_c$	=	time of concentration, hr

**Figure 405-4** illustrates the various time relationships important to the development of the dimensionless unit hydrograph and resulting basin specific hydrographs within the NRCS Unit Hydrograph Method.



Source: NRCS, 2007, Part 630 Hydrology, National Engineering Handbook, Chapter 16  
Hydrographs, p. 16A-1

<https://www.nrcs.usda.gov/wps/portal/nrcs/detailfull/national/water/?&cid=stelprdb1043063>

**Figure 405-4 Graphical Representation of Relationships Between Lag,  $T_p$  and  $T_c$**

## **405.9 HEC-HMS Computation Interval and Duration Guidance**

### **405.9.1 Computation Interval**

The computation interval or time step for modeling within HEC-HMS can be specified for a range of intervals as follows:

1, 2, 3, 4, 5, 6, 10, 15, 20, 30 (minutes)

1, 2, 3, 6, 8, 12, 24 (hours)

Selection of the appropriate computation interval can also affect the modeling results. The HEC-HMS Technical Reference Manual (USACE, March 2000) states: “*that for adequate definition of the ordinates on the rising limb of the NRCS Unit Hydrograph, a computational interval,  $\Delta t$ , that is less than 29% of  $t_{lag}$  must be used (USACE 1998).*”

Therefore, if basin Lag=0.6 Tc, (Lag is the same as  $t_{lag}$ ) then the maximum computational interval for use within HEC-HMS to adequately define the rising limb of the hydrograph (and often to capture the peak) is given by:

$$\Delta t < 0.29 \times 0.60 T_c < 0.17 T_c$$

**405-2**

Note that  $0.29 \times 0.60 = 0.17$ , therefore this equation reduces to

$$\Delta t < 0.17 T_c$$

(USACE, March 2000, “Hydrologic Engineering Center, “HEC-HMS Technical Reference Manual, p. 55)

[http://www.hec.usace.army.mil/software/hec-hms/documentation/HEC-HMS\\_Technical%20Reference%20Manual\\_\(CPD-74B\).pdf](http://www.hec.usace.army.mil/software/hec-hms/documentation/HEC-HMS_Technical%20Reference%20Manual_(CPD-74B).pdf)

The following items are offered as additional guidance for selecting the minimum model computation interval selection:

1. Generally, the computation interval “ $\Delta t$ ” should be based on the Tc of the smallest subbasin in the model.
2. Note that the shortest rainfall interval available from NOAA is 5 minutes, selecting a shorter computation interval will require HEC-HMS to extrapolate to find a smaller than 5-minute rainfall increment.
3. For 24-hour storm distributions, use a computation interval “ $\Delta t$ ” of 5 minutes or greater, unless there are other compelling reasons for deviating from 5 minutes.
4. For basins with Tc shorter than 30 minutes, be aware that the computed runoff volume will be accurate but that the model may misstate the peak. Peak rates developed with HEC-HMS for basins with Tc shorter than 30 minutes should always be checked against other methods and experience.
5. Note that shorter and more numerous computation intervals do not always result in better answers (accuracy versus precision).

### **405.9.2 Duration of Simulation**

The model simulation duration (the beginning and ending date and time) should be long enough to capture the entire storm runoff hydrograph. After an initial model run duration of 24 hours, the engineer should review the terminal basin outfall hydrograph to determine if the discharge has returned to zero. If zero discharge is not achieved, extend the model duration and simulate again to obtain zero discharge. Durations greater than 24 hours will generally be required for larger basins (greater than 10 square miles) and for models which contain reservoir routings with long detention times.

### **405.10 Transmission Losses (Channel Losses)**

HEC-HMS has the ability to include the effects of channel losses to the hydrograph. This function is available only in the Modified Puls and Muskingum-Cunge hydrograph routing Methods. Channel losses are included in the “Reach” description within the Basin Model Manager within HEC-HMS. Generally, channel losses do not significantly affect the peak rate of discharge for larger, infrequent flood events, but may have a significant and measurable effect on floods up to the 5-year flood. Therefore, transmission losses should not be considered in the modeling of floods events equal to or greater than the 10-year event. Models constructed for the purpose of evaluating water quality and for determining channel stability and sediment transport will benefit from consideration of transmission losses. If the need to determine the values for use in calculating channel losses on NMDOT projects should arise, use the Percolation Loss/Gain method as outlined in the HEC-HMS User’s Manual (p. 234) and the NRCS, 2007, Part 630 Hydrology National Engineering Handbook, Chapter 19, Appendix 19C “Estimating Transmission Losses When No Observed Data are Available”.

<http://www.hec.usace.army.mil/software/hec-hms/>

<http://www.wcc.nrcs.usda.gov/ftpref/wntsc/H&H/NEHhydrology/ch19.pdf>

### **405.11 Flood Routing**

HEC-HMS offers a total of six hydrologic routing methods for simulating flow in open channels. For most NMDOT project applications, the Muskingum-Cunge Method is the preferred method. HEC-HMS can also include flood hydrograph routings through diversions, reservoirs, and pump stations.

The Muskingum-Cunge Routing Method is based on the combination of the conservation of momentum and the conservation of mass. This Method relates storage to both inflow and outflow discharges from both the channel and floodplain within each analysis reach. This Method is sometimes referred to as a Variable Coefficient Method because routing parameters are recalculated every time step based on channel properties and the flow depth. The computations attempt to simulate the attenuation of flood waves and can be used in reaches with a mild slope.

### **405.12 Model Results Reporting**

Once the model has been run and the results have been checked for reasonableness, the engineer must include the summary results for each storm frequency simulated in the report. See **Figure 405-5** for the HEC-HMS “Global Summary Table”.

Global Summary Results for Run "100yr 24hr Run"

Project: Hydrology    Simulation Run: 100yr 24hr Run

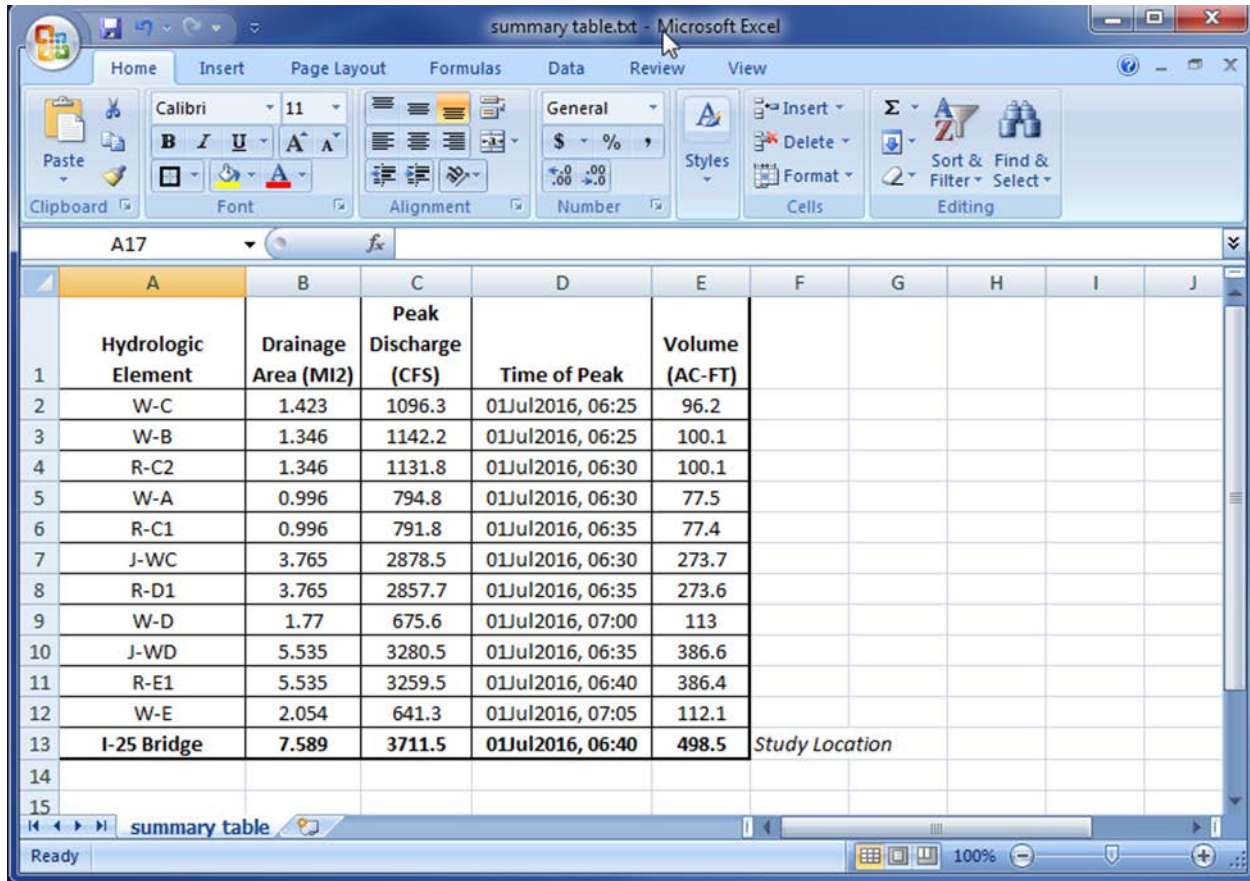
Start of Run: 01Jul2016, 00:00    Basin Model: Ojitos  
 End of Run: 02Jul2016, 00:15    Meteorologic Model: 100yr 24hr Storm  
 Compute Time: 13Jan2016, 09:57:58    Control Specifications: 24hr Storm

Show Elements: All Elements    Volume Units: ☐ IN ☒ AC-FT    Sorting: Hydrologic

Hydrologic Element	Drainage Area (MI <sup>2</sup> )	Peak Discharge (CFS)	Time of Peak	Volume (AC-FT)
W-C	1.423	1096.3	01Jul2016, 06:25	96.2
W-B	1.346	1142.2	01Jul2016, 06:25	100.1
R-C2	1.346	1131.8	01Jul2016, 06:30	100.1
W-A	0.996	794.8	01Jul2016, 06:30	77.5
R-C1	0.996	791.8	01Jul2016, 06:35	77.4
J-WC	3.765	2878.5	01Jul2016, 06:30	273.7
R-D1	3.765	2857.7	01Jul2016, 06:35	273.6
W-D	1.77	675.6	01Jul2016, 07:00	113.0
J-WD	5.535	3280.5	01Jul2016, 06:35	386.6
R-E1	5.535	3259.5	01Jul2016, 06:40	386.4
W-E	2.054	641.3	01Jul2016, 07:05	112.1
I-25 Bridge	7.589	3711.5	01Jul2016, 06:40	498.5

**Figure 405-5 HEC-HMS Global Summary Results Example**

Sort the results in the Global Summary Table using "Hydrologic" order, and also select the "Volume Units" to be in ac-ft. Then the HEC-HMS "Global Summary Table" can be exported as a text file to any number of spreadsheet programs for formatting needs as shown in **Figure 405-6**.



The screenshot shows a Microsoft Excel window titled 'summary table.txt - Microsoft Excel'. The ribbon includes Home, Insert, Page Layout, Formulas, Data, Review, and View. The active cell is A17. The table data is as follows:

	A	B	C	D	E	F	G	H	I	J
	Hydrologic Element	Drainage Area (MI <sup>2</sup> )	Peak Discharge (CFS)	Time of Peak	Volume (AC-FT)					
1	W-C	1.423	1096.3	01Jul2016, 06:25	96.2					
2	W-B	1.346	1142.2	01Jul2016, 06:25	100.1					
3	R-C2	1.346	1131.8	01Jul2016, 06:30	100.1					
4	W-A	0.996	794.8	01Jul2016, 06:30	77.5					
5	R-C1	0.996	791.8	01Jul2016, 06:35	77.4					
6	J-WC	3.765	2878.5	01Jul2016, 06:30	273.7					
7	R-D1	3.765	2857.7	01Jul2016, 06:35	273.6					
8	W-D	1.77	675.6	01Jul2016, 07:00	113					
9	J-WD	5.535	3280.5	01Jul2016, 06:35	386.6					
10	R-E1	5.535	3259.5	01Jul2016, 06:40	386.4					
11	W-E	2.054	641.3	01Jul2016, 07:05	112.1					
12	I-25 Bridge	7.589	3711.5	01Jul2016, 06:40	498.5	Study Location				
13										
14										
15										

**Figure 405-6 HEC-HMS Discharge Summary Table Example**

In addition, a Basin Model map generated in HEC-HMS (**Figure 405-7**) should be included in the report. This can be created simply by utilizing a screen capture program to copy the screen from HEC-HMS. This Basin Model Map is a schematic that is valuable to assist in understanding the model organization, and the order that basin elements were applied to simulate the basin storm runoff.

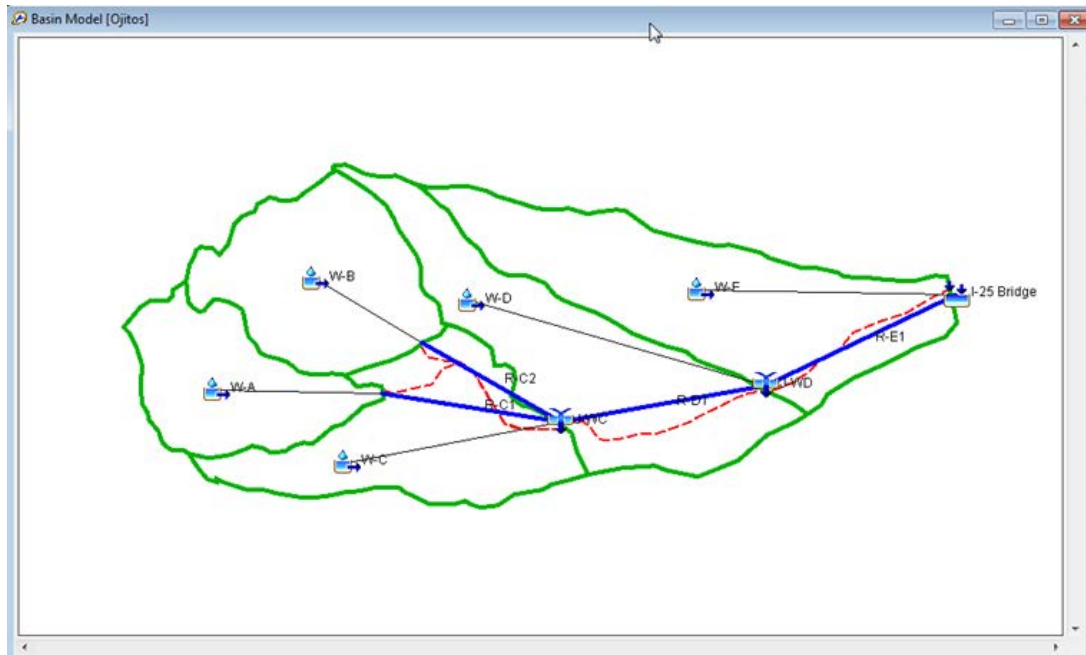


Figure 405-7 HEC-HMS Basin Model Example

The hydrograph shape can be found under the element results (**Figure 405-8**).

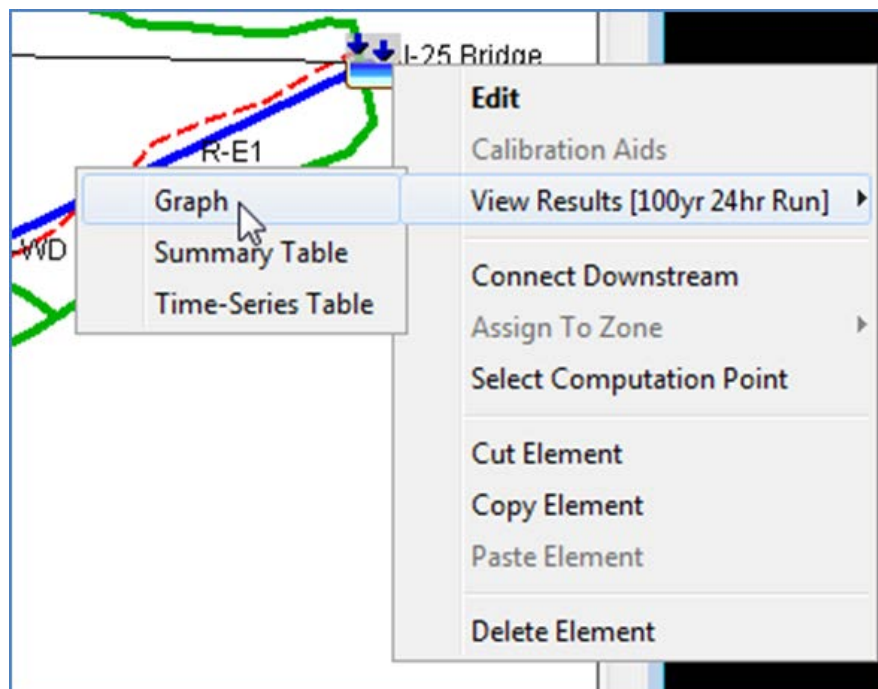


Figure 405-8 HEC-HMS Display Hydrograph Menu Example



The HEC-HMS Method Worksheet (**Figure 405-9**) should be filled out as well.

**HEC-HMS Method Worksheet**

Structure Location: MP: \_\_\_\_\_ County: \_\_\_\_\_  
District: \_\_\_\_\_

Structure Description: \_\_\_\_\_  
Drainage Area: **A** = \_\_\_\_\_ acres \_\_\_\_\_ mi<sup>2</sup>

**Meteorological Model Summary**  
Elevation at Centroid of Watershed: **Elev** = \_\_\_\_\_ ft \*  
Location of Centroid: **Lat**: \_\_\_\_\_ **Long**: \_\_\_\_\_

<b>Design Frequency Flood</b>	_____ - year	_____ - year
24-hour Rainfall Depth (NOAA PFDS):	<b>P<sub>24</sub></b> _____ in.	<b>P<sub>24</sub></b> = _____ in.

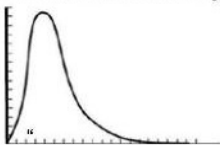
**Basin Model Summary**  
Number of Sub-Basins \_\_\_\_\_  
Curve Number Range Used for modeling Low: \_\_\_\_\_ High: \_\_\_\_\_  
Basin Lag Range Used for modeling Low: \_\_\_\_\_ min High: \_\_\_\_\_ min

**Control Specifications Summary**  
Total Model Duration \_\_\_\_\_ Hrs:Min Time Interval \_\_\_\_\_ (min\hrs)

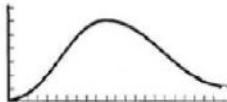
**Summary Output (at Structure Location)**

<b>Design Frequency Flood</b>	_____ - year	_____ - year
Peak Discharge (cfs)	<b>Q</b> = _____ cfs	<b>Q</b> = _____ cfs
Discharge per acre	_____ cfs/ac	_____ cfs/ac
Total Volume (ac-ft)	<b>V</b> = _____ ac-ft	<b>V</b> = _____ ac-ft
Total Runoff (in)	<b>V</b> = _____ in	<b>V</b> = _____ in

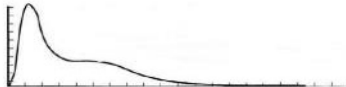
Approximate Outflow Hydrograph Shape:



“Peaky”



“Broad”



“Mixed”

Project Location: \_\_\_\_\_  
CN#: \_\_\_\_\_  
Date: \_\_\_\_\_  
Computed By: \_\_\_\_\_  
Checked By: \_\_\_\_\_

\* If elevation is greater than 7500 ft, use NRCS Unit Hydrograph method

**Figure 405-9 HEC-HMS Method Worksheet**



### 405.13 References

Easterling, Charles, M. 1979, "Urban Watershed Modeling with HYMO", ASCE Irrigation and Drainage Division Specialty Conference".

NOAA Hydrometeorological Design Studies Center Precipitation Frequency Data Server (PFDS).

<http://hdsc.nws.noaa.gov/hdsc/pfds/>

NRCS Web Soil Survey Website.

<https://websoilsurvey.sc.egov.usda.gov/App/HomePage.htm>

NRCS, 2004, "Part 630 Hydrology, National Engineering Handbook, Chapter 9 Hydrologic Soil-Cover Complexes".

<http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=17758.wba>

NRCS, 2007, "Part 630 Hydrology, National Engineering Handbook, Chapter 16 Hydrographs".

<http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=17755.wba>

NRCS, 2007, "Part 630 Hydrology, National Engineering Handbook, Chapter 19 Transmission Losses".

<http://www.wcc.nrcs.usda.gov/ftpref/wntsc/H&H/NEHhydrology/ch19.pdf>

NRCS, 2007, "Part 630 Hydrology, National Engineering Handbook, Chapter 19, Appendix 19C, Estimating Transmission Losses When No Observed Data are Available".

<http://www.wcc.nrcs.usda.gov/ftpref/wntsc/H&H/NEHhydrology/ch19.pdf>

NRCS, 2010, "Part 630 Hydrology, National Engineering Handbook, Chapter 15 Time of Concentration".

<https://directives.sc.egov.usda.gov/27002.wbas>

USACE Hydrologic Engineering Center - HEC-HMS, "HEC-HMS User's Manual", "HEC-HMS Technical Reference Manual", "HEC-HMS Application Guide".

<http://www.hec.usace.army.mil/software/hec-hms/>

USACE, March 2000, "Hydrologic Engineering Center, "HEC-HMS Technical Reference Manual".

[http://www.hec.usace.army.mil/software/hec-hms/documentation/HEC-HMS\\_Technical%20Reference%20Manual\\_\(CPD-74B\).pdf](http://www.hec.usace.army.mil/software/hec-hms/documentation/HEC-HMS_Technical%20Reference%20Manual_(CPD-74B).pdf)

## 406 Watersheds with Stream Gage Data

When considering the use of statistical analysis of gage data for design purposes, it is important to determine if the present watershed conditions are represented by the stream gage record or if there has been a significant change in land use. If there has been a significant increase in urbanization or change in agricultural practices, the historical record may not represent current conditions. While many hydrologic techniques are available for the prediction of frequency of flow events, this section presents concepts and techniques for analyzing peak flows using stream gage data and, to a lesser extent, low flows, following the recommendations of

USGS, England, J.F., Jr., Cohn, T.A., Faber, B.A., Stedinger, J.R., Thomas, W.O., Jr., Veilleux, A.G., Kiang, J.E., Mason, R.R., Jr., 2018, "Guidelines for Determining Flood Flow Frequency, Bulletin #17C, Chapter 5 of Section B, Surface Water, Book 4, Hydrologic Analysis and Interpretation, Techniques and Methods 4-B5".

<https://pubs.er.usgs.gov/publication/tm4B5>

Elements of risk and uncertainty are inherent in any flood frequency analysis. It is possible to standardize many elements of flood frequency analysis, but reliable results are only possible where available records are adequate to warrant statistical analysis of the data.

Flow frequency analysis relates the magnitude of a given flow event with the frequency or probability of that event's exceedance. If a stream gage is available and the conditions applicable, a gage analysis is generally considered preferable to deterministic methods (Rational Formula Method, Simplified Peak Discharge Method or NRCS Unit Hydrograph Method within HEC-HMS). Since a gage represents the actual rainfall-runoff behavior of the watershed in relation to the stream. A variety of Federal, state, and local agencies operate and maintain stream gages. Currently, the USGS operates about 7,000 active stream gaging stations across the country. Data are also available for about 13,000 discontinued gaging stations. Data is available for 155 currently active sites in New Mexico and for a total of 495 sites when the discontinued sites are included.

The USGS has determined station specific flood frequency data for 293 gage locations for recurrence intervals of 2-, 5-, 10-, 25-, 50-, 100- and 500-years that generally have 10 or more years of record (through 2004). Historical peak flow data for both active and discontinued gages can be found at the following USGS website at:

<http://nwis.waterdata.usgs.gov/usa/nwis/peak>.

This information is also found in Appendix 1 of the USGS report prepared for New Mexico in cooperation with the NMDOT: "Analysis of the Magnitude and Frequency of Peak Discharge and Maximum Observed Peak Discharge in New Mexico and Surrounding Areas", Scientific Investigations Report 2008-5119, USGS, Waltermeyer, Scott, D., 2008.

<http://pubs.usgs.gov/sir/2008/5119/>

The USGS has also developed a web-based flood-frequency analysis tool called "PeakFQ-Flood-Frequency Analysis", for determining the stream flood statistics at gaging stations with sufficiently long records. This program is available at:

<https://water.usgs.gov/software/PeakFQ/>

Streamflow data from gages other than USGS gages should not be used for design of NMDOT projects (unless approved by the NMDOT), but may be useful for checking against peak discharge estimates derived from other methods and sources. There are several general scenarios in which data from a non-USGS streamflow gage may be utilized:

1. The gage has been in place for a sufficient number of years (Bulletin 17C recommends at least 10 years)
2. The gage data is reasonably representative of the average watershed conditions during the period of record
3. The gage is located at the highway drainage structure
4. The gage is located upstream or downstream at some distance from the highway

The majority of the gage data in New Mexico has been collected by the USGS. For most of their active streamflow gage sites and many of their inactive sites, the USGS has computed flood frequency estimates. These estimates can be used directly for design if the gage is located at or near (as defined below) the highway crossing. The current USGS study of peak stream flows in New Mexico (USGS, Waltemeyer, Scott, D., 2008) includes tabulated flood frequency estimates for most USGS gage sites in New Mexico.

If the gage data set represents a relatively short period of record, a correction weighting procedure is recommended. The gage frequency distribution peak flood estimate is weighted according to the length of record and equivalent years from the USGS regression analysis. Waltemeyer (USGS, 1996) describes a procedure for improving flood frequency estimates at gaged sites, using USGS regression equations. In the event that the USGS gage at the highway drainage structure was not included in Waltemeyer's study, then a frequency distribution analysis is necessary. A comprehensive discussion of frequency analysis is beyond the scope of this manual. There are several publications which describe the process in great detail. References for two such publications are provided below:

USGS, England et al., 2018, "Guidelines for Determining Flood Flow Frequency, Bulletin #17C, Chapter 5 of Section B, Surface Water, Book 4, Hydrologic Analysis and Interpretation, Techniques and Methods 4-B5".

<https://pubs.er.usgs.gov/publication/tm4B5>

U.S. Army Corps of Engineers, 1993, "Engineering and Design, Hydrologic Frequency Analysis".

[http://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM\\_1110-2-1415.pdf](http://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-1415.pdf)

Typically, a Log-Pearson Type III probability distribution is fit to the set of streamflow data. The use of a partial duration series may be appropriate rather than an annual series depending on data availability and quality.

When the USGS streamflow gage is located on the same stream but some distance upstream or downstream of the highway, the gage site can still be used to provide a weighted flood frequency estimate. The area weighted correction procedure (USGS, Waltemeyer, Scott, D., 1996) includes a drainage area ratio adjustment which can be used when the ratio of ungaged watershed area to gaged watershed area is within the limits 0.5 to 1.5. The following excerpt from Waltemeyer explains that process.

#### **406.1 Ungaged Site on a Stream Having a Nearby Gaging Station**

This information in this section was obtained from "Analysis of the Magnitude and Frequency of Peak Discharge and Maximum Observed Peak Discharge in New Mexico and Surrounding Areas", Scientific Investigations Report 2008-5119, USGS, Waltemeyer, Scott, D., 2008.

<http://pubs.usgs.gov/sir/2008/5119/>

Flood-frequency estimates can be made for ungaged sites upstream or downstream from gaging stations by using a method developed by Sauer (1974). Using this method, flood-frequency data at the gaging station is transferred to the ungaged site by using the following drainage-area ratio adjustment equation:

$$Q_{T(u)} = Q_{T(g)} (A_u / A_g)^x \quad 406-1$$

(USGS, Waltemeyer, Scott, D., 2008, "Analysis of the Magnitude and Frequency of Peak Discharge and Maximum Observed Peak Discharge in New Mexico and Surrounding Areas", Scientific Investigations Report 2008-5119, Eq. 3, p.11)

where:

$Q_{T(u)}$	=	weighted flood-frequency estimate at the ungaged site, ft <sup>3</sup> /s
$Q_{T(g)}$	=	flood-frequency estimate at the gaging station, ft <sup>3</sup> /s
$A_u$	=	drainage area at the ungaged site, square miles
$A_g$	=	drainage area at the gaging station, square miles
$x$	=	exponent of the drainage area of the applicable regional regression equation is listed in Table 2 found on pages 9 and 10 of the USGS document "Analysis of the Magnitude and Frequency of Peak Discharge and Maximum Observed Peak Discharge in New Mexico and Surrounding Areas", by Scott D. Waltemeyer 2008

According to Sauer (1974), the equation is applicable when the drainage-area ratio ( $A_u/A_g$ ), is between 0.5 and 1.5. For example, to estimate a 50-year peak discharge at an ungaged site in Region 2 upstream from gaging station Cisco Wash near Cisco, Utah (09163700), the station value listed in Appendix 1 is 4,670 ft<sup>3</sup>/s. Note that the weighted value of 5,500 ft<sup>3</sup>/s was not used because when using this technique, a regional adjustment is made by using the exponent from the regional equation. The weighted value is considered the best flood-frequency value, but when using this technique, a double weight would be made based on the regional flood information. The drainage area at the gaging station is 90.7 square miles (Appendix 1, USGS, 2008). The 50-year recurrence interval regression equation exponent for the drainage area is 0.308 for Region 2 (Table 2, USGS, 2008). The drainage area at the ungaged site is 75.5 square miles, and when equation 4 (USGS, 2008) is used (equation below), the peak discharge at the ungaged site is:

$$Q_{50u} = Q_{50g} (A_u / A_g)^x \quad 406-2$$

$$Q_{50u} = (4,670) (75.5 / 90.7)^{0.308} = 4,410 \text{ ft}^3/\text{s}$$

(USGS, Waltemeyer, Scott, D., 2008, "Analysis of the Magnitude and Frequency of Peak Discharge and Maximum Observed Peak Discharge in New Mexico and Surrounding Areas", Scientific Investigations Report 2008-5119, Eq. 3, p.12)

<http://pubs.usgs.gov/sir/2008/5119/>

Note: The USGS has developed a web application called "StreamStats". StreamStats incorporates a Geographic Information System (GIS) to provide users with access to an assortment of analytical tools that are useful for a variety of water resources planning and management purposes, and for engineering and design purposes.

<https://water.usgs.gov/osw/streamstats/>

## 406.2 References

Sauer, V.B., 1974, "Flood Characteristics of Oklahoma Streams: U.S. Geological Survey Water-Resources Investigations Report 52-73".

U.S. Army Corps of Engineers, March 1993, "EM 1110-2- 1415 Hydrologic Frequency Analysis".  
[http://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM\\_1110-2-1415.pdf](http://www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-1415.pdf)

USGS, Website, "PeakFQ-Flood-Frequency Analysis".  
<https://water.usgs.gov/software/PeakFQ/>

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<https://water.usgs.gov/osw/streamstats/>

USGS, Website, "Stream Flow Gage Data, Active and Discontinued Gages"  
<http://nwis.waterdata.usgs.gov/usa/nwis/peak>

USGS, Waltemeyer, Scott, D., 1996, "Analysis of the Magnitude and Frequency of Peak Discharge and Maximum Observed Peak Discharge in New Mexico", Water-Resources Investigations Report 96-4112.  
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<https://pubs.er.usgs.gov/publication/tm4B5>

USGS, Waltemeyer, Scott, D., 2008, "Analysis of the Magnitude and Frequency of Peak Discharge and Maximum Observed Peak Discharge in New Mexico and Surrounding Areas", Scientific Investigations Report 2008-5119.  
<http://pubs.usgs.gov/sir/2008/5119/>

## 407 Statistical Methods in Watersheds without Stream Gage Data

The USGS's (Waltemeyer, 2008) report titled "Analysis of the Magnitude and Frequency of Peak Discharge and Maximum Observed Peak Discharge in New Mexico and Surrounding Areas", Scientific Investigations Report 2008-5119, was prepared in cooperation with the NMDOT. The report summarized the analyses and equations developed for estimating peak discharges for recurrence intervals of 2-, 5-, 10-, 25-, 50-, 100- and 500-years at ungaged sites by use of data collected through 2004 for 293 gaging stations on unregulated streams that have 10 or more years of record.

The regional flood frequency equation values shown in Table 2 of the above-referenced report list the "Average Standard Error of Estimates" for each of the nine hydrologic regions and for recurrence intervals of 2-, 5-, 10-, 25-, 50-, 100- and 500-years. Flood magnitude estimates from the USGS are based on information collected from stream gage data as well as from estimates of flood magnitude using high water marks and eyewitness accounts when gages

were damaged or destroyed by the flood. Many records are relatively short compared to the exceedance frequency projected by the statistics. There are also inherent accuracy problems with some of the data collected by means other than from a properly functioning gage. Hence the estimates produced may differ from those that would have been produced if the records were long and accurate.

It is important to consider the Standard Error when using USGS regression estimates as it affects the accuracy of the estimates and, therefore, the reliance that can be placed on the interpretations drawn from the data.

The USGS states in the above-referenced report: The average Standard Error of prediction, which includes average sampling error and average Standard Error of regression, ranged from 38 to 93 percent (mean value is 62, and median value is 59) for the 100-year flood. The 1996 investigation Standard Error of prediction for the flood regions ranged from 41 to 96 percent (mean value is 67, and median value is 68) for the 100-year flood that was analyzed by using generalized least-squares regression analysis. Overall, the equations based on generalized least-squares regression techniques are more reliable than those in the 1996 report because of the increased length of record and improved geographic information system (GIS) method to determine basin and climatic characteristics.

The Standard Error measure indicates the extent to which a regression estimate is likely to deviate from the true population and is expressed as a number. The Relative Standard Error (RSE) is the Standard Error expressed as a fraction of the estimate and is usually displayed as a percentage. Estimates with a RSE of 25% or greater are subject to high sampling and regression error and should be used with caution.

The average Standard Error of estimates listed in Table 2 of the above referenced USGS report all exceed 25% (with some exceeding 100%). Therefore, the use of the USGS regional regression equations for New Mexico should be limited to:

1. Determination that the peak discharges calculated using one of the three approved hydrologic peak discharge analyses methods are within reason and supported by the exercise of judgment, and
2. For very preliminary peak discharge estimation when scoping a project
3. USGS regional regression equations may be used for design when checked against one of the hydrologic peak discharge analysis methods and approved by the NMDOT Drainage Engineer

The tabulation of maximum observed peak discharges for sites within each of the nine hydrologic regions around New Mexico are listed in Appendix 3 of the Waltemeyer 2008 report. The engineer is encouraged to review that Appendix when performing drainage analyses to gain further understanding of the hydrologic response of the various regions around the state. An excerpt from Appendix 3 is shown below (**Figure 407-1**) for reference.



**Appendix 3.** Miscellaneous sites, maximum peak discharge, drainage area, and site elevations.

[DMS, degrees minutes seconds; NGVD 29, National Geodetic Vertical Datum of 1929]

Map identi- fier (fig. 2)	Latitude (DMS)	Longi- tude (DMS)	Miscellaneous site name	Date	Maximum peak discharge (cubic feet per second)	Drainage area (square miles)	Site elevation (feet above NGVD 29)	Region
1	362958	1030731	Apache Creek at State Highway 18 near Clayton, New Mexico	1952	2,500	50.8	4,780	1
2	354906	1034621	Arroyo del Alamo near Mosquero, New Mexico	05/16/54	3,440	27.4	4,380	1
3	350110	1040426	Arroyo Laguna tributary near Montoya, New Mexico	07/05/60	2,660	3.40	4,460	1
4	350216	1033407	Barranca Creek near Norton, New Mexico	08/23/59	3,870	1.47	4,040	1
5	350714	1035416	Blanco Creek tributary at Palomas, New Mexico	07/05/60	1,540	2.90	4,330	1
6	365451	1030515	Bontz Arroyo near Guy, New Mexico	07/06/58	2,410	4.90	4,450	1
7	364546	1042933	Canadian River tributary near Hebron, New Mexico	06/17/65	2,130	2.01	6,300	1
8	364818	1030100	Carrizozo Creek tributary near Moses, New Mexico	07/06/58	307	0.15	4,730	1
9	352646	1034249	Carros Creek near Gallegos, New Mexico	07/08/60	2,590	9.40	4,050	1
10	361550	1031235	Carrizo Creek near Clayton, New Mexico	05/28/57	29,500	305	4,780	1
11	365210	1042246	Chicorica Creek near Raton, New Mexico	06/17/65	12,800	78.8	6,460	1
12	365100	1042125	Chicorica Creek tributary near Raton, New Mexico	06/17/65	1,810	1.33	6,470	1
13	362122	1032812	Carrizo Creek above State Road 56 near Clayton, New Mexico	06/17/65	9,270	305	5,245	1
14	353616	1045352	Conchas River tributary near Trujillo, New Mexico	07/05/67	4,530	3.43	6,480	1
15	353622	1045208	Conchas River near Trujillo, New Mexico	07/05/67	9,810	18.6	6,430	1
16	364216	1043536	Crow Creek below Waldron Canyon near Koehler, New Mexico	06/17/65	30,400	59.8	6,260	1
17	363755	1043225	Crow Creek near Maxwell, New Mexico	06/17/65	13,100	78.4	6,030	1
18	351813	1041843	Cuervo Creek near Conchas Dam, New Mexico	06/18/69	12,800	135	4,280	1
19	360020	1033331	Del Muerto Creek near Bueyeros, New Mexico	07/16/72	34,600	29.0	4,690	1
20	365540	1042130	East Fork Chicorica Creek at Yankee, New Mexico	06/17/65	13,500	22.7	6,780	1
21	351256	1031216	Frost Creek near Porter, New Mexico	07/16/58	2,910	10.0	3,900	1
22	354430	1040722	La Cinta Creek near Roy, New Mexico	08/17/77	13,300	116.5	4,700	1
23	361249	1043900	Ocate Creek at Colmar, New Mexico	07/04/51	25,000	434	5,900	1
24	354113	1030746	Minneosa Creek near Nara Visa, New Mexico	07/23/54	20,400	118	4,300	1
25	345838	1034502	Paris Creek near Quay, New Mexico	07/17/56	3,410	9.20	4,220	1

Source: USGS, Waltemeyer, Scott, D., 2008, "Analysis of the Magnitude and Frequency of Peak Discharge and Maximum Observed Peak Discharge in New Mexico and Surrounding Areas", Appendix 3, p. 91.

<http://pubs.usgs.gov/sir/2008/5119/>

**Figure 407-1 USGS Appendix 3 Excerpt**

## 407.1 References

USGS, Waltemeyer, Scott, D., 2008, "Analysis of the Magnitude and Frequency of Peak Discharge and Maximum Observed Peak Discharge in New Mexico and Surrounding Areas", Scientific Investigations Report 2008-5119.

<http://pubs.usgs.gov/sir/2008/5119/>

## 408 Risk and Uncertainty in Hydrologic Analysis

Highway drainage structures are designed to safely pass a certain magnitude flood. On most New Mexico highways, the Design Flood will be the "50-year" frequency flood. This flood is theoretically equivalent to the largest flood which will occur at that location on average at least

once every fifty years. By designing drainage structures to safely pass relatively rare events, the risk to users of the highway is reduced to an acceptable level. There is always some chance, or risk, that a flood will occur which exceeds the design flood used to size a particular drainage structure. While it might be desirable to design all drainage structures to pass the largest possible flood, economic realities prevent this option. Instead, a level of protection must be provided which is both responsible and reasonable.

Design exceptions or variances may be required as a result of budget impacts, right-of-way limitations, environmental and property impacts, or other constraints. Such variances are only allowed when all other options have been considered and found to be inadequate. If deviation from the criteria and design standards for major drainage structures or systems is necessary, a risk assessment may be required. If a jurisdiction or organization has more stringent criteria than the NMDOT criteria, those criteria shall govern the drainage design. Even though the 50-year flood occurs on average at least once every 50 years, there is some small, but very real possibility (2% chance) that this flood could occur in any given year. Stated another way, just because a 50-year flood occurred last year, does not mean that it could not occur again this year. The probability of a 50-year flood occurring or being exceeded this year and every year is remains at 2%.

In order to better quantify the risk associated with a certain design frequency the following example is provided:

Consider a drainage structure capable of passing the 100-year frequency event with a structural design life of 50-years. What is the probability or risk, that the structure will see a 100-year flood (or greater) during its design life? The logical answer might be 1 chance in 2, or 50%. However statistical analyses show that the risk is lower, actually at 39.5%. Statistically, the concept of risk is described by a binomial distribution

USGS, England et al., 2018, “Guidelines for Determining Flood Flow Frequency, Bulletin #17C, Chapter 5 of Section B, Surface Water, Book 4, Hydrologic Analysis and Interpretation, Techniques and Methods 4-B5”.

<https://pubs.er.usgs.gov/publication/tm4B5>

**Equation 408-1** describes this statistical relationship.

$$R = 1 - \left(1 - \frac{1}{T_r}\right)^m \times 100 \quad 408-1$$

where:

R	=	the risk of design discharge being exceeded at least once during the design life, percent
$T_r$	=	the recurrence interval or frequency of the design flood, years
m	=	the design life of the structure, years

$$R = 1 - \left(1 - \frac{1}{100}\right)^{50} \times 100 = 39.5\% \text{ for the example above.}$$



Assuming that the structure is designed for the 50-year flood and has a design life of 50 years, then **Equation 408-1** predicts that the structure's capacity has a 63.6% chance of being equaled or exceeded during the structure's design life.

$$R = 1 - \left(1 - \frac{1}{50}\right)^{50} \times 100 = 63.6\%$$

**Table 408-1** lists computed values of risk for a range of structure design lives.

**Table 408-1 Tabulation of Risk of at Least One Exceedance during the Design Life**

Recurrence Interval	Design Life - Years					
	2	5	10	25	50	100
2	75.0%	97.0%	100.0%	100.0%	100.0%	100.0%
5	36.0%	67.0%	89.0%	100.0%	100.0%	100.0%
10	19.0%	41.0%	65.0%	93.0%	99.0%	100.0%
25	8.0%	18.0%	34.0%	64.0%	87.0%	98.0%
50	4.0%	10.0%	18.0%	40.0%	64.0%	87.0%
100	2.0%	5.0%	10.0%	22.0%	39.0%	63.0%
500	0.4%	1.0%	2.0%	5.0%	10.0%	18.0%
1000	0.2%	0.5%	1.0%	2.0%	5.0%	10.0%

Another way of looking at the concept of risk is to define an acceptable level of risk and then compute the design flood which would have to be accommodated by the drainage structure to satisfy that level of risk. **Equation 408-1** can be rearranged to solve for the required return period, yielding **Equation 408-2**.

$$Tr = \frac{1}{1 - \left(1 - \frac{R}{100}\right)^{1/m}} \quad 408-2$$

Assume that a 10% level of risk is desirable, or stated another way, there is a 90% confidence level that the structure is adequate. Then **Equation 408-2** predicts that the structure with the design life of 50 years must be capable of passing the 475-year flood.

$$Tr = \frac{1}{1 - \left(1 - \frac{10}{100}\right)^{\frac{1}{50}}} = 475 \text{ years}$$

It becomes apparent that risk cannot be completely eliminated, but may be reduced to a level acceptable to society. Even if there were unlimited funds to build drainage structures, the ability to accurately calculate the magnitude of flood events decreases as the design flood magnitude increases. All of the current flood prediction methods, whether analytical or parametric, are based on observed flood flows from watersheds with measured response characteristics, and

occasionally rain gage data. The effective period of recorded data in New Mexico reaches 100 years in only a few locations. Thus, the prediction of a 475-year flood is done by extrapolating the data, since the desired flood has only a small chance of being included in the data set. The uncertainty in predicted flood flows increases as the return period lengthens.

The accuracy of predicted flood magnitudes up to the 100-year event is, while not perfect, certainly much better. For the analytic methods presented in this manual, risk takes the form of uncertainty in the input parameters. A drainage area can be measured by multiple engineers and the answers from each, should all be within two or three percent. Use of a consistent method to compute  $T_c$  reduces variability in the estimation of  $T_c$ . However, the selection of a Rational Formula Method Runoff Coefficient "C", or a NRCS Runoff Curve Number "CN" involves considerable judgement. Even meticulous measurement of watershed areas, land uses, and Hydrologic Soil Groups may not accurately describe the response of the watershed for every storm. There is some inherent variability of the data, and of its interpretation, leading to uncertainty in the selection of the correct "C" or "CN". This uncertainty cannot be universally quantified, and thus becomes part of the overall risk and uncertainty in predicting peak flood magnitudes.

With the analytic methods in this manual, one approach to qualitatively assess the risk is to perform a sensitivity analysis. This is done by varying a particular input parameter across its range of reasonable values and comparing the resulting range of predicted peak flows. The most sensitive analytic parameter in larger watersheds will probably be the "C" or "CN". Use the "C" or "CN" value obtained by normal design methods to compute a peak flow, as well as the lowest and highest "C" or "CN" values which could occur in the watershed. (Note: In small watersheds,  $T_c$  can be the most sensitive input value, but the process is the same.)

The resulting three computed peak flow values provide an estimate of the range of most probable peak flood flows. This is not a precise computed range of risk, but it does help to bracket the most likely peak flow value. The middle peak flood flow value will often be used to size the structure, while the upper limit peak flood flow can be used to assess the "worst case" headwater or overtopping condition. If the risk and consequences of an overtopping or significant backwater are unacceptably adverse to the roadway or nearby property, consider an alternate design.

## 408.1 Reference

USGS, England, J.F., Jr., Cohn, T.A., Faber, B.A., Stedinger, J.R., Thomas, W.O. Jr., Veilleux, A.G., Kiang, J.E., Mason, R.R., Jr., 2018, "Guidelines for Determining Flood Flow Frequency, Bulletin #17C, Chapter 5 of Section B, Surface Water, Book 4, Hydrologic Analysis and Interpretation, Techniques and Methods 4-B5".

<https://pubs.er.usgs.gov/publication/tm4B5>

[https://water.usgs.gov/osw/bulletin17b/dl\\_flow.pdf](https://water.usgs.gov/osw/bulletin17b/dl_flow.pdf)

# 500 HYDRAULICS

## 501 Introduction

### 501.1 Overview

This Section presents the most common hydraulic equations and analysis methods that should be applied to NMDOT projects. References are provided that will assist in obtaining additional information, equations, or methods that may be required for specific situations not presented in this section. The hydraulic structures and related topics included in this section are listed here:

- Open channels
- Culverts
- Pavement related drainage facilities
  - Curb and gutter
  - Roadside and median channels
  - Rundown channels
  - Bridge deck surface drainage
  - Concrete wall barrier and concrete barrier railing
  - Drop inlets
  - Median drop inlets
  - Catch basins
  - Storm drains
  - Manholes
  - Hydraulic grade line (HGL) analysis discussion
- Underdrains
- Bridges
- Stormwater storage facilities
- Construction and maintenance considerations

There is overlap in the information required for hydraulic analyses and subsequent design due to the interrelationship of the various components of a hydraulic system. Comprehensive data and information collection for the entire drainage system at the project inception will minimize assumptions required to complete the analyses and design.

### 501.2 Field Surveys

Accurate topographic surveys are required prior to hydraulic analyses. The NMDOT Project Development Engineer (PDE) is required to include the drainage engineer in the topographic survey scoping effort. This pre-survey coordination will assist in the development of a complete survey that will be a key component of the drainage analysis and design effort.

The drainage engineer is responsible for verification of the survey to determine if it accurately depicts the actual field conditions with respect to hydraulic information requirements. If required, the engineer should request corrected or additional field data from the surveyors prior to the start of analysis and design.

As part of the overall field survey, the engineer should visit the site prior to the start of the design process and conduct the following tasks as appropriate for the project scope:

- Take color photographs of the watercourse bed and banks upstream and downstream of the highway crossing (particularly bed armor locations, head cuts, severe erosion, and bank failure locations, etc.)
- Collect sediment samples at appropriate locations as required for sediment transport analyses or corrosivity analyses
- Record plant species

The photographs provide a record of the existing conditions and provide a good resource during hydraulic analysis of existing or proposed structures. There are many items that are required to be prepared for and many items that should be considered during the field visit. **APPENDIX 3** contains a Survey Checklist that identifies watercourse and structure survey information requirements. This checklist should be filled out and provided to the surveyor with the intent of communicating the information required by the engineer. In addition, an annotated map would be valuable information to provide to the surveyor. The map could indicate survey reach lengths, widths, and specific locations or features that should be surveyed.

### **501.3 Record Drawings or Proposed Plans**

Record Drawings must be acquired and reviewed. These will provide valuable information particularly for culvert and bridge projects as the drawings may provide information regarding watercourse bed elevation or cross section changes. Proposed plans must be acquired as soon as possible in order to define required survey limits and information and to begin hydraulic analyses.

## **502 Hydraulics of Open Channels**

### **502.1 Introduction**

The following overview of basic open channel hydraulics is geared toward design professionals with knowledge of engineering hydraulics. The engineer is encouraged to consult the references for additional information, clarification, and derivation of the formulae presented herein.

The following information is typically required for understanding and conducting open channel analyses:

- Base flow rates of perennial streams and rivers
- Design Flood and Check Flood peak flow rates
- Channel slope, geometry, and geomorphology
- Roughness coefficients for channel and adjacent floodplain
- Composition and gradation of channel bed and bank material
- Extent and location of bank armoring, if any
- Field assessment of the bank and bed stability for a considerable distance as needed and practical upstream and downstream of the roadway crossing
- Location of bed head cuts, if any

- Characteristics of contributing watershed(s) and tributary channel(s) including:
  - Land use type and areal extent. Uses may include various types of human activities or development types
  - Upstream stormwater detention, lined or unlined channels, or other large drainage infrastructure
- History of channel meander patterns
- Design capacity, including freeboard for man-made channels
- Existing utility locations

## 502.2 Manning's Equation

Flow in an open channel may occur in several different regimes. Flow characteristics may be constant or variable with time or variable with distance along the channel. Steady flow occurs when, at a given location in the channel, the flow characteristics are constant over time. Unsteady flow occurs when one or more characteristics change over time, such as discharge, the depth of flow or the velocity. Uniform flow occurs when the gravity forces driving flow downstream balance with the resistance-to-flow forces such as friction. This balance induces constant velocity and therefore a constant flow cross section along the length of the channel. Non-uniform or varied flow occurs when gravity and resistance forces are out of balance causing the flow to have acceleration or deceleration, and thus a changing flow cross section.

Flow conditions are generally described by a combination of the temporal and spatial occurrence of flow. For example, flow can be steady, uniform flow where flow conditions are constant over time and along the length of the channel. Although this condition seldom occurs in natural channels, it is easily achieved in a laboratory flume where controlled experiments can be performed and equations, such as Manning's Equation, can be validated.

Throughout this manual, Manning's Equation is used in various forms to obtain values for velocity (V), flow rate (Q), depth of flow (y), and other channel parameters under steady and uniform flow conditions.

Manning's Equation is:

$$V = \frac{1.49}{n} R^{2/3} S^{1/2} \quad 502-1$$

where:

V	=	velocity, ft/s
R	=	hydraulic radius, = A / P, ft
A	=	channel cross sectional area, ft <sup>2</sup>
P	=	wetted perimeter, ft
S	=	slope of the channel, ft/ft
n	=	Manning's roughness coefficient

### Manning's Roughness Coefficient "n"

Manning's "n" values can be obtained from most hydraulic texts and references. Chow (1959) contains an extensive list of "n" values. Two additional references are by H.H. Barnes (1987), which includes color photograph examples, and G.V. Arcement (1989).

**Table 502-1** through **Table 502-5** present "n" values for various materials and bed forms. The variable  $D_{50}$  indicates the sediment at which 50% of the sample is finer by weight.

**Table 502-1** presents a partial list of typical Manning's "n" values from the well-known text book titled "Open-Channel Hydraulics" by Chow, 1959.

**Table 502-1 Typical Values of Manning's "n"**

Source: Chow, 1959, Table 5-5, p.109.

<b>Pipes flowing partially full</b>	<b>Manning's "n"</b>
Corrugated metal	0.024
Concrete	0.013
Vitrified clay	0.015
<b>Man-made channels (lined)</b>	
Steel	0.012
Timber	0.012
Concrete (trowel finish) / Concrete (float finish)	0.013 / 0.015
Masonry or brickwork	0.015
Rubble masonry	0.025
Asphalt	0.013 - 0.016
Earth (clean after weathering)	0.022
Earth (with vegetation)	0.027 - 0.035
<b>Natural Channels</b>	
Clean and straight	0.030
Winding with some pools and shoals	0.040
Very weedy, deep pools	0.100
Mountain streams	0.040 - 0.050
Major streams (width greater than 100 ft at flood stage)	0.025 - 0.100

**Table 502-2** presents typical values of “n” for alluvial sand bed channels.

**Table 502-2 Typical Values of Manning’s “n” for Alluvial Sand Bed Channels**

Source: FHWA, April 2012, HEC-20, Table 3.3, p. 3.13.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12004.pdf>

<b>Alluvial Sand Bed Channels (no vegetation) (a)</b>	
<b>Flow Regime</b>	<b>Manning’s “n”</b>
<b>Lower Flow Regime</b>	
Plane bed	0.014 - 0.020
Ripples	0.018 - 0.030
Dunes	0.020 - 0.040
<b>Transition</b>	
Washed out dunes	0.014 - 0.025
<b>Higher Flow Regime</b>	
Plane bed	0.010 - 0.013
Standing waves	0.010 - 0.015
Antidunes	0.12 - 0.020

(a) Data is limited to sand channels with  $D_{50} < 1.0$  mm.

$D_{50}$  is defined as 50% of the sample is finer by weight.

The value of “n” can also vary with flow depth. Shallow flows lead to larger “n” values relative to deeper flows because the surface has more effect on the flow profile. Typical values of “n” for various channel lining materials, and in relation to flow depth are given in **Table 502-3**.

**Table 502-3 Manning's “n” for Selected Linings and Flow Depths**

Source: FHWA, September 2005, HEC-15, Tables 2.1 and 2.2, p. 2-3.

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/05114/05114.pdf>

		Manning's “n”		
Lining Category	Lining Type	Maximum	Typical	Minimum
Rigid	Concrete	0.015	0.013	0.011
	Grouted Riprap	0.040	0.030	0.028
	Stone Masonry	0.042	0.032	0.030
	Soil Cement	0.025	0.022	0.020
	Asphalt	0.018	0.016	0.016
Unlined	Bare Soil	0.025	0.020	0.016
	Rock Cut (smooth, uniform)	0.045	0.035	0.025
Erosion Control Products	Open-weave textile	0.028	0.025	0.022
	Erosion control blankets	0.045	0.035	0.028
	Turf reinforcement mat	0.036	0.030	0.024
		Manning's “n” for Selected Flow Depths		
Lining Category	Lining Type	0.5 ft (0.15 m)	1.6 ft (0.50 m)	3.3 ft (1.0 m)
Gravel Mulch	D <sub>50</sub> = 25 mm (1 in.)	0.040	0.033	0.031
	D <sub>50</sub> = 50 mm (2 in.)	0.056	0.042	0.038
Cobbles	D <sub>50</sub> = 0.10 m (0.33 ft)	-- (a)	0.055	0.047
Rock Riprap	D <sub>50</sub> = 0.15 m (0.5 ft)	-- (a)	0.069	0.056
	D <sub>50</sub> = 0.30 m (1.0 ft)	-- (a)	-- (a)	0.080

(a) Refer to HEC-15 for additional information regarding values in this table.



**Table 502-4** presents typical values of “n” for other common materials.

**Table 502-4 Typical Values of Manning’s “n” for Other Materials**

Source: From various sources as listed below

<b>Material</b>	<b>Manning’s “n”</b>
Corrugated Polyethylene (PE) –smooth interior (a)	0.009 – 0.015
Corrugated Polyethylene (PE) – corrugated interior (a)	0.018 – 0.025
Polyvinyl Chloride PVC – with smooth inner walls (a)	0.009 – 0.011
Corrugated Metal Pipe (helical corrugations) (a)	
2-2/3 by ½ in.	0.011-0.023
6 by 1 in.	0.022-0.025
Corrugated Metal Pipe, Pipe-Arch, and Box (annular corrugations) (a)	
2-2/3 by ½ in.	0.022-0.027
5 by 1 in.	0.025-0.026
3 by 1 in.	0.027-0.028
Corrugated Metal Structural Plate (annular corrugations)	
6 by 2 in.	0.033-0.035
9 by 2 ½ in.	0.033-0.037
Gabions and reno mattresses (b)	See note 1
Shotcrete (c)	0.017
<sup>1</sup> Roughness characteristics of gabion mattresses are governed by the size of the rock in the baskets and the wire mesh enclosing the rock. For practical purposes, the effect of the mesh can be neglected. Therefore, Manning’s roughness should be determined using the D <sub>50</sub> of the basket rock.	

(a) Source: FHWA, April 2012, HDS-5, Table B.1, p. B-6

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/12026/hif12026.pdf>

(b) Source: FHWA, September 2005, HEC-15, p. 7-1

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/05114/05114.pdf>

(c) FHWA, June 2008, HDS-4, Table B.2, p. B-2

[https://www.fhwa.dot.gov/engineering/hydraulics/pubs/08090/HDS4\\_608.pdf](https://www.fhwa.dot.gov/engineering/hydraulics/pubs/08090/HDS4_608.pdf)

**Table 502-5** presents typical values of “n” for street and pavement gutters. Note that **Table 502-3** provides a range of “n” values for many of the same items presented here.

**Table 502-5 Manning’s “n” for Street and Pavement Gutters**

Source: FHWA, August 2013, HEC-22, Table 4-3, p. 4-9.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

Type of Gutter or Pavement	Manning’s “n”
Concrete gutter, troweled finish	0.012
Asphalt pavement:	---
Smooth texture	0.013
Rough texture	0.016
Concrete gutter with asphalt pavement:	---
Smooth	0.013
Rough	0.015
Concrete pavement:	---
Float finish	0.014
Broom finish	0.016
For gutters with small slope, where sediment may accumulate, increase above values of “n” by	0.002

### Roughness Variation in Natural Channels

Natural channels are usually characterized by a main channel, banks, and overbanks. Flow in the natural channel may occur in all of these areas during infrequent storm events such as the 25-year, 50-year, or 100-year storms. In these instances, one value of Manning’s “n” is insufficient to describe the watercourse roughness. The variation with horizontal distance across the watercourse cross section should be simulated. The HEC-RAS River Analysis System program (USACE, current download) allows simple horizontal distance and “n” value data as input for each cross section of the watercourse. Other methods, and the reasons for computing “n” for other situations and conditions are presented in the following sections.

USACE, Current Program Download, “HEC-RAS River Analysis System”.

<http://www.hec.usace.army.mil/software/hecras/downloads.aspx>

USACE, Current Download, “HEC-RAS River Analysis System – Users Manual”.

<http://www.hec.usace.army.mil/software/hecras/documentation.aspx>

USACE, Current Download, “HEC-RAS River Analysis System – Hydraulic Reference Manual”.

<http://www.hec.usace.army.mil/software/hecras/documentation.aspx>

### Sand Bed Flow Resistance in Sand Bed Watercourses

Bed form may be comprised of either ripples, dunes, or both. The flow resistance or Manning’s “n” due to bed form can often be greater than the “n” for the sand material, considered here to

be less than 2 mm in diameter. The interaction between the flow and bed material and the interdependency among the variables causes the analysis of flow in alluvial sand bed streams to be extremely complex. The explanation of the various flow regimes and bed forms that occur with each regime is presented in detail in “Highways in the River Environment” Chapter III, pg. III-14, (FHWA, February 1990). Please refer to that document for more information.

FHWA, February 1990, "Highways in the River Environment".  
<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hire1990.pdf>

### Coarse Bed Flow Resistance in Coarse Bed Watercourses

Coarse bed material is considered here to be greater than 2 mm in diameter (# 10 sieve). Refer to Chapter III, p. III-33 of “Highways in the River Environment” (FHWA, February 1990) for further discussion and equations.

FHWA, February 1990, "Highways in the River Environment".  
<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hire1990.pdf>

### Resistance to Flow Considering Other Factors

The general approach to estimating resistance to flow in a stream is to estimate a single roughness coefficient “n”. However, an alternative approach that requires more effort, accounts for adding corrections to the base “n<sub>b</sub>” value to account for other factors that are presented here.

$$n = (n_b + n_1 + n_2 + n_3 + n_4) \times m \quad 502-2$$

(FHWA, April 2012, HEC-20, Eq. 3.9, p. 3.10)  
<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12004.pdf>

where:

n <sub>b</sub>	=	base value for a straight, uniform channel
n <sub>1</sub>	=	value for the surface irregularities of the cross section
n <sub>2</sub>	=	value for the variation in the shape and size of the channel
n <sub>3</sub>	=	value for obstructions
n <sub>4</sub>	=	value for vegetation and flow conditions
m	=	correction factor for sinuosity of the channel

Refer to HEC-20 (FHWA, April 2012, p. 3.10) for further information regarding this approach and for required “n” value tables.

FHWA, April 2012, “HEC-20, Stream Stability at Highway Structures, Fourth Edition”.  
<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12004.pdf>

### Discharge Based on Manning's Equation

Manning's Equation computes velocity (V), which is one component of discharge computations. Flow area (A) is the second component required to compute discharge (Q) that represents flow rate as volume per time, as cubic feet per second (ft<sup>3</sup>/s).

The discharge equation follows.

$$Q = A V \quad 502-3$$

where:

Q	=	discharge, ft <sup>3</sup> /s
A	=	flow area, ft <sup>2</sup>
V	=	velocity, ft/s

#### 502.2.1 Specific Energy

The specific energy (E) is defined as the normal depth of flow ( $y_o$ ) plus the velocity head ( $V^2/2g$ ).

$$E = y_o + \frac{V^2}{2g} \quad 502-4$$

(Henderson, 1966, Eq. 2-4, p. 31)

where:

E	=	the specific energy, ft
$y_o$	=	the normal depth of flow, ft
V	=	flow velocity (= Q/A), ft/s
g	=	gravitational acceleration, 32.2 ft/s <sup>2</sup>

For the purpose of illustrating the concept of specific energy (E), assume a wide rectangular channel. The flow rate can be approximated by flowrate per unit width (q), and the flow cross sectional area (A) can be approximated by the flow depth (y). Now **Equation 502-4** can be rewritten as:

$$E = y + \frac{1}{2g} \left( \frac{q}{y} \right)^2 \quad 502-5$$

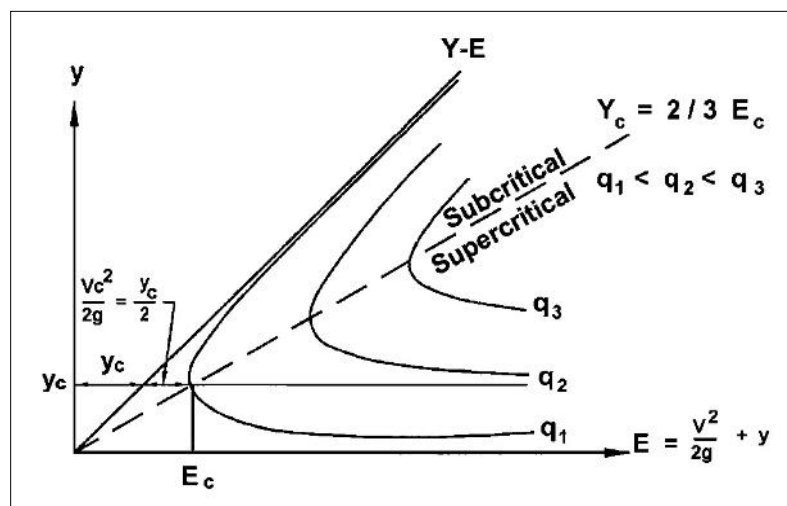
(Henderson, 1966, Eq. 2-5, p. 31)

where:

E	=	the specific energy, ft
y	=	flow depth, ft
g	=	gravitational acceleration constant, 32.2 ft/s <sup>2</sup>
q	=	discharge per unit width of channel (Q/b), ft <sup>3</sup> /s/ft
Q	=	discharge, ft <sup>3</sup> /s
b	=	width of rectangular channel, ft

Assuming a constant flow rate, the energy (E) can be computed for any flow depth (y). Plotting energy (E) against flow depth (y) yields the specific energy diagram, as shown in **Figure 502-1**. At the point on the graph where energy (E) is a minimum, critical depth ( $y_c$ ) occurs. Critical depth ( $y_c$ ) is a line between two different zones of flow. At minimum energy (E), and thus critical depth ( $y_c$ ), the Froude number (Fr) is equal to one. At depths less than critical depth ( $y_c$ ) the flow is supercritical, and at depths greater than critical depth the flow is subcritical.

Referring to **Figure 502-1**, the diagram indicates that for a given specific energy two depths of flow are possible, one in the subcritical range and one in the supercritical range. These two depths are known as alternate depths.



Source: FHWA, August 2013, HEC-22, Figure 5.2, p. 5-3

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

**Figure 502-1 Specific Energy Diagram**

### 502.3 Flow Regimes and Hydraulic Concepts

In open channels and in pipes and culverts flowing partially full, flow can be categorized as either subcritical, critical, or supercritical. Each of these flow regimes is determined by computing the Froude number (Fr) which describes the relationship between inertial forces and gravitational forces and is defined as:

$$Fr = \frac{V}{\sqrt{gy}}$$

502-6

where:

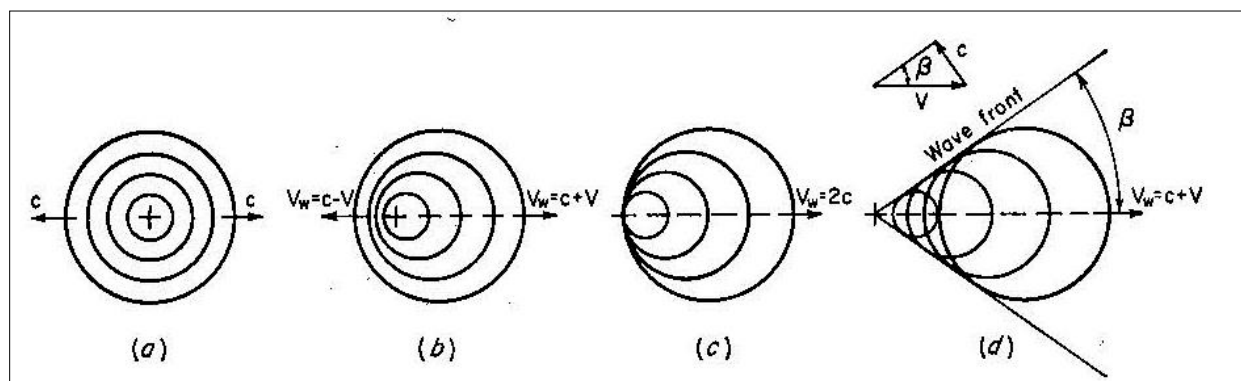
$V$	=	velocity, ft/s
$g$	=	gravitational acceleration, 32.2 ft/s <sup>2</sup>
$y$	=	depth of flow, ft

For critical flow to occur, the inertial force,  $V$ , must be equal to the gravitational force,  $\sqrt{gy}$ ; in this case,  $Fr = 1$ . When  $V < \sqrt{gy}$ ,  $Fr < 1$ , the flow is subcritical; and when  $V > \sqrt{gy}$ ,  $Fr > 1$ , the flow is supercritical.

The gravitational term,  $\sqrt{gy}$ , can also be defined as the celerity (critical velocity) of a small gravity wave which occurs in shallow water due to some disturbance.

Refer to **Figure 502-2** for the following discussion of observed open channel wave generation with velocity in the downstream direction. A gravity wave is generated when a stone is tossed into a still pond, and a series of concentric waves are generated and move outward from the center (**Figure 502-2a**). Gravity waves can be propagated upstream when flow is subcritical ( $Fr < 1$ ) (**Figure 502-2b**), but are swept downstream in supercritical flow ( $Fr > 1$ ) (**Figure 502-2d**). When flow is critical ( $Fr = 1$ ), the waves propagate in the downstream direction but remain stationary at the point of generation and cannot move in the upstream direction (**Figure 502-2c**).

This is a good, although rough, technique to use in the field when an approximation of flow regime is needed. However, it should be noted that natural channels usually vary through all flow regimes and this field test will indicate the flow regime only at the particular location in the channel where the test is performed.



Source: Chow, 1959, Figure 18-8, p. 539

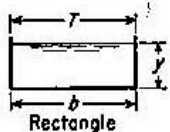
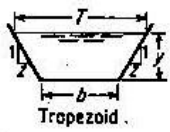
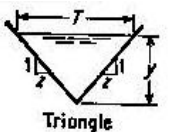
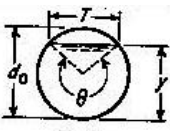
**Figure 502-2 Wave Propagation**

### Normal Depth

Uniform flow occurs in a prismatic channel with a constant flow rate, and thus velocity ( $V$ ) and depth ( $y$ ) are also constant. For uniform flow conditions, Manning's Equation (**Equation 502-1**) can be solved by trial and error for flow depth ( $y$ ), given channel slope ( $S$ ), geometry, " $n$ ", and velocity ( $V$ ) of flow, or flow rate ( $Q$ ). The depth ( $y$ ) obtained is called the normal depth ( $y_o$ ). For example, given a rectangular channel at a given slope and known flow rate, Manning's Equation is solved by selecting a value for the normal depth ( $y$ ) and computing  $Q$  at that depth. If the computed  $Q$  does not equal the given  $Q$ , then select another value for depth and solve for  $Q$ . This process is repeated until the computed  $Q$  at the selected depth equals the given  $Q$ . **Table 502-6** shows geometric elements of a few common man-made channel shapes.

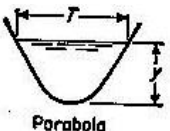
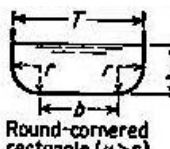
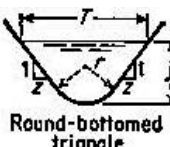
**Table 502-6 Geometric Elements of Channel Sections**

Source: Chow, 1959, Table 2-1, p. 21.

Section	Area	Wetted Perimeter, P	Hydraulic Radius, R	Top Width, T	Hydraulic Depth, D	Section factor, Z
 Rectangle	$by$	$b + 2y$	$\frac{by}{b + 2y}$	$b$	$y$	$by^{1.5}$
 Trapezoid	$(b + zy)y$	$b + 2y\sqrt{1 + z^2}$	$\frac{(b + zy)y}{b + 2y\sqrt{1 + z^2}}$	$b + 2y$	$\frac{(b + zy)y}{b + 2zy}$	$\frac{[(b + zy)y]^{1.5}}{\sqrt{b + 2zy}}$
 Triangle	$zy^2$	$2y\sqrt{1 + z^2}$	$\frac{zy}{2\sqrt{1 + z^2}}$	$2zy$	$\frac{1}{2}y$	$\frac{\sqrt{2}}{2}zy^{2.5}$
 Circle	$zy^2$	$\frac{1}{2}\theta d_o$	$\frac{1}{4}\left(1 - \frac{\sin \theta}{\theta}\right)d_o$	$(\sin 1/2\theta)d_o$ or $2\sqrt{y(d_o - y)}$	$\frac{1}{8}\left(\frac{\theta - \sin \theta}{\sin 1/2\theta}\right)d_o$	$\frac{\sqrt{2}}{32}\frac{(\theta - \sin \theta)^{1.5}}{(\sin 1/2\theta)^{0.5}}d_o^{2.5}$



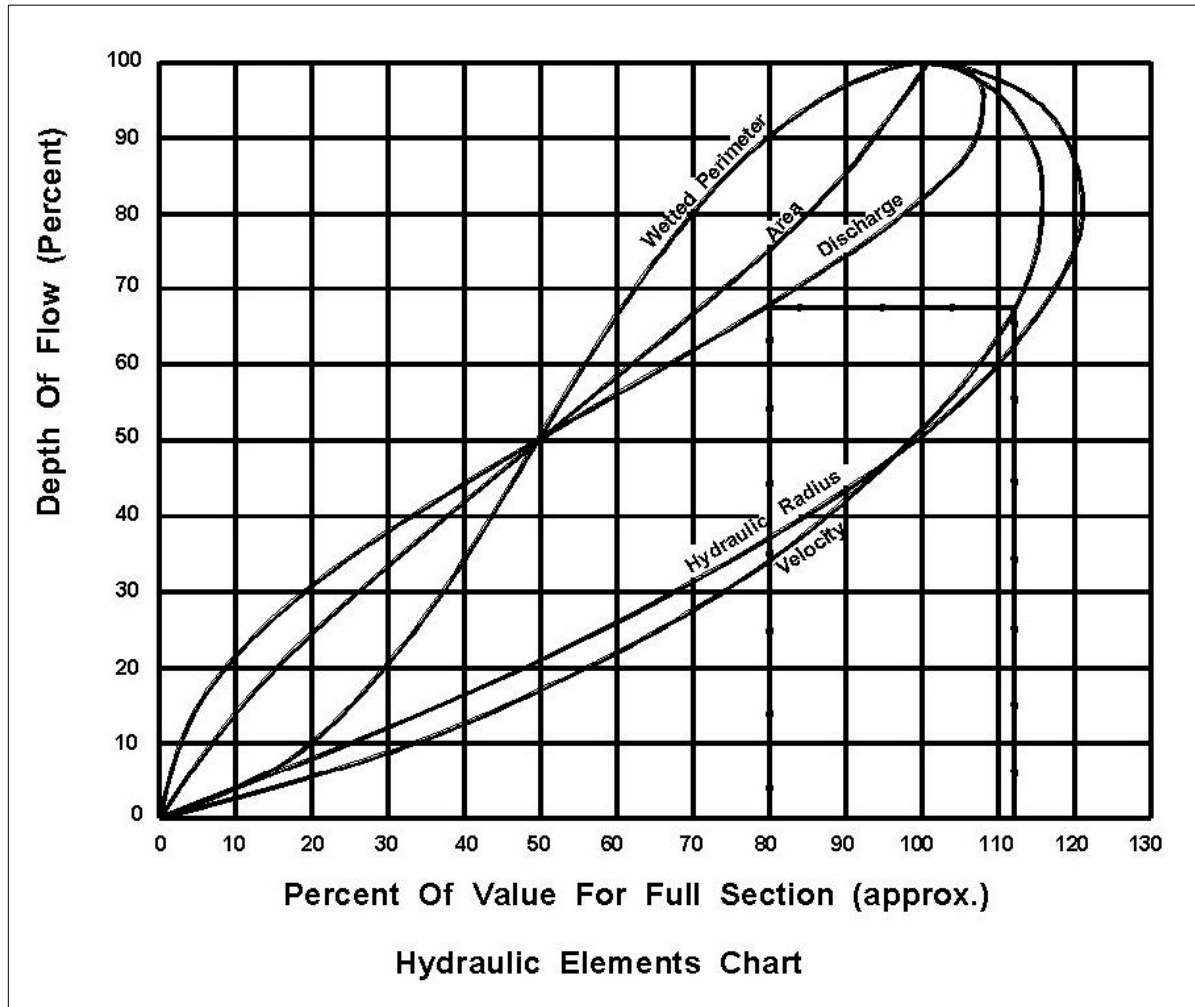
**Table - Geometric Elements of Channel Section (Continued)**

Section	Area	Wetted Perimeter, P	Hydraulic Radius, R	Top Width, T	Hydraulic Depth, D	Section factor, Z
 Parabola	$\frac{2}{3} T y$	$T + \frac{8}{3} \frac{y^2}{T}$	$\frac{2 T^2 y}{3 T^2 + 8 y^2}$	$\frac{3}{2} \frac{A}{y}$	$\frac{2}{3} y$	$\frac{2}{9} \sqrt{6} T y^{1.5}$
 Round-cornered rectangle ( $y > r$ )	$\left(\frac{\pi}{2} - 2\right) r^2 + (b + 2r) y$	$(\pi - 2) r + b + 2y$	$\frac{(\pi / 2 - 2) r^2 + (b + 2r) y}{(\pi - 2) r + b + 2y}$	$b + 2r$	$\frac{(\pi / 2 - 2) r^2}{b + 2r} + y$	$\frac{[(\pi / 2 - 2) r^2 + (b + 2r) y]^{1.5}}{\sqrt{b + 2r}}$
 Round-bottomed triangle	$\frac{T^2}{4z} - \frac{r^2}{z} (1 - z \cot^{-1} z)$	$\frac{T}{z} \sqrt{1 + z^2} - \frac{2r}{z} (1 - z \cot^{-1} z)$	$\frac{A}{P}$	$2 \left[ z(y - r) + r \sqrt{1 + z^2} \right]$	$\frac{A}{T}$	$A \sqrt{\frac{A}{T}}$

There are also several commercially available computer programs which solve Manning's Equation and compute normal depth. Most will also compute critical depth, the Froude number, and a variety of other hydraulic results.

## Circular Pipe

**Figure 502-3** presents the hydraulic elements nomograph for circular pipe. This is another tool to compute hydraulic characteristics for a circular section. Given a known relationship between partial and full sections for either velocity, hydraulic radius, area, wetted perimeter, or discharge, a relationship between partial and full flow depth can be determined. Note that maximum discharge occurs when the pipe is flowing approximately 93% full and maximum velocity occurs when the pipe is flowing approximately 80% full.



Source: FHWA, August 2013, HEC-22, Chart 24, p. A-42.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

**Figure 502-3 Hydraulic Elements Nomograph for Circular Pipe**

## Critical Depth

Critical flow is defined as the state at which the specific energy ( $E$ ) is at a minimum for a given discharge ( $Q$ ). The flow depth ( $y$ ) for this condition is the critical depth ( $y_c$ ). Computation of critical depth is important as a means of identifying the flow regime of a channel. Depths less

than the critical depth ( $y_c$ ) are supercritical and depths greater than critical depth ( $y_c$ ) are subcritical. If the flow rate is known, critical depth in a rectangular channel can be computed by:

$$y_c = \sqrt[3]{\frac{q^2}{g}} \quad 502-7$$

(Vennard and Street, 1982, Eq. 10-15, p. 462)

where:

$y_c$	=	critical depth, ft
$q$	=	flow rate per unit width of channel, ft <sup>3</sup> /s/ft
$g$	=	gravitational acceleration, 32.2 ft/s <sup>2</sup>

It should be noted that critical depth ( $y_c$ ) is dependent on flow rate ( $Q$ ) only. Once critical depth is found, other critical parameters can be computed. Critical velocity ( $V_c$ ) in a rectangular channel is:

$$V_c = \sqrt{g y_c} \quad 502-8$$

(Vennard and Street, 1982, p. 462)

and critical slope ( $S_c$ ) in a rectangular channel is:

$$S_c = \frac{g n^2}{u^2 y_c^{1/3}} \quad 502-9$$

(Vennard and Street, 1982, Eq. 10-17, p. 463)

where:

$S_c$	=	critical slope, ft/ft
$g$	=	gravitational acceleration, 32.2 ft/s <sup>2</sup>
$u$	=	1. 49 for English units
$n$	=	Manning's roughness coefficient
$y_c$	=	critical depth, ft

Channel slopes less than the critical slope are called mild slopes, whereas channel slopes greater than the critical slope are called steep slopes.

The preceding equations were derived for flow in a rectangular channel section. In non-rectangular channels the equations for critical parameters are similar but more complex. The equation for critical depth is:

$$\frac{Q^2}{g} = \frac{A^3}{T} \quad \text{or} \quad \frac{Q^2 T}{g A^3} = 1 \quad 502-10$$

(Brater and King, 1982, Eq. 8-19, p.8-7)

where:

Q	=	flow rate, ft <sup>3</sup> /s
T	=	channel top width, ft
A	=	flow area, ft <sup>2</sup>
g	=	gravitational acceleration, 32.2 ft/s <sup>2</sup>

In general, this equation is solved by trial and error. For a known flow rate (Q), a depth (y) is selected and used to compute the associated top width (T) and then to compute the flow cross sectional area (A). The depth (y), top width (T), and thus area (A), are adjusted until the value equals unity. The depth (y) at which the equation equals one is the critical depth (y<sub>c</sub>).

The critical slope (S<sub>c</sub>) in a non-rectangular channel is found using the equation:

$$S_c = \frac{g n^2}{u^2} \left( \frac{A}{T R^{4/3}} \right) \quad 502-11$$

(Vennard and Street, 1982, Eq. 10.21, p. 467)

where:

S <sub>c</sub>	=	critical slope, ft/ft
g	=	gravitational acceleration, 32.2 ft/s <sup>2</sup>
n	=	Manning's roughness coefficient
u	=	1.49 for English units
A	=	cross sectional area, ft <sup>2</sup>
T	=	channel top width, ft
R	=	hydraulic radius (A / P), ft
P	=	wetted perimeter, ft

### **Energy Grade Line and Hydraulic Grade Line**

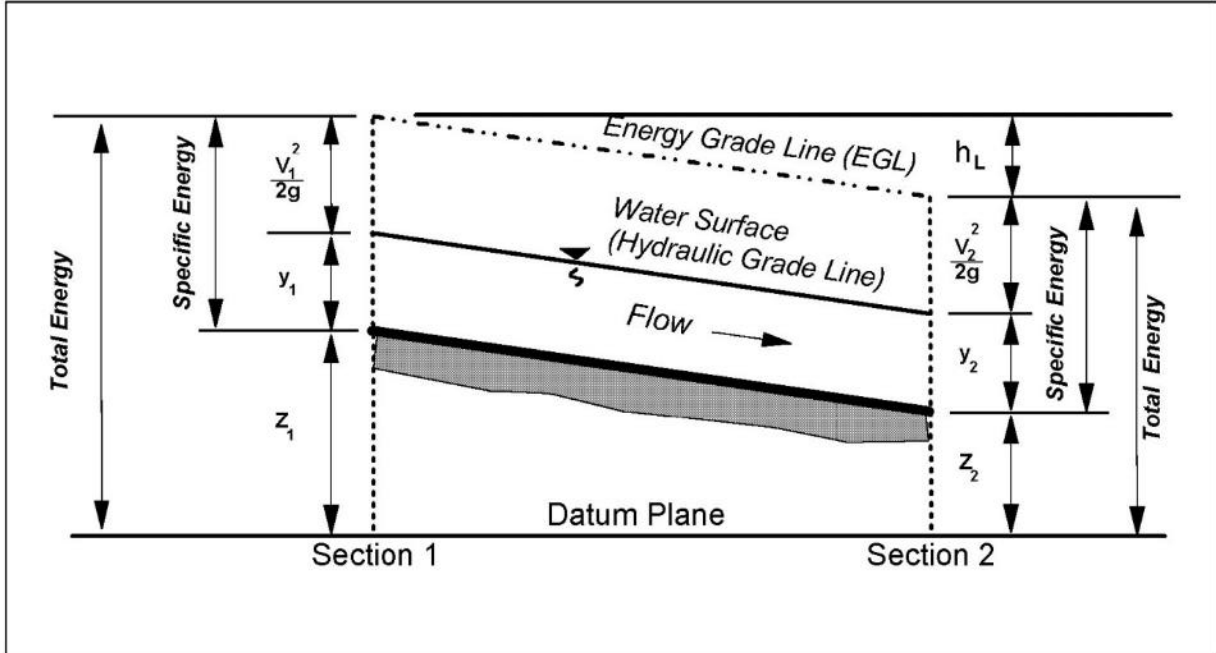
The energy grade line and hydraulic grade line are graphical representations of the energy present in a hydraulic system of pipes and/or open channels. The energy grade line represents the total energy which includes pressure head, kinetic energy, and potential energy. It is the level to which a column of water in a piezometer tube would rise if the tube were inserted in the center of the flow stream. **Figure 502-4** illustrates the energy and hydraulic grade lines.

The hydraulic grade line represents the total energy minus the kinetic energy, or velocity head component. It is the level to which a column of water in a piezometer tube would rise if the tube were set flush with the boundary or wall of the flow stream where the velocity is theoretically zero.

In a closed hydraulic system which flows under pressure, such as a network of manholes/storm sewers flowing full, the level of the energy grade line is important for determining discharge velocity and whether manholes or in-line inlets will surcharge, or overflow. In this type of system, both the energy and hydraulic grade lines normally rise above the top of the pipe at design flow rate.

In open channels and in pipes flowing partially full, the hydraulic grade line is assumed to be coincident with the free water surface, and the energy grade line is above the water surface by a

distance equal to the velocity head, ( $V^2/2g$ ). This relationship is important for determining such hydraulic parameters as tailwater levels, extent of flooding, and flow depth. Except in cases of rapidly varied flow, such as at a hydraulic jump or a drop structure, the slope of the hydraulic grade line, the free water surface, and the channel bed, are assumed to be equal.



Source: FHWA, 2008, HDS-4, Figure 3.2, p. 3-7.

[https://www.fhwa.dot.gov/engineering/hydraulics/pubs/08090/HDS4\\_608.pdf](https://www.fhwa.dot.gov/engineering/hydraulics/pubs/08090/HDS4_608.pdf)

**Figure 502-4 Open Channel Flow Definition Sketch**

### Energy Grade Line

The total energy at any point in a hydraulic system is the sum of the pressure head, the kinetic energy, and the potential energy. Mathematically, the total energy is commonly written in the form of the Bernoulli Equation as:

$$\frac{P}{\gamma} + \frac{V^2}{2g} + Z = H_T$$

**502-12**

where:

P	=	pressure, absolute or gage, lb/ft <sup>2</sup>
γ	=	specific weight of fluid, lb/ft <sup>3</sup>
V	=	velocity, ft/s
g	=	gravitational acceleration, 32.2 ft/s <sup>2</sup>
Z	=	elevation, ft
H <sub>T</sub>	=	Bernoulli constant, ft

The Bernoulli constant is more commonly known to hydraulic engineers as the total head ( $H_T$ ). If there are no friction losses, the total head would be constant throughout the extent of the hydraulic system. Therefore, the total head ( $H_T$ ) components at a specific point can be computed given the total head at any other point in the same system. Assuming no friction losses, this is shown in equation form as:

$$\frac{P_1}{\gamma} + \frac{V_1^2}{2g} + Z_1 = \frac{P_2}{\gamma} + \frac{V_2^2}{2g} + Z_2 \quad 502-13$$

However, in reality, flowing water will experience a loss in energy due to channel boundary roughness, or friction, and other hydraulic influences, when flowing from upstream to downstream. This leads to the following form of the Bernoulli Equation:

$$\frac{P_1}{\gamma} + \frac{V_1^2}{2g} + Z_1 - h_{L_{1-2}} = \frac{P_2}{\gamma} + \frac{V_2^2}{2g} + Z_2 \quad 502-14$$

### Hydraulic Grade Line

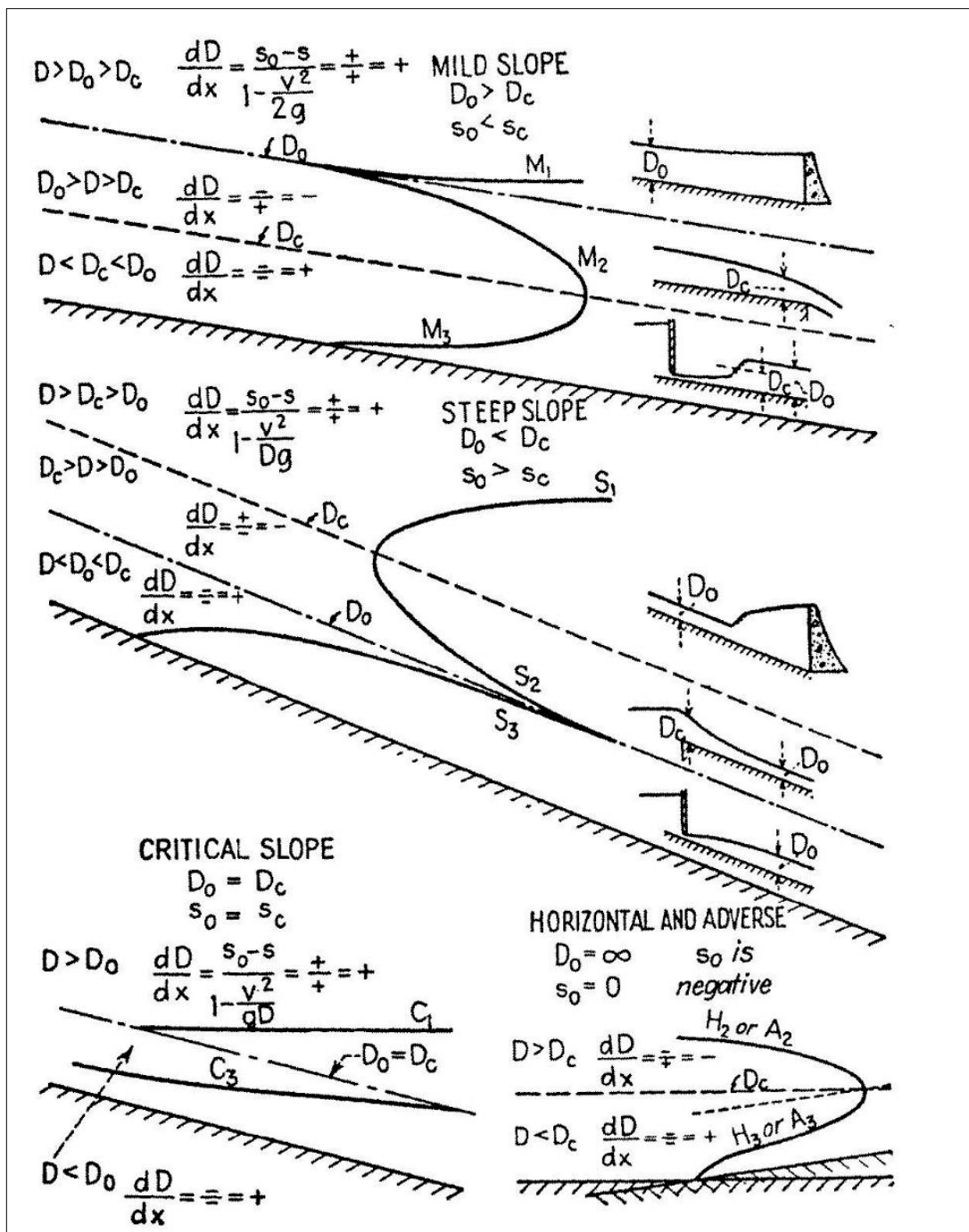
For pipes flowing partially full and in open channel flow, the hydraulic grade line is assumed to be the same as the free water surface. Additionally, flow is assumed to occur due to the forces of gravity rather than under pressure as in a closed pressure conduit. Therefore, in uniform flow, the pressure terms in the Bernoulli Equation are equal at each cross section because they are hydrostatic pressures induced by the depth of the fluid at each cross section as shown in **Figure 502-4**. Under uniform flow conditions, the pressure terms at an upstream and downstream cross section are equal and cancel each other.

In gradually varied flow, the hydraulic grade line is not parallel with the channel slope, and therefore the flow depth is not the same along the length of the channel. The pressures at the upstream and downstream cross sections are not equal, although the difference is relatively small and can usually be neglected for most hydraulic design analyses.

### Longitudinal Profiles

Because uniform flow in a prismatic channel maintains the same depth of flow throughout the length of the channel, computation of uniform flow depth will render the longitudinal water surface profile. This profile will be a line parallel to the channel bottom and a distance above it equal to the normal depth. In non-uniform flow or varied flow, the water surface changes relative to the channel bottom and may be parallel to the bottom for only short sections of channel where uniform flow is established.

These changes in the water surface profile are caused by controls within the channel. Fortunately, the depth of flow at these control points, as well as the slope between control points can be calculated. By computing critical depth and critical slope and comparing them with the normal depth and actual channel slope, an approximation of the water surface profile can be drawn with the help of **Figure 502-5**. The water surface profile is a representation of the profile of flow in a channel under varied or non-uniform flow conditions.



Source: Brater and King, 1982, Figure 8-21, p. 8-38.

Figure 502-5 Water Surface Profiles for Gradually Varied Flow in Open Channels

### 502.4 Non-Uniform Flow in Channels

Although the flow rate remains unchanged in non-uniform flow, some change in the channel geometry, slope, roughness coefficient, or introduced channel control, causes a change in flow depth. The water surface profile will transition from upstream uniform flow through a section of non-uniform flow and, in a long channel, reaches uniform flow depth downstream. Although the analysis of flow through the non-uniform flow section is difficult, the analysis of flow at the two uniform flow sections upstream and downstream of the non-uniform flow section is straightforward. By knowing what channel changes or controls occur, the water surface profile between the two uniform flow sections can be approximated and a composite profile created using **Figure 502-5**, which is important in understanding flow in the channel.

Precise computation of flow depths along the length of a channel can be performed by a variety of methods. One-dimensional and two-dimensional computer programs, such as HEC-RAS, are available for computing water surface profiles. Water surface profiles indicate flooding levels in channels and adjacent areas needed for FEMA mapping, provide information on flow regime (subcritical or supercritical), determine the location of hydraulic jumps, and help evaluate the extent (height) of riprap protection required in areas of high flow velocities.

### 502.5 Channel Expansions, Contractions, and Bends

As in pipe flow, open channel flow experiences energy losses through channel expansions, contractions, and bends. The losses and behavior of flow are significantly different under subcritical and supercritical flow conditions.

If detailed hydraulic analyses are required regarding channel expansions, contractions, wave patterns including in curved channels or oblique hydraulic jumps, refer to the following references for further guidance.

Chow, V.T., 1959, Open-Channel Hydraulics.

Henderson, F.M., 1966, Open Channel Flow.

#### Freeboard

Refer to **Section 204** for rectangular and trapezoidal channel freeboard requirements.

### 502.6 Spillways

Most spillway designs for small detention ponds and retention ponds related to NMDOT projects will normally be designed as simple broad-crested weirs, trapezoidal weirs, sharp-crested-horizontal rectangular weirs, or sharp-crested V-notch weirs. This is because these spillway shapes are simple to analyze, design, and build.

#### Ogee Spillway

If a more complex spillway is required for a larger dam, an “ogee” spillway may be appropriate. Refer to the USBR 1987 document titled “Design of Small Dams” Third Edition, that presents detailed “ogee” spillway hydraulic formulas and design information. The hotlink to that document is provided here.

<http://www.usbr.gov/tsc/techreferences/mands/mands-pdfs/SmallDams.pdf>



## 502.7 Channel Controls and Transitions

Channel controls are features located within the flow path which produce a specific relationship between discharge and flow depth in the immediate vicinity of the control. Examples include weirs, orifices, sluice gates, spillways, and energy dissipators. Channel transitions are changes in either the direction, slope, or geometry of the channel which produce a change in the state of the flow. Examples include channel contractions and expansions, bends, and culvert entrances and exits. This section presents a selection of governing equations for controls and transitions that address most of the design situations engineers are likely to encounter. The listed references will provide a more extensive discussion on each subject. Consult these references when unsure as to when or how to use these equations.

### 502.7.1 Weirs

Brater and King is a well-known hydraulics reference and is cited in this manual. Note – Brater and King, Sixth Edition, 1982 is in English units, the Seventh Edition, 1996, is in metric units.

#### Broad-Crested Weirs

A broad-crested weir is a weir in which the breadth (b) (width) of the crest, parallel to the flow direction, is long enough to force critical depth to occur somewhere along the crest. The head (H) should be measured at a distance approximately 2.5 H upstream from the weir (See **Figure 502-6**).

Flow rate (Q) over a broad-crested weir is given by:

$$Q = C L H^{3/2} \quad 502-15$$

(Brater and King, 1982, Eq. 5-10, p. 5-3)

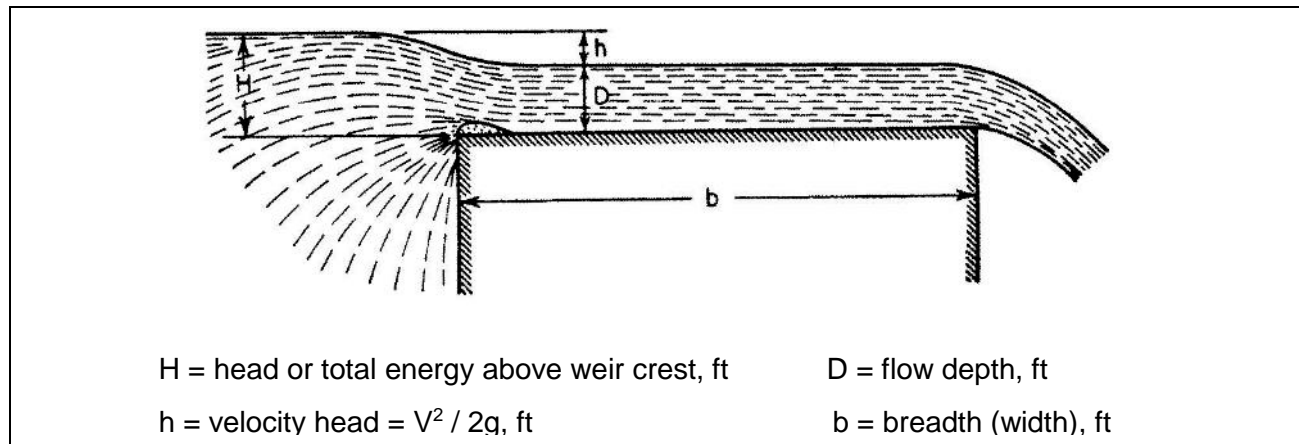
where:

Q	=	discharge, ft <sup>3</sup> /s
C	=	discharge coefficient
L	=	length of weir perpendicular to flow direction, ft
H	=	head or total energy above the weir crest, ft

Brater and King (1982) present several figures and tables for values of “C” under various conditions of irregular weir crest and dam crest shapes. For most design conditions, the value of “C” ranges from approximately 2.3 to 3.3.

**Table 502-7** contains broad-crested weir coefficients. Engineering judgment is required to determine the value of the discharge coefficient for each specific design situation.

There are a number of situations where discharge can be estimated using a broad-crested weir analysis. These include flow over a curb or road perpendicular to the direction of travel, discharge over a spillway from a stormwater storage pond, or flow over or through a grade control structure such as a drop structure.



Source: Brater and King, 1982, Figure 5-6, p. 5-23.

**Figure 502-6 Broad-Crested Weir**

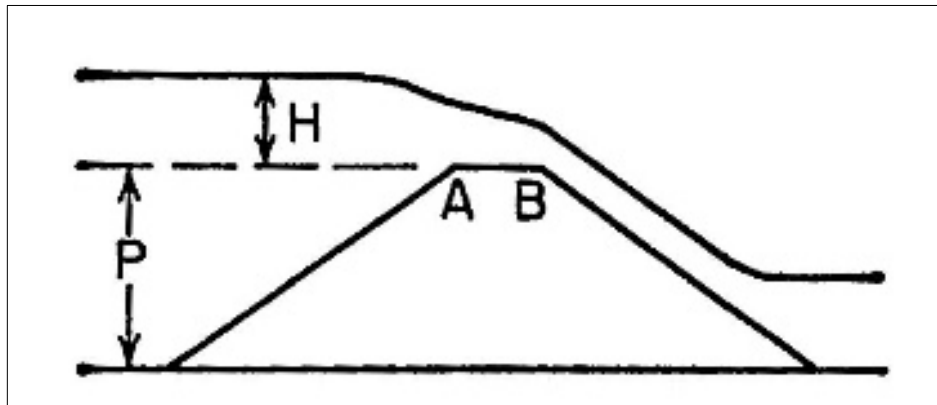
**Table 502-7 Broad-Crested Weir Discharge Coefficients ("C")**

Source: Brater and King, 1982, Table 5-3, p. 5-40.

Measured head in feet, $H$	Breadth of crest of weir, ft										
	0.50	0.75	1.00	1.50	2.00	2.50	3.00	4.00	5.00	10.00	15.00
0.2	2.80	2.75	2.69	2.62	2.54	2.48	2.44	2.38	2.34	2.49	2.68
0.4	2.92	2.80	2.72	2.64	2.61	2.60	2.58	2.54	2.50	2.56	2.70
0.6	3.08	2.89	2.75	2.64	2.61	2.60	2.68	2.69	2.70	2.70	2.70
0.8	3.30	3.04	2.85	2.68	2.60	2.60	2.67	2.68	2.68	2.69	2.64
1.0	3.32	3.14	2.98	2.75	2.66	2.64	2.65	2.67	2.68	2.68	2.63
1.2	3.32	3.20	3.08	2.86	2.70	2.65	2.64	2.67	2.66	2.69	2.64
1.4	3.32	3.26	3.20	2.92	2.77	2.68	2.64	2.65	2.65	2.67	2.64
1.6	3.32	3.29	3.28	3.07	2.89	2.75	2.68	2.66	2.65	2.64	2.63
1.8	3.32	3.32	3.31	3.07	2.88	2.74	2.68	2.66	2.65	2.64	2.63
2.0	3.32	3.31	3.30	3.03	2.85	2.76	2.72	2.68	2.65	2.64	2.63
2.5	3.32	3.32	3.31	3.28	3.07	2.89	2.81	2.72	2.67	2.64	2.63
3.0	3.32	3.32	3.32	3.32	3.20	3.05	2.92	2.73	2.66	2.64	2.63
3.5	3.32	3.32	3.32	3.32	3.32	3.19	2.97	2.76	2.68	2.64	2.63
4.0	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.70	2.64	2.63
4.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.74	2.64	2.63
5.0	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.64	2.63
5.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.64	2.63

**Trapezoidal weirs**

Trapezoidal weirs have upstream and downstream inclined faces as presented in **Figure 502-7**



Source: Brater and King, 1982, Figure 5-9, p. 5-25

**Figure 502-7 Trapezoidal Weir**

H	=	head or total energy above weir crest, ft
P	=	upstream weir crest height, ft
AB	=	breadth (width), ft

Trapezoidal weirs may be applicable in design of emergency spillways on pond embankments as these spillways can be built on the same slopes as the upstream and downstream embankments. The broad-crested weir formula, **Equation 502-15** is applicable, and discharge coefficients for various upstream and downstream inclined slopes for low head and high head applications may be selected from values presented in **Table 502-8** and **Table 502-9**, respectively.

**Table 502-8 Trapezoidal Weir Low Head Discharge Coefficients (“C”)**

Source: Brater and King, 1982, Table 5-9, p. 5-43

Slope of upstream face	Slope of down- stream face	Width of crest in feet	Head in feet, H										
			0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.5
Hor. Vert.	Hor. Vert.												
1 to 2	1 to 1	0.66	2.70	2.82	2.89	3.02	3.13	3.24	3.34	3.44	3.52	3.66	3.82
1 to 2	2 to 1	0.66	2.71	2.79	2.83	2.92	3.03	3.14	3.27	3.32	3.38	3.50	3.61
1 to 2	3 to 1	0.66	2.70	2.76	2.80	2.91	3.00	3.07	3.14	3.21	3.27	3.37	3.45
1 to 2	4 to 1	0.66	2.71	2.74	2.84	2.88	2.98	3.06	3.12	3.17	3.21	3.28	3.35
1 to 2	5 to 1	0.66	2.71	2.80	2.86	2.88	2.93	3.02	3.08	3.12	3.17	3.23	3.26
1 to 2	2 to 1	1.32	....	2.71	2.77	2.80	2.80	2.84	2.88	2.93	2.98	3.08	3.22
1 to 2	4 to 1	1.32	....	2.76	2.80	2.82	2.82	2.85	2.88	2.91	2.94	3.01	3.10
1 to 2	6 to 1	1.32	....	....	2.79	2.80	2.82	2.85	2.87	2.90	2.93	2.98	3.08
2 to 1	2 to 1	0.67	2.82	2.94	3.04	3.13	3.20	3.26	3.32	3.38	3.43	3.51	3.61
1 to 1	2 to 1	0.67	2.73	2.86	2.92	3.02	3.12	3.21	3.29	3.36	3.42	3.53	3.65
1 to 3	2 to 1	0.67	2.50	2.62	2.75	2.87	2.99	3.09	3.18	3.27	3.34	3.46	3.55
Vertical	2 to 1	0.67	2.55	2.58	2.66	2.77	2.90	2.99	3.09	3.18	3.26	3.39	3.51

**Table 502-9 Trapezoidal Weir High Head Discharge Coefficients (“C”)**

Source: Brater and King, 1982, Table 5-10, p. 5-43

Slope of upstream face	Slope of down- stream face	Width of crest in feet	Head in feet, H									
			1.6	1.8	2.0	2.5	3.0	3.5	4.0	4.5	5.0	5.5
Hor. Vert.	Hor. Vert.											
2 to 1	2 to 1	0.67	3.57	3.56	3.56	3.57	3.58	3.60	3.62	3.65	3.68	3.70
2 to 1	5 to 1	0.33	3.58	3.56	3.53	3.48	3.44	3.43	3.48	3.54	3.57	3.58

## Sharp-Crested Weirs

Sharp-crested weirs of various configurations are commonly used as flow measuring devices. However, their use may be applicable in some highway design applications. Therefore, discharge equations for two of the more common types of sharp-crested weirs are presented.

### Sharp-Crested-Horizontal Rectangular Weir

**Figure 502-8** illustrates a profile through the centerline of a sharp-crested-horizontal rectangular weir. The discharge ( $Q$ ) is given by one of many equations developed by various investigators. Again, engineering judgment is required regarding the application of a specific equation to a specific design situation. The following general weir discharge equation applies.

$$Q = C_e L H_e^{3/2} \quad 502-16$$

(Brater and King, 1982, Eq. 5-31, p. 5-9)

where:

$Q$	=	discharge, ft <sup>3</sup> /s
$C_e$	=	discharge coefficient
$L$	=	length of weir perpendicular to flow direction, ft
$H_e$	=	head ( $H$ ) or total energy above the weir crest, ft
		$H_e = H + 0.004$

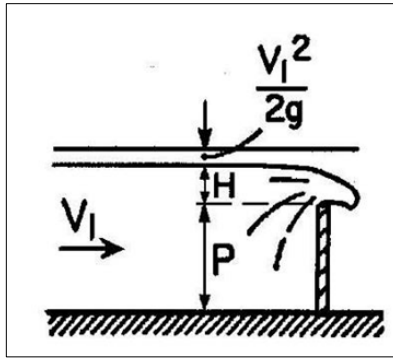
For a sharp-crested-horizontal rectangular weir, Rehbock (1929) developed an empirical formula for the discharge coefficient ( $C_e$ ) given by:

$$C_e = 3.22 + 0.44 \frac{H}{P} \quad 502-17$$

(Brater and King, 1982, Eq. 5-33, p. 5-9)

where:

$C_e$	=	discharge coefficient
$H$	=	head or total energy above the weir crest, ft
$P$	=	weir height, ft (see <b>Figure 502-8</b> )

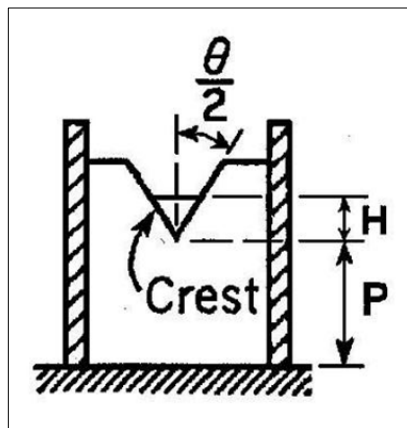


Source: Brater and King, 1982, Figure 5-1, p. 5-2.

**Figure 502-8 Sharp-Crested-Horizontal Rectangular Weir Profile**

Sharp-Crested V-Notch Weir

**Figure 502-9** illustrates a sharp-crested V-notch weir.



Source: Brater and King, 1982, Figure 5-1, p. 5-2.

**Figure 502-9 Sharp-Crested V-Notch Weir Cross Section**

The general discharge ( $Q$ ) equation for a sharp-crested V-notch weir is:

$$Q = C H^{2.5}$$

**502-18**

(Brater and King, 1982, Eq. 5-15, p. 5-4)

where:

$Q$	=	discharge, $\text{ft}^3/\text{s}$
$C$	=	weir coefficient
$H$	=	depth above weir crest, ft

Discharge (Q) equations for various crest angles ( $\theta$ ) are presented here:

For  $\theta = 60^\circ$

$$Q = 1.47 H^{2.51} \quad 502-19$$

(Brater and King, 1982, Eq. 5-47a, p. 5-16)

For  $\theta = 90^\circ$

$$Q = 2.52 H^{2.47} \quad 502-20$$

(Brater and King, 1982, Eq. 5-43, p. 5-16)

For  $\theta = 120^\circ$

$$Q = 4.33 H^{2.5} \quad 502-21$$

(Brater and King, 1982, Eq. 5-48b, p. 5-17)

### 502.7.2 Orifices

An orifice is a device normally used to measure flow or to restrict flow. However, some conduits or openings may be designed as an orifice or will implicitly function as an orifice.

#### Free Discharge

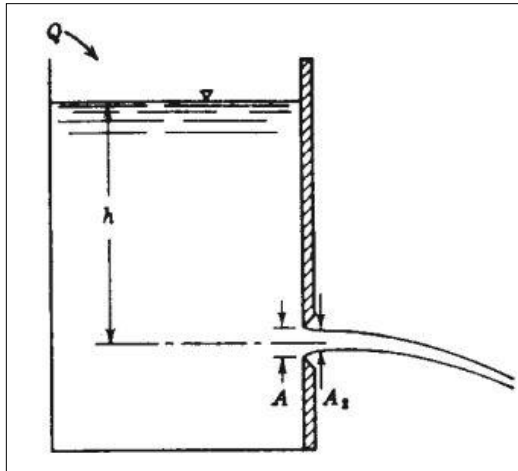
For flow through an orifice discharging to the atmosphere refer to **Figure 502-10**, the discharge equation is:

$$Q = C A \sqrt{2 g h} \quad 502-22$$

(Brater and King, 1982, Eq. 4-10, p. 4-3)

where:

Q	=	discharge, ft <sup>3</sup> /s
C	=	coefficient (see <b>Table 502-10</b> )
A	=	area of the orifice opening, ft <sup>2</sup>
g	=	gravitational acceleration, 32.2 ft/s <sup>2</sup>
h	=	height of water above the orifice centerline, ft (see <b>Figure 502-10</b> )



Source: Vennard and Street, 1982, p. 535.

**Figure 502-10 Orifice Discharging Freely**

**Table 502-10** presents a useful table of orifice coefficients for free discharge.

**Table 502-10 Smith's Discharge Coefficients ("C") for Circular and Square Orifices with Full Contraction**

Source: Brater and King, 1982, Table 4-3, p. 4-29.

Diameter of circular orifice, ft			Head, ft	Side of square orifice, ft		
0.2	0.6	1.0		0.2	0.6	1.0
0.601	0.593	---	0.6	0.605	0.598	---
0.601	0.594	0.590	0.8	0.605	0.600	0.597
0.600	0.595	0.591	1	0.605	0.601	0.599
0.600	0.596	0.593	1.5	0.605	0.602	0.601
0.599	0.597	0.595	2	0.605	0.604	0.602
0.599	0.598	0.596	2.5	0.605	0.604	0.602
0.599	0.598	0.597	3	0.605	0.604	0.603
0.599	0.597	0.596	4	0.605	0.603	0.602
0.598	0.597	0.596	6	0.604	0.603	0.602
0.598	0.596	0.596	8	0.604	0.603	0.602
0.597	0.596	0.595	10	0.603	0.602	0.601
0.596	0.596	0.594	20	0.602	0.601	0.600
0.594	0.594	0.593	50	0.600	0.599	0.599
0.592	0.592	0.592	100	0.598	0.598	0.598



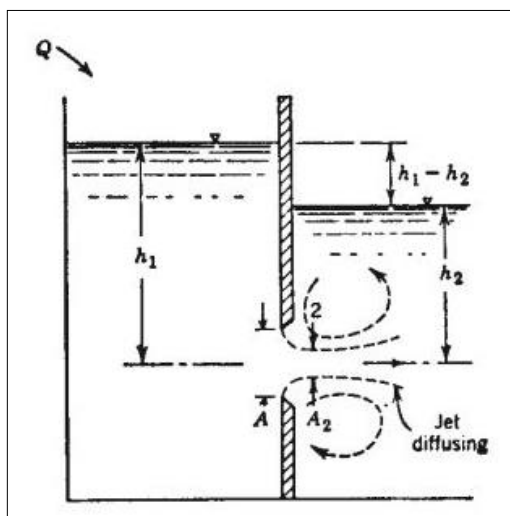
## Submerged Discharge

For flow through an orifice with a submerged discharge (**Figure 502-11**), the discharge equation is:

$$Q = C A \sqrt{2 g (h_1 - h_2)} \quad 502-23$$

(Brater and King, 1982, Eq. 4-21, p. 4-9)

where  $h_1 - h_2$  is defined in **Figure 502-11** and all other terms are as previously defined.



Source: Vennard and Street, 1982, p. 534.

**Figure 502-11 Submerged Orifice**

**Table 502-11** provides discharge coefficients for submerged orifices.

**Table 502-11 Miscellaneous Discharge Coefficients ("C") for Various Sharp-Edged Submerged Orifices**

Source: Brater and King, 1982, Table 4-6, p. 4-32.

Dimensions of orifice, ft d = diameter/depth, ft l = length, ft	Authority	Head, ft							
		0.3	0.5	1.0	2.0	4.0	6.0	10.0	18.0
Square, 0.10 by 0.10	H. Smith	0.607	0.605	0.604	0.603	0.604	---	---	---
Rectangle, l = 3.0, d = 0.05	H. Smith	---	0.621	---	---	0.620	0.620	0.618	---
Circle, d = 1.0	Ellis*	---	---	---	0.608	0.602	0.603	0.600	0.601
Square, 1.0 by 1.0	Ellis*	---	---	---	0.601	0.601	0.603	0.605	0.606
Square, 4.0 by 4.0	Stewart	0.614	---	---	---	---	---	---	---

\*The two orifices experimented on by Ellis were horizontal. All other orifices were vertical.

## 502.8 References

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## 503 Hydraulics of Culverts

### 503.1 Information Required

For existing culverts, the following information will be required before beginning the hydraulic analyses:

- Design Flood and Check Flood hydrographs (particularly for sediment transport analyses)
- Design Flood and Check Flood peak flow rates
- Physical condition, dimensions, invert. and soffit elevations
- Characteristics of the upstream contributing watershed
- Erosion or sedimentation at inlets, outlets, and within the culverts
- Past maintenance requirements
- Design capacity required compared to existing capacity
- Existing utility locations

For proposed culverts, the following information will be required before beginning the hydraulic analyses:

- Design Flood and Check Flood hydrographs (particularly for sediment transport analyses)
- Design Flood and Check Flood peak flow rates
- Characteristics of upstream contributing watershed type and condition of downstream receiving waters
- Characteristic of drainage-way just prior to culvert installation
- Transportability of soils by flowing water
- Durability (consider abrasion and corrosion)
- Existing utility locations

#### 503.1.1 Culvert Flow Control

In the context of this manual, a culvert is defined as a short conduit used to convey stream flow and/or stormwater beneath a roadway. Culverts are often referred to as "hydraulically short", meaning they usually flow partially full, although they can flow full for a short part of the overall culvert length. A "hydraulically long" culvert is one in which full flow occurs over the majority of the overall culvert length.

Culverts come in a variety of shapes and materials. Shapes are usually circular, box, rectangular, elliptical, or arch. While several different types and sizes of culverts may adequately convey the

same design discharge, the engineer should also consider expense, durability, corrosivity potential, abrasion potential, and hydraulic efficiency when designing the culvert.

A culvert's performance is affected by the type of flow control. Flow can be controlled by the inlet conditions, known as inlet control, or by outlet conditions, known as outlet control. In the case of hydraulically long culverts, the culvert cross sectional shape and roughness may control the flow; this is termed barrel control.

### 503.1.2 Culvert Design – General

The information presented in this section is based on the principles in Hydraulic Design of Highway Culverts (FHWA, April 2012, HDS-5). Refer to this document for an in-depth discussion and derivation of the equations presented herein. The design procedures in HDS-5 are based on the principles of the control section and minimum performance.

FHWA, April 2012, "HDS-5, Hydraulic Design of Highway Culverts, Third Edition".

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/12026/hif12026.pdf>

The control section is the location where there is a unique relationship between the flow rate and the upstream water surface elevation. Inlet control is governed by the inlet geometry. Outlet control is governed by a combination of the culvert geometry, the barrel characteristics, and the tailwater depth. Minimum performance is assumed by analyzing both inlet and outlet control and using the highest headwater condition as the flow control. The culvert may operate more efficiently at times (more flow for a given headwater level), but it will not operate at a lower level of performance than calculated.

### 503.1.3 Inlet Control

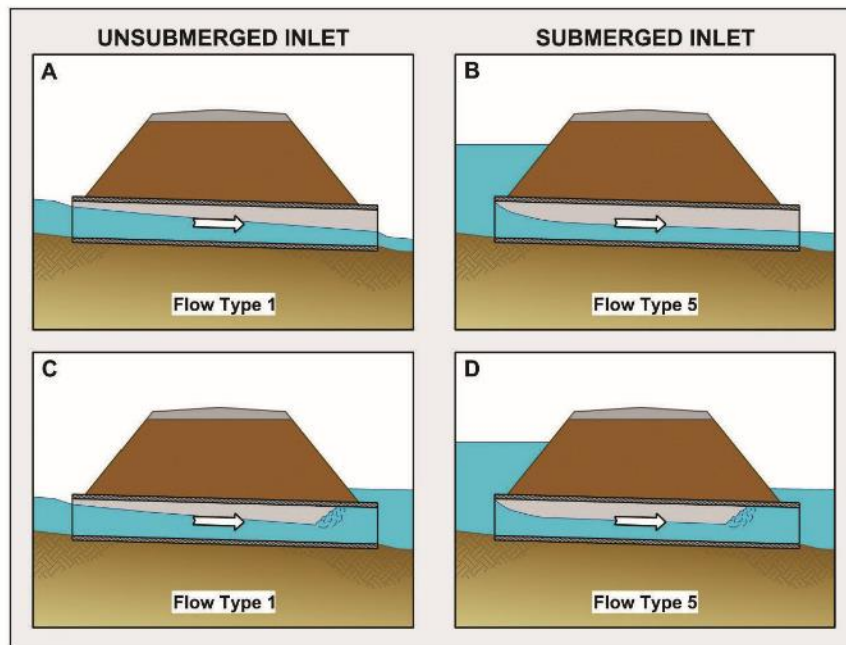
Inlet control occurs when the culvert barrel is able to convey more water than the inlet conditions will allow. In this case the inlet geometry and headwater conditions control the flow at the inlet. Inlet geometry includes the culvert shape, the inlet edge shape, and the culvert size. Critical flow depth occurs at the entrance to the culvert. Flow through the culvert is supercritical.

**Figure 503-1**, adapted from HDS-5 (FHWA, April 2012), shows types of inlet control. The flow type depends on the submergence of the inlet and outlet ends of the culvert. In all of these examples, the control section is at the inlet end of the culvert. Depending on the tailwater, a hydraulic jump may occur downstream of the inlet.

#### Types of Inlet Control (refer to **Figure 503-1**)

##### Flow Type 1

**Figures A and C** illustrate Flow Type 1 where the inlet is not submerged. The flow passes through critical depth just downstream of the culvert entrance, and the flow in the barrel is supercritical. In **Figure A**, the barrel flows partly full over its length, and the flow approaches normal depth at the outlet end. In **Figure C**, submergence of the outlet end of the culvert does not assure outlet control. In this case, the flow just downstream of the inlet is supercritical and a hydraulic jump forms in the culvert barrel.



Source: FHWA, April 2012, HDS-5, Figure 3.1, p. 3.2.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/12026/hif12026.pdf>

**Figure 503-1 Types of Inlet Control**

#### Flow Type 5

**Figures B and D** illustrate Flow Type 5 where the inlet is submerged. In **Figure B**, the inlet end is submerged, and the outlet end flows freely. The flow is supercritical, and the barrel flows partly full over its length. Critical depth is located just downstream of the culvert entrance, and the flow is approaching normal depth at the downstream end of the culvert. **Figure D** is an unusual condition where submergence of both the inlet, and the outlet ends of the culvert does not assure full flow. In this case, a hydraulic jump will form in the barrel. Sub-atmospheric pressures could develop which might create an unstable condition during which the barrel would alternate between full flow and partly full flow.

#### **Factors Influencing Inlet Control**

Since the control is at the upstream end, only the headwater and the inlet factors affect the culvert performance.

Under low headwater conditions, when the entrance is unsubmerged, the culvert entrance will perform as a weir, as shown in **Figure 503-1, Figures A and C** (called Flow Type 1). When high headwater submerges the inlet as shown in **Figure 503-1, Figures B and D** (called Flow Type 5), the entrance performs as an orifice.

For inlet control, the control section is at the upstream end of the barrel (the inlet). The flow passes through critical depth near the inlet and becomes shallow, high velocity (supercritical) flow in the culvert barrel. Depending on the tailwater depth, a hydraulic jump may occur anywhere downstream of the inlet.

Inlet edge configuration describes the entrance type. Some typical inlet edge configurations are thin edge projecting, mitered, square edges in a headwall, and beveled edge.

Inlet shape is usually the same as the shape of the culvert barrel. Typical shapes are rectangular, circular, elliptical, and arch. Check for an additional control section, if the inlet shape is different than the barrel shape. Tapered inlets are discussed later in this section.

Three conditions of flow occur under inlet control: unsubmerged, transition, and submerged. The unsubmerged condition occurs when the headwater is below the crown of the inlet, and the entrance operates as a weir. The submerged condition occurs when the headwater is above the inlet, and the culvert operates as an orifice. The transition zone is located between the unsubmerged and the submerged flow conditions where the flow is poorly defined. This zone is approximated by plotting the unsubmerged and submerged flow equations and connecting them with a line tangent to both curves.

The equations and tables of coefficients required to perform the analysis described above for unsubmerged and submerged inlet control are presented in Appendix 1 of HDS-5 (FHWA, April 2012).

FHWA, April 2012, "HDS-5, Hydraulic Design of Highway Culverts, Third Edition".

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/12026/hif12026.pdf>

#### **503.1.4 Outlet Control**

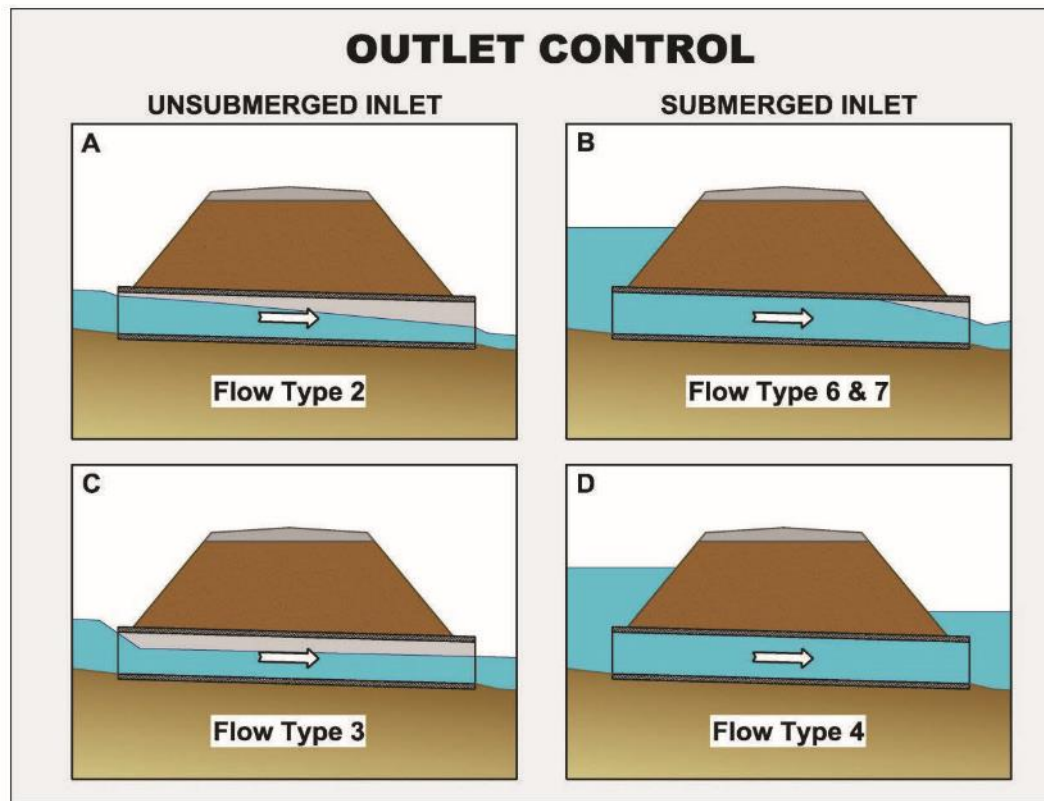
Outlet and barrel control occurs when the culvert barrel cannot convey as much flow as the inlet allows. Control of flow is at the outlet end of the culvert or further downstream in the receiving channel. Flow may be subcritical or pressure flow through the culvert.

Culverts operating under outlet control have depths and velocities that are subcritical. The tailwater depth is either assumed to be critical depth near the culvert outlet or the downstream channel depth, whichever is higher.

Full flow in the culvert barrel is assumed for the analysis of outlet control hydraulics. Outlet control flow conditions can be calculated based on an energy balance from the tailwater pool to the headwater pool.

The performance of a culvert may change from inlet to outlet control at different flow rates. Therefore, a culvert is designed for the worst case (minimum performance) with the assumption that it may perform more efficiently in some situations, but never will it perform less efficiently than at design flow rates.

**Figure 503-2** shows types of outlet control. The flow type depends on the submergence of the inlet and outlet ends of the culvert.



Source: FHWA, April 2012, HDS-5, Figure 3.7, p. 3.8.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/12026/hif12026.pdf>

**Figure 503-2 Types of Outlet Control**

### Types of Outlet Control (refer to **Figure 503-2**)

#### Flow Types 2 and 3

**Figures A and C** illustrate USGS Flow Types 2 and 3 where both the inlet and outlet are unsubmerged. The headwater is shallow so that the inlet crown is exposed as the flow contracts into the culvert. The barrel flows partly full over its entire length and is subcritical.

For Flow Type 2 (**Figure A**), the flow passes through critical depth at the outlet. For Flow Type 3 (**Figure C**), the tailwater is higher than critical depth, and if higher the culvert crown may cause the exit to full flow.

For the partly full flow situations (**Figures A and C**), the flow in the barrel is subcritical.

#### Flow Types 6 and 7

**Figure B** illustrates USGS Flow Types 6 and 7. The culvert entrance is submerged by the headwater and the outlet end is unsubmerged. For Flow Type 6, the barrel is assumed to flow full for most of its length (full flow). For Flow Type 7, the barrel flows partly full over at least part of its length (subcritical flow). For both Flow Types 6 and 7, the flow passes through either critical depth just upstream of the outlet or the tailwater depth, if higher.



### Flow Type 4

**Figure D** illustrates USGS Flow Type 4 which is the classical full barrel flow where both the inlet and outlet are submerged. The barrel is in pressure flow throughout its length. This condition is often assumed in calculations and was used to construct nomographs. Flow Type 4 can also occur when the exit is unsubmerged by tailwater. This is a rare condition. It requires either an extremely high headwater to maintain full barrel flow with no tailwater or critical depth that is higher than the culvert.

### **Factors Influencing Outlet Control**

The geometry of the culvert, as well as the hydraulic characteristics of the culvert control the flow conditions through the culvert for outlet control. These parameters include headwater and tailwater conditions as well as slope, length, shape, area, and roughness of the culvert barrel. Control of flow is at the outlet end of the culvert or further downstream in the receiving channel.

### **Summary of Factors that Influence Culvert Inlet and Outlet Control**

The following table is a brief summary of inlet and outlet control factors that influence culvert capacity hydraulic analyses.

Factor	Inlet Control	Outlet Control
Headwater	X	X
Area	X	X
Shape	X	X
Inlet Configuration	X	X
Barrel Roughness	-	X
Barrel Length	-	X
Barrel Slope	X	X
Tailwater	-	X
Note: For inlet control the area and shape factors relate to the inlet area and shape. For outlet control they relate to the barrel area and shape.		

The following FHWA documents with links may be used as reference for additional culvert analysis and design information.

FHWA, April 2012, "HDS-5, Hydraulic Design of Highway Culverts, Third Edition".

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/12026/hif12026.pdf>

FHWA, November 1972, "HEC-10, Capacity Charts for the Hydraulic Design of Highway Culverts".

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hec/hec10.pdf>

FHWA, August 1972, "HEC-13, "Hydraulic Design of Improved Inlets for Culverts".

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hec/hec13.pdf>



### 503.1.5 Head Loss in Culverts

The total head loss in a culvert is the sum of the entrance and outlet losses, which vary with the design of the end treatments, plus the loss of head due to friction, which varies directly with the roughness, area, shape, length, and slope.

### 503.1.6 Design Nomographs

Nomographs for culvert design under inlet control and outlet control can be found in the latest version of HDS-5 (FHWA, April 2012).

FHWA, April 2012, "HDS-5, Hydraulic Design of Highway Culverts, Third Edition".

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/12026/hif12026.pdf>

The full flow outlet control nomographs were developed assuming that the culvert barrel is flowing full, and the following:

- Tailwater (TW) > culvert diameter (D), or
- Critical depth ( $d_c$ ) > D
- Upstream velocity is small, and its velocity head can be considered to be a part of the available headwater (HW) used to convey the flow through the culvert
- Downstream velocity is small, and its velocity head can be neglected

The partly full flow outlet control nomographs were developed based on numerous backwater calculations performed by the FHWA. They found that the hydraulic grade line pierces the plane of the culvert outlet at a point one-half way between critical depth and the top of the barrel or  $(d_c + D) / 2$  above the outlet invert. TW should be used if higher than  $(4 + D) / 2$ .

### 503.1.7 Culvert Hydraulic Analysis Programs

Some commonly used culvert analysis programs include HY-8 (public domain), CulvertMaster (not public domain), and BCAP (public domain). The FHWA has free culvert hydraulic analysis software known as HY-8 Version 7.50 (FHWA, 2016). This software may provide an easier solution to culvert hydraulic analyses as compared to culvert nomographs. Other software packages may be used upon approval by the NMDOT Drainage Engineer. In addition, the HEC-RAS (public domain) program has convenient culvert analysis available during open channel flow analyses.

Appendix 7 contains **Example Problem 7-1** that presents culvert analyses simulated with the HY-8 program.

FHWA, July 28, 2016, "HY-8 Version 7.50 – Culvert Hydraulic Analysis Program".

<https://www.fhwa.dot.gov/engineering/hydraulics/software/hy8/>

### BCAP – Broken-Back Culvert Analysis Program

The Nebraska Department of Roads, through research with the University of Nebraska, has developed a software program called BCAP (Broken-Back Culvert Analysis Program). This program provides hydraulic analysis for steep culverts having one (Single Broken-back) or two (Double Broken-back) breaks in the vertical profile. BCAP uses the same routines as the FHWA HY-8 culvert program to determine headwater depth at the culvert entrance. BCAP then calculates the water surface profile through the entire culvert, using gradually varied flow

equations and boundary conditions at each vertical break. The program tests for the occurrence of a hydraulic jump in each culvert segment to help determine outlet depth and velocity.

Nebraska Department of Roads and the University of Nebraska, Internet Download, “Broken-Back Culvert Analysis Program (BCAP)”.

<http://www.roads.nebraska.gov/business-center/design-consultant/custom-apps/>

### **HEC-RAS River Analysis System Program for Culvert Analysis**

The HEC-RAS program (USACE, current download) includes the culvert analysis routines from the HY-8 program and the HEC-RAS program is more convenient for culvert analysis when an entire stream profile upstream and downstream of, and including a culvert is required.

USACE, Current Program Download, “HEC-RAS River Analysis System”.

<http://www.hec.usace.army.mil/software/hec-ras/downloads.aspx>

USACE, Current Download, “HEC-RAS River Analysis System – Users Manual”.

<http://www.hec.usace.army.mil/software/hec-ras/documentation.aspx>

USACE, Current Download, “HEC-RAS River Analysis System – Hydraulic Reference Manual”.

<http://www.hec.usace.army.mil/software/hec-ras/documentation.aspx>

### **503.1.8 Outlet Velocity**

Culvert outlet velocities are calculated to determine the need for erosion protection downstream of the culvert. Culvert outlet velocities are usually higher than the natural stream velocities. These outlet velocities may require flow re-adjustment or energy dissipation to prevent downstream erosion. Additional discussion on energy dissipation is covered in **Section 608.7**.

### **Erosion and Sedimentation**

For culverts flowing full, use Manning’s Equation (**Equation 502-1**) to determine flow velocity. For culverts flowing partially full, **Figure 502-3** can be used to find the flow velocity.

Scour analyses at concrete box culvert and other pipe structure outfalls are required on FHWA projects. Refer to Chapter 15 of HEC-14 (FHWA, July 2006) for scour analysis methods.

FHWA, July 2006, “HEC-14, Hydraulic Design of Energy Dissipators for Culverts and Channels, Third Edition”.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/06086/hec14.pdf>

### **503.1.9 Roadway Overtopping**

Roadway overtopping begins when the headwater rises above the highest elevation of the roadway. The overtopping will usually occur at the low point of a sag vertical curve on the roadway. The flow will be similar to flow over a broad-crested weir. Overtopping flow rate can be computed with the following equation:

$$Q_o = C_d L HW_r^{1.5}$$

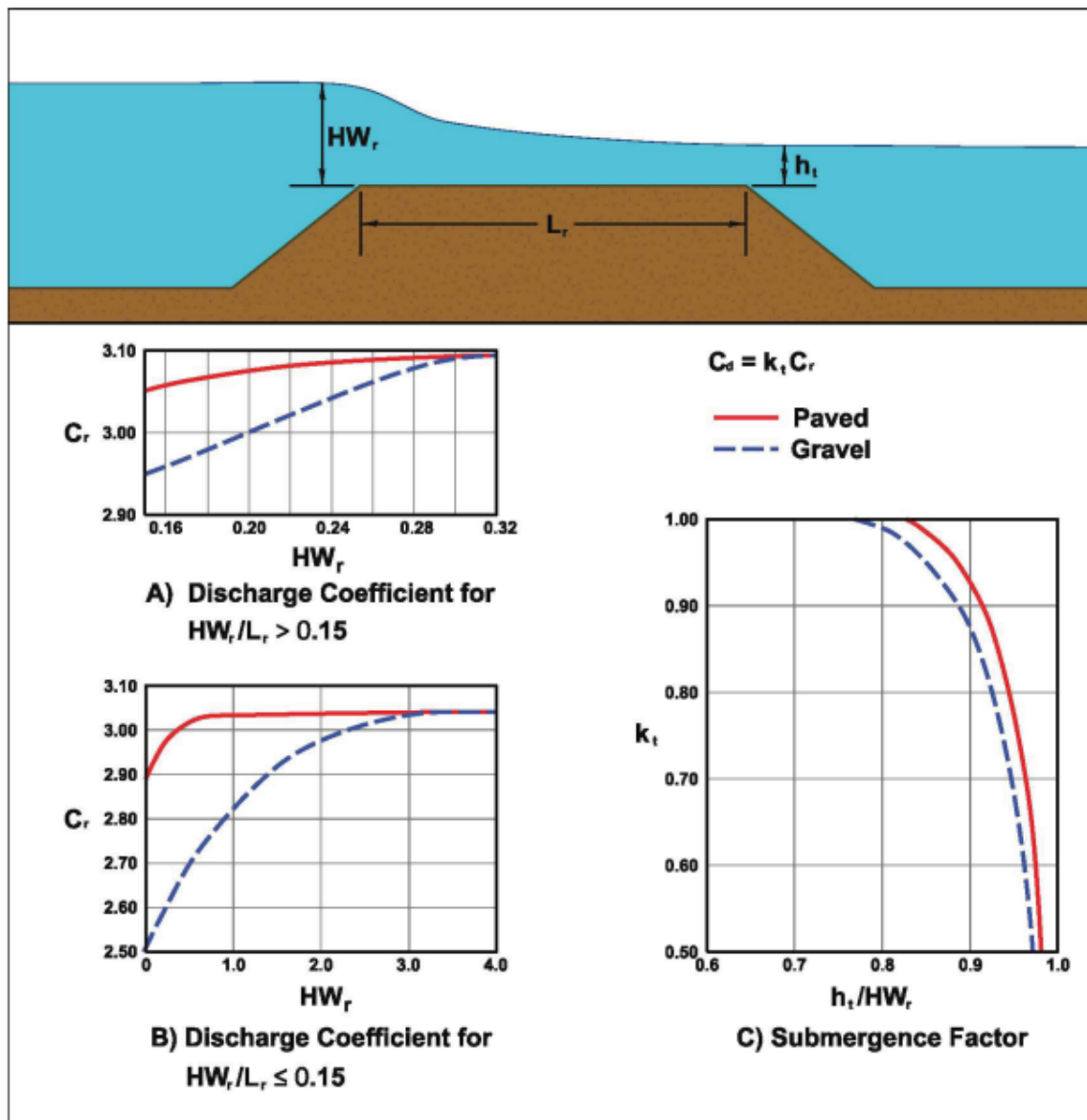
**503-1**

(FHWA, HDS-5, April 2012, Eq. 3.9, p. 3-15)

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/12026/hif12026.pdf>

where:

$Q_o$	=	overtopping flow rate, ft <sup>3</sup> /s
$C_d$	=	overtopping discharge coefficient $C_d = k_t C_r$ (see <b>Figure 503-3</b> )
$k_t$	=	submergence coefficient (see <b>Figure 503-3</b> )
$C_r$	=	discharge coefficient (see <b>Figure 503-3</b> )
$L$	=	length of the roadway crest, ft
$HW_r$	=	the upstream depth, measured above the roadway crest, ft



Source: FHWA, April 2012, HDS-5, Figure 3.11, p. 3.16.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/12026/hif12026.pdf>

**Figure 503-3 Discharge Coefficients for Roadway Overtopping**

### 503.1.10 Performance Curves

A performance curve is a plot of flow rate versus headwater depth, elevation, or velocity. The culvert performance curve is made up of the controlling portions of the individual performance curves for each of the following control sections (see **Figure 503-4**).

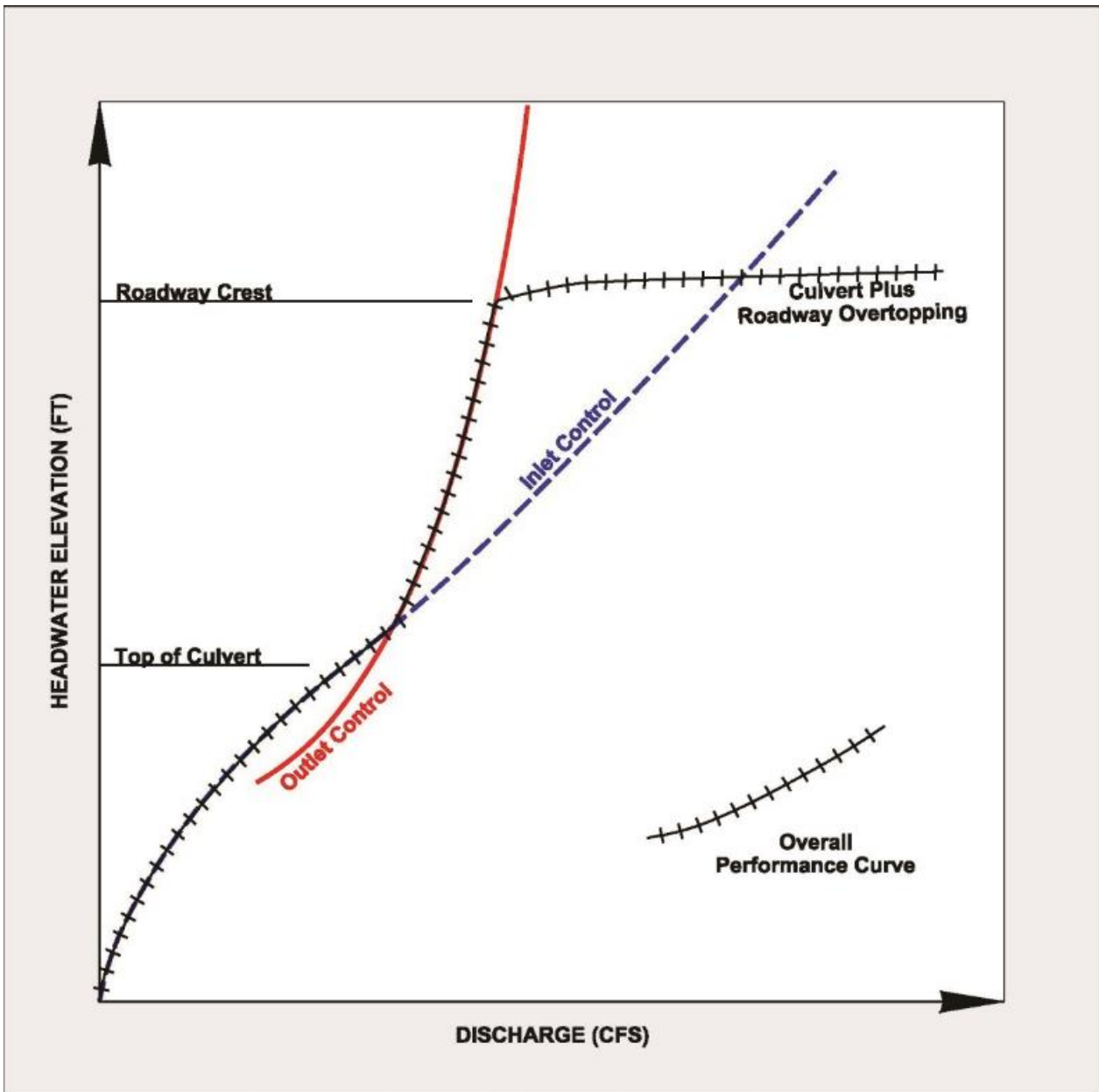
**Inlet** – The inlet performance curve is developed using the inlet control nomographs (refer to HDS-5 for inlet control nomographs).

Outlet – The outlet performance curve is developed using the equations presented in HDS-5, the outlet control nomographs (refer to HDS-5), or backwater calculations.

Roadway – The roadway performance curve is developed using **Equation 503-1**.

The overall performance curve is the sum of the flow through the culvert and the flow across the roadway, can be determined by performing the following steps:

1. Select a range of flow rates and determine the corresponding headwater elevations for the culvert flow alone. These flow rates should fall above and below the design discharge and cover the entire flow range of interest. Both inlet and outlet control headwaters must be calculated.
2. Combine the inlet and outlet control performance curves to define a single performance curve for the culvert.
3. When the culvert headwater elevations exceed the roadway crest elevation, overtopping will begin. Calculate the upstream water surface depth above the roadway for each selected flow rate. Use these water surface depths and **Equation 503-1** to calculate flow rates across the roadway.
4. Add the culvert flow and the roadway overtopping flow at the corresponding headwater elevations to obtain the overall culvert performance curve as shown in **Figure 503-4**.



Source: FHWA, April 2012, HDS-5, Figure 3.16, p. 3.21.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/12026/hif12026.pdf>

**Figure 503-4 Culvert Performance Curve with Roadway Overtopping**

### 503.1.11 HDS-5 Design Procedure

The step-by-step design procedure presented in HDS-5 (FHWA, April 2012) provides a convenient and organized method for designing culverts for a selected discharge, considering inlet and outlet control. The engineer should become familiar with the equations presented in HDS-5 before using the design procedures.

FHWA, April 2012, “HDS-5, Hydraulic Design of Highway Culverts, Third Edition”.  
<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/12026/hif12026.pdf>

### 503.1.12 Tapered Inlets

A tapered inlet is a flared culvert inlet with an enlarged face section and a hydraulically efficient throat section. A tapered inlet may have a depression, or fall, incorporated into the inlet structure or located upstream of the inlet. The depression is used to exert more head on the throat section for a given headwater elevation thereby impairing performance. Therefore, tapered inlets improve culvert performance by providing a more efficient control section (the throat).

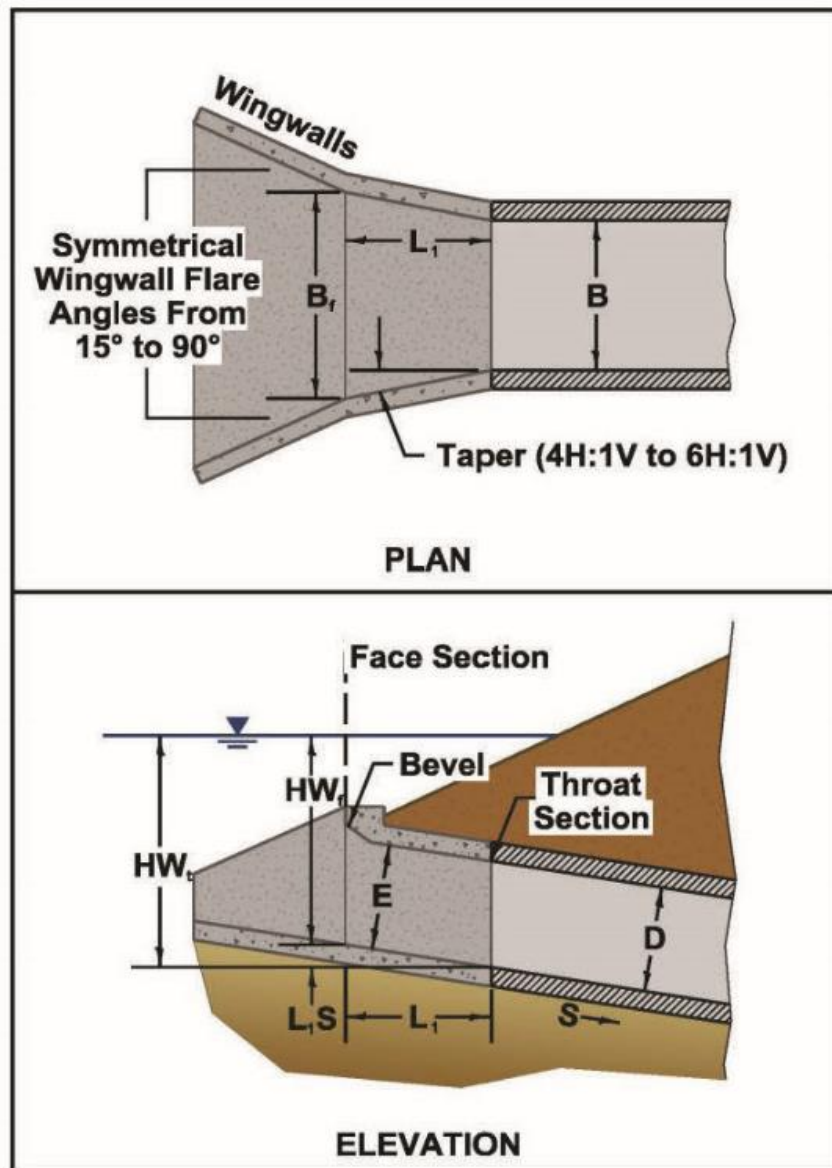
Tapered inlets are not recommended for use on culverts flowing under outlet control since a simple beveled edge provides the same benefit. Design methods have been developed for two basic tapered inlets, the side-tapered inlet and the slope-tapered inlet, with site conditions determining the use of each type. Refer to HEC-13, “Hydraulic Design of Improved Inlets for Culverts”, (FHWA, August 1972) for further discussion, design procedures, and examples for improved culvert inlets.

#### Side-tapered Inlet

The side-tapered inlet has an enlarged face section with the transition to the culvert barrel accomplished by tapering the side walls (**Figure 503-5**). The face section is about the same height as the barrel height and the inlet floor is an extension of the barrel floor. The inlet roof may slope upward slightly, provided that the face height does not exceed the barrel height by more than ten percent. The intersection of the tapered sidewalls and the barrel is defined as the throat section.

There are two possible control sections, the face, and the throat. Headwater depth ( $HW_f$ ), shown in **Figure 503-5**, is the headwater depth measured from the face section invert, and  $HW_t$  is the headwater depth measured from the throat section invert. The throat of a side-tapered inlet is an efficient control section. The flow contraction is nearly eliminated at the throat. In addition, the throat is always slightly lower than the face so that more head is exerted on the throat for a given headwater elevation.

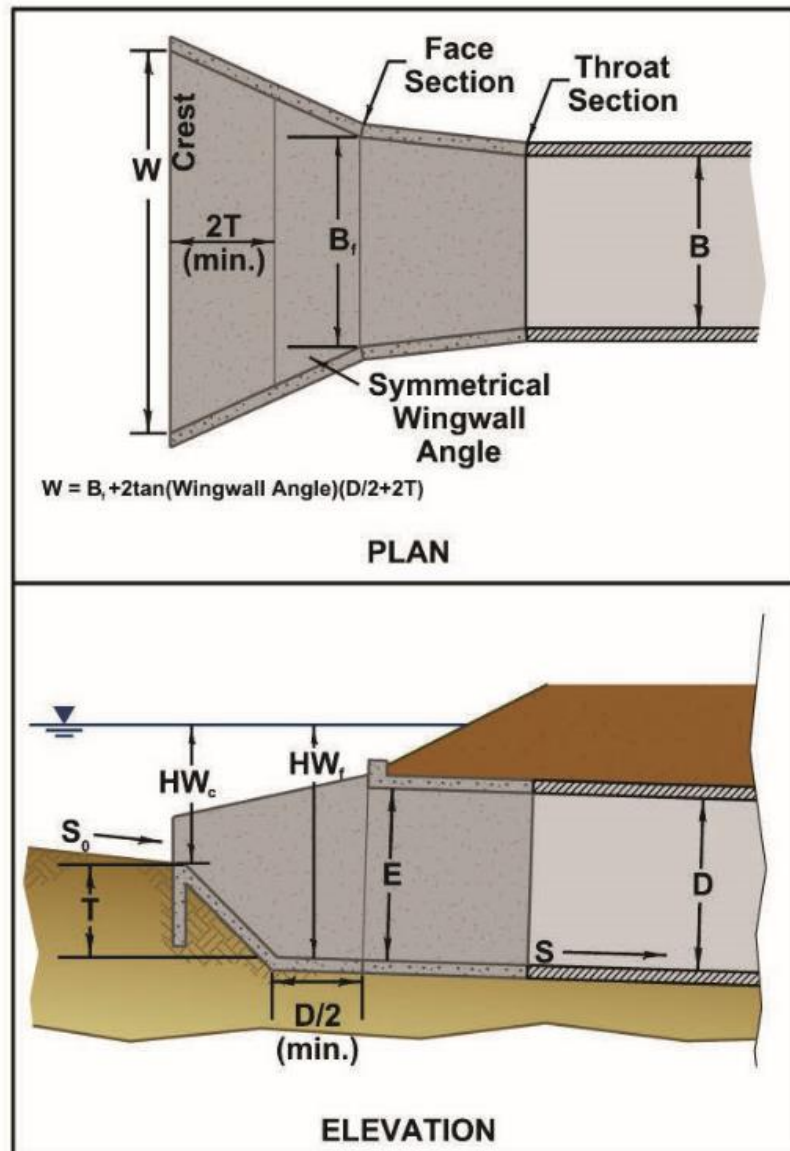
The beneficial effect of depressing the throat section below the streambed can be increased by installing a depression upstream of the side-tapered inlet. **Figure 503-6** depicts a side-tapered inlet with the depression contained between wingwalls. For this type of depression, the floor of the barrel should extend upstream from the face a minimum distance of  $D/2$  before sloping upward more steeply. The length of the resultant upstream crest, where the slope of the depression meets the streambed, should be checked to assure that the crest will not control the flow at the design flow and headwater. If the crest length is too short, the crest may act as a weir control section.



Source: FHWA, April 2012, HDS-5, Figure 3.19, p. 3.29.  
<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/12026/hif12026.pdf>

**Figure 503-5 Side-Tapered Inlet**





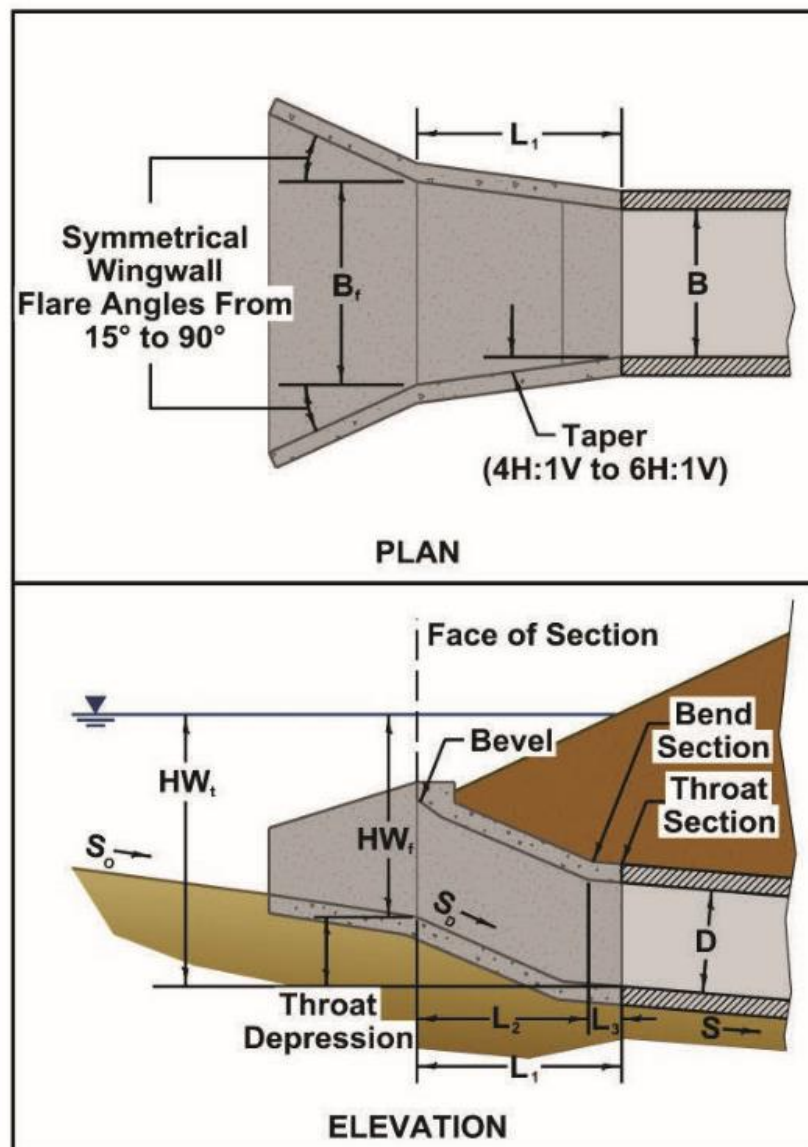
Source: FHWA, April 2012, HDS-5, Figure 3.20, p. 3.29.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/12026/hif12026.pdf>

**Figure 503-6 Side-Tapered Inlet with Inlet Depression**

### Slope-tapered Inlet

The slope-tapered inlet, like the side-tapered inlet, has an enlarged face section with tapered sidewalls meeting the culvert barrel walls at the throat section (**Figure 503-7**). In addition, a vertical fall is incorporated into the inlet between the face and throat sections. This fall concentrates more head on the throat section. At the location where the steeper slope of the inlet intersects the flatter slope of the barrel, a third section, designated as the bend section, is formed.



Source: FHWA, April 2012, HDS-5, Figure 3.21, p. 3.30.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/12026/hif12026.pdf>

**Figure 503-7 Slope-Tapered Inlet with Vertical Walls**

A slope-tapered inlet has three possible control sections, the face, the bend, and the throat. The dimensions of the face and the throat section are determined by the design procedures of this manual. The size of the bend section is established by locating it a minimum distance upstream from the throat so that it will not control the flow.

The slope-tapered inlet combines an efficient throat section with additional head on the throat. The face section does not benefit from the fall between the face and throat; therefore, the face sections of these inlets are larger than the face sections of equivalent depressed side-tapered inlets. The required face size can be reduced by the use of bevels or other similar edge configurations. The vertical face slope-tapered inlet design is shown in **Figure 503-7**.

The slope-tapered inlet is the most complex inlet improvement recommended in this manual. Construction difficulties are inherent, but the benefits in increased performance can be great. With proper design, a slope-tapered inlet passes more flow at a given headwater elevation than any other configuration. Slope-tapered inlets can be applied to both box culverts and circular pipe culverts. For the circular pipe culverts, a square to round transition is normally used to connect the rectangular slope-tapered inlet to the circular pipe.

### Hydraulic Design (Inlet Control)

Tapered inlets have several possible control sections including the face, the bend (for slope-tapered inlets), and the throat. In addition, a depressed side-tapered inlet has a possible control section at the crest upstream of the depression. Each of these inlet control sections has an individual performance curve. The headwater depth for each control section is referenced to the invert of the section. One method of determining the overall inlet control performance curve is to calculate performance curves for each potential control section, and then select the segment of each curve which defines the minimum overall culvert performance.

#### Side-tapered Inlet

The side-tapered inlet throat should be designed to be the primary control section for the design range of flows and headwaters. Since the throat is only slightly lower than the face, it is likely that the face section will function as a weir or an orifice with downstream submergence within the design range. At lower flow rates and headwaters, the face will usually control the flow.

#### Slope-tapered Inlet

The slope-tapered inlet throat can be the primary control section with the face section submerged or unsubmerged. If the face is submerged, the face acts as an orifice with downstream submergence. If the face is unsubmerged, the face acts as a weir, with the flow plunging into the pool formed between the face and the throat. As previously noted, the bend section will not act as the control section if the dimensional criteria of FHWA, April 2012, HDS-5 are followed. However, the bend will contribute to the inlet losses which are included in the inlet loss coefficient,  $k_e$ . Values of  $k_e$  are given in **Table 503-1**. When a culvert with a tapered inlet performs in outlet control, the tapered inlet entrance loss coefficient ( $k_e$ ) is 0.2 for both side-tapered and slope-tapered inlets. This loss coefficient includes contraction and expansion losses at the face, increased friction losses between the face and the throat, and the minor expansion and contraction losses at the throat.

Tapered inlet design begins with the selection of the culvert barrel size, shape, and material. The design procedure is similar to designing a culvert with other control sections (face and throat). The result will be one or more culvert designs, with and without tapered inlets, all of which meet the site design criteria. The engineer must select the best design for the site under consideration.

In the design of tapered inlets, the goal is to maintain control at the efficient throat section in the design range of headwater and discharge. This is because the throat section has the same geometry as the barrel, and the barrel is the costliest part of the culvert. The inlet face is then sized large enough to pass the design flow without acting as a control section in the design discharge range. Some slight oversizing of the face is beneficial because the cost of constructing the tapered inlet is usually minor compared with the cost of the barrel.

**Table 503-1 Entrance Loss Coefficients ( $K_e$ ), Outlet Control, Full or Partly Full**

Source: FHWA, APRIL 2012, HDS-5, Table C.2, p. C.6.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/12026/hif12026.pdf>

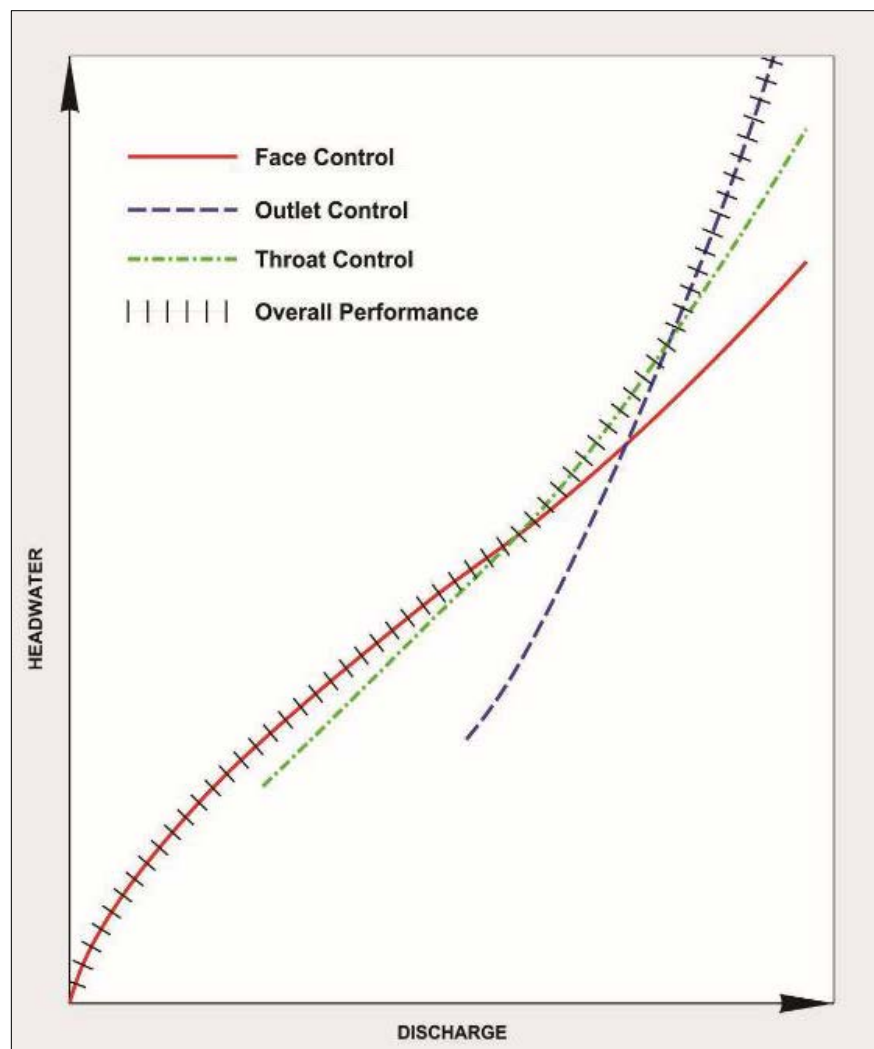
$$H_E = K_e \left[ \frac{V^2}{2g} \right]$$

Type of Structure and Design of Entrance	Coefficient $K_e$
<u>Pipe, Concrete</u>	
Mitered to conform to fill slope	0.7
*End section conforming to fill slope	0.5
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls	
Square-edge	0.5
Socket end of pipe (groove-end)	0.2
Rounded (radius = 1/12D)	0.2
Projecting from fill, socket end (groove end)	0.2
Beveled edges, 33.7° or 45° bevels	0.2
Side-tapered or slope-tapered inlet	0.2
<u>Pipe or Pipe-Arch, Corrugated Metal</u>	
Projecting from fill (no headwall)	0.9
Mitered to conform to fill slope, paved or unpaved slope	0.7
Headwall or headwall and wingwalls, square edge	0.5
*End section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side-tapered or slope-tapered inlet	0.2
<u>Box, Reinforced Concrete</u>	
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side-tapered or slope-tapered inlet	0.2
Wingwalls at 10° to 25° or 30° to 75° to barrel	
Square-edged at crown	0.5
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 of barrel dimension, or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension, or beveled top edge	0.2

\*NOTE: "End Section conforming to fill slope," made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operations to a headwall in both inlet and outlet control. Some end sections, incorporating a taper in their design have a superior hydraulic performance. These latter sections can be designed using the information given for the beveled inlet.

## Performance Curves

Performance curves are important in understanding the operation of a culvert with a tapered inlet. Each potential control section (face, throat, and outlet) has a performance curve, based on the assumption that that particular section controls the flow. Calculating and plotting the various performance curves results in a graph similar to **Figure 503-8**, containing the face control, throat control, and outlet control curves. The overall culvert performance curve is represented by the hatched line. In the range of lower discharges, face control governs; in the intermediate range, throat control governs; and in the higher discharge range, outlet control governs. The crest and bend performance curves are not calculated since they do not govern in the design range.



Source: FHWA, April 2012, HDS-5, Figure 3.23, p. 3.34.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/12026/hif12026.pdf>

**Figure 503-8 Culvert Performance Curve (Schematic)**

### 503.1.13 Inverted Siphons

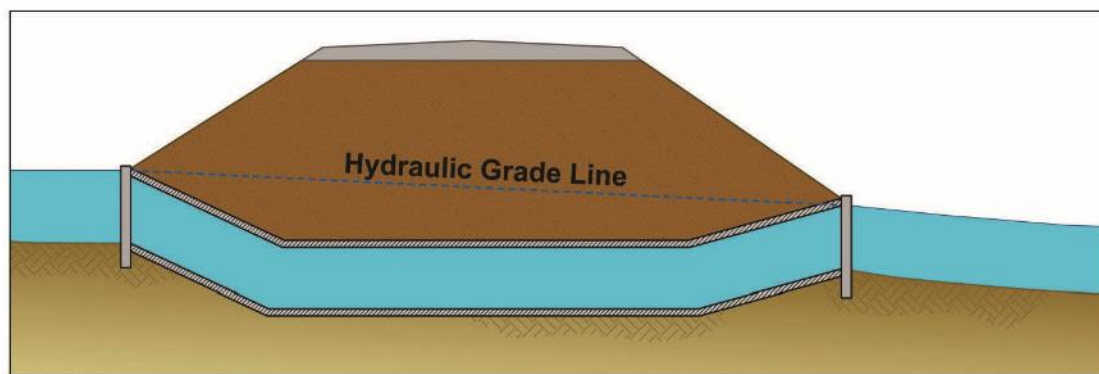
Inverted siphons (sometimes called sag culverts or sag lines) are used to convey water by gravity under roads, railroads, other structures, various types of drainage channels, and depressions. An inverted siphon is a closed conduit designed to run full and under pressure. The structure should operate without excess head when flowing at design capacity. **Figure 503-9** shows a typical inverted siphon. In this section, the term siphon refers to an inverted siphon. Refer to Brater & King (1982) for a discussion on true siphons. True siphons are generally not used in highway applications.

Inverted siphons are easily designed, constructed, and have proven a reliable means of water conveyance. Normally, erosion at the ends of the siphon is inconsequential in earth waterways, provided the transition and any erosion protection is properly designed and constructed. Inverted siphons cannot be used for drainage or irrigation where freezing may block the siphon's waterway.

Costs of design, construction, and maintenance for an inverted siphon are higher than for a culvert that might be used for the same purpose. However, the cost of raising the roadway or structure gradeline may offset this higher cost.

The USBR (1978) publication, "Design of Small Canal Structures," provides an in-depth discussion on inverted siphons with an example beginning on page 29.

<https://www.usbr.gov/tsc/techreferences/mands/mands-pdfs/SmallCanals.pdf>



Source: FHWA, April 2012, HDS-5, Figure 5.2, p. 5.3.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/12026/hif12026.pdf>

**Figure 503-9 Inverted Siphon (Sag Culvert)**

#### Components

**Siphon** - The portion lying between the inlet and outlet. Since siphons are subjected to internal pressure, they should have watertight joints. Jointed pipe requires gaskets to ensure watertightness. Inverted siphons must be able to withstand the internal hydrostatic pressure measured at the centerline of the siphon.

**Transitions** - The inlet and outlet portion of an inverted siphon. Transitions are nearly always used at the inlet and outlet of a siphon to reduce head losses and prevent erosion in unlined channels. Concrete transitions are preferred for this purpose although riprap may be used.

**Collars** - Collars are placed at intervals along the siphon to reduce the velocity of any water moving along the outside of the siphon or through the surrounding earth thereby preventing removal of soil particles (piping) at the point of emergence. Collars are also used to discourage rodents from burrowing along the siphon. Note that the practice of collars is being replaced by the practice of gravel filter drains, particularly for dam design.

**Blowoff Structures** - Provided at or near the low point of inverted siphons to permit draining the siphon for inspection and maintenance or wintertime shutdown. Essentially the blowoff structure consists of a valved drain pipe tapped into the siphon. Although less convenient and requiring higher long-term maintenance costs, short siphons may be dewatered when necessary by pumping from either end of the siphon instead of using a blowoff structure.

**Channel Freeboard** - Upstream channel freeboard is commonly provided to accommodate intercepted storm runoff, improper operation, or blockage by debris.

**Wasteways** - Often placed upstream from a siphon transition for the purpose of diverting the channel flow in case of an emergency. Wasteways may also be integrated into the upstream transition geometry.

**Safety Devices** - Safety measures must be installed near siphons to protect people and animals from injury and loss of life. A hazard can occur both when the siphon is operational as well as dry.

## **Design Procedure**

Available head, economy, and allowable siphon velocities determine the size of the siphon. In order to use Bernoulli's Equation, it is necessary to assume dimensions for the siphon and compute the head losses such as those associated with the entrance, siphon friction, siphon bends, and exit. The sum of all the computed losses should be less than the available head.

## **Design Steps**

- Step 1 Determine inlet and outlet structure types and approximate size of the siphon.
- Step 2 Select preliminary transition geometry.
- Step 3 Make a preliminary layout of the siphon profile to include the siphon, required inlet and outlet structures, existing ground line, roadway geometry, channel properties, and channel stations and elevations at the siphon ends. This layout should indicate the required cover, slope, bend angles, and provide siphon submergence requirements at transitions, or control and siphon inlets.
- Step 4 Compute the siphon head losses using this preliminary layout and trial inlet and outlet geometry. If the computed losses are greater than the difference in upstream and downstream channel water surface elevations, the siphon will probably cause backwater in the channel upstream from the siphon. If backwater exists, the siphon size should be increased or, where feasible, the channel profile revised to provide adequate head.

- Step 5 If the computed losses are appreciably less than the difference in upstream and downstream channel water surface elevations, it may be possible to decrease the size of siphon or, again if feasible; the channel profile may be revised so the available head is approximately the same as the head losses.
- Step 6 On long siphons, there is the possibility of blowback and related unsatisfactory operating conditions. The inlet shall be routinely checked for proper performance for any expected low flows and adjustments made if necessary. On long siphons, vents shall be considered at points where air may be trapped.
- Step 7 Determine the need for siphon components such as erosion protection and siphon collars, as well as determine the required safety needs: wasteways, grates, and fencing. Note that properly designed gravel filter drains are less expensive and may provide better seepage control than collars.
- Step 8 Enter all computed siphon dimensions and angles on the final siphon layout.
- Step 9 Determine the final transition geometry and compute actual head losses. If the actual head losses exceed the available head, return to Step 3.

### **503.2 Irrigation Structures**

All irrigation structures within the State of New Mexico fall under the jurisdiction of public or quasi-public organizations. Modifications to existing structures must conform to the design guidelines and regulations of the jurisdictional body. For those jurisdictions that don't have formal design guidelines, it is advised to seek out guidelines from a larger, adjacent jurisdiction such as the Middle Rio Grande Conservancy District (MRGCD), the Elephant Butte Irrigation District (EBID), or from the USBR.

### **503.3 References**

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## **504 Hydraulics of Pavement Related Drainage Facilities**

### **504.1 Introduction**

Highway and roadway storm drainage facilities should collect stormwater runoff and convey it through the roadway right-of-way in a manner which adequately drains the roadway and minimizes the potential for flooding and erosion to properties adjacent to the right-of-way. Pavement related facilities consist of the following items:

- Curb and gutter
- Concrete wall barrier and concrete barrier railing
- Roadside and median channels
- Rundown channels
- Bridge deck surface drainage
- Drop inlet - a drop inlet consists of a curb opening or not, a grate and a reinforced concrete basin below the grate. Drop inlets are also named “inlets” and catch basins
- Median drop inlets
- Storm drains
- Manholes

The following is a summary of guidelines largely excerpted from HEC-22, “Urban Drainage Design Manual” (FHWA, August 2013). These guidelines should be followed for pavement related storm drainage design and analysis.

FHWA, August 2013, “HEC-22, Urban Drainage Manual, Third Edition”.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

Information Required for Existing Storm Drain Related Items Include:

- Design Flood and Check Flood peak flow rates
- Physical condition, size, design capacity compared to existing capacity of existing inlets and pipes
- System configuration - horizontal and vertical existing utility locations

Information Required for Proposed Storm Drain Related Items Include:

- Existing and proposed conditions Design Flood and Check Flood peak flow rates
- Characteristics of contributing watersheds
- Hydraulic capacity of receiving waters (existing storm drain or open channel)
- Existing utility locations
- Water quality standards
- Durability (consider abrasion and corrosion)

## **504.2 Pavement and Surface Drainage**

Effective drainage of highway pavements is essential to the maintenance of highway service level and to traffic safety. Water on the pavement can interrupt traffic, reduce skid resistance, increase potential for hydroplaning, limit visibility due to splash and spray, and cause difficulty in steering a vehicle when the front wheels encounter puddles.

Pavement drainage design requires computation of the allowable stormwater spread on the pavement surface and allowable hydraulic grade line elevations for storm drain systems. Refer to **Section 200** for drainage criteria. The following subsections present discussion and hydraulic analysis procedures for pavement related drainage facilities.

### **504.2.1 Curb and Gutter**

Curbs are normally located in urban locations and located at the outside edge of pavement for low-speed, highway facilities, and in some instances adjacent to shoulders on moderate to high-speed facilities. Curbs may also be located on the inside of the roadway near the median. For cases of a roadway with superelevation, these interior curbs will capture runoff. Curbs serve the following purposes:

- Contain the surface runoff within the roadway and away from adjacent properties
- Prevent erosion on fill slopes
- Provide pavement delineation
- Enable the orderly development of property adjacent to the roadway
- Provide access control

A gutter is defined as a section of pavement adjacent to the roadway which conveys water during a storm runoff event. It may include a portion of or all of the travel lanes. Gutter sections can be categorized as conventional or shallow swale type as illustrated in **Figure 504-1**. Conventional gutters begin at the inside base of the curb and usually extend from the curb face toward the roadway centerline a distance of 1.0 to 3.0 ft. Shallow swale gutters are often used in paved median areas on roadways with inverted crowns.

As illustrated in **Figure 504-1**, gutters can have uniform, composite, or curved sections. Uniform gutter sections have a cross-slope which is equal to the cross-slope of the shoulder or travel lane adjacent to the gutter. Gutters having composite sections are depressed in relation to the adjacent pavement slope. That is, the paved gutter has a cross-slope which is steeper than that of the adjacent pavement.

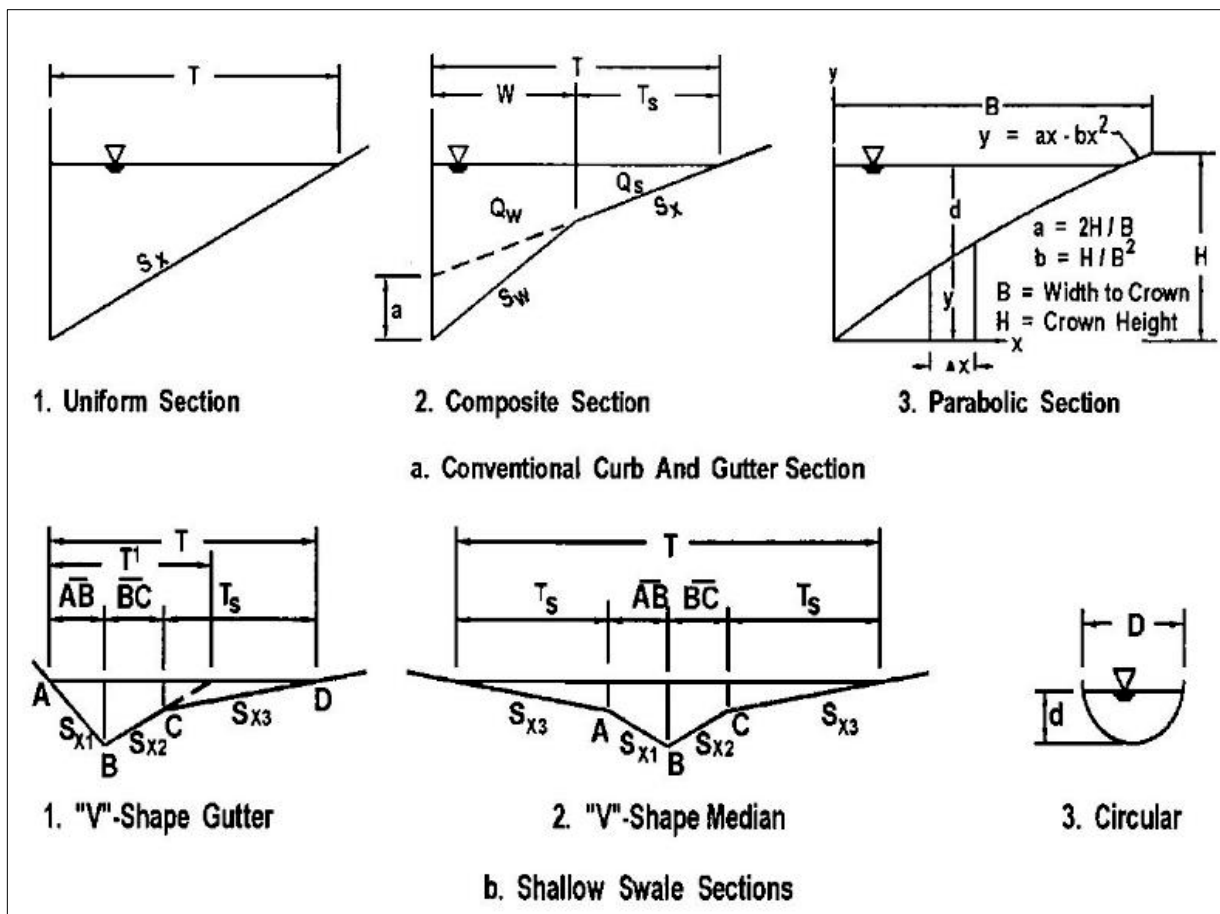
A curb and gutter combination forms a triangular channel that can convey runoff equal to or less than the design flow without interruption of the traffic. When a design flow occurs, there is a spread or widening of the conveyed water surface. The water spreads to include not only the gutter width, but also parking lanes or shoulders, and portions of the traveled surface.

The following pages contain equations that relate to all variables presented in **Figure 504-1**.

Spread (T) is measured perpendicular from the curb face to the extent of the water on the roadway as shown in **Figure 504-1**.

Where practical, runoff from cut slopes and other areas draining toward the roadway should be intercepted before it reaches the highway. By doing so, the deposition of sediment and other debris on the roadway as well as the gutter flow quantity will be minimized. Where curbs are not needed for traffic control, shallow ditch sections at the edge of the roadway pavement or shoulder offer advantages over curbed sections by providing less of a hazard to traffic than a near-vertical curb and by providing hydraulic capacity that is not dependent on spread on the pavement. Ditch sections are particularly appropriate where curbs have historically been used to prevent water from eroding fill slopes when there is sufficient right-of-way.

Storm runoff at rollovers (superelevation transitions) must be evaluated to ensure concentrated flow from a gutter (or other location at the transition) does not cross traffic lanes. Drainage across the lanes must be minimized to reduce traffic hazards due to hydroplaning.



Source: FHWA, August 2013, HEC-22, Figure 4-1, p. 4-7.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

**Figure 504-1 Typical Gutter Sections**

### Gutter Flow Calculations

The following form of Manning's Equation should be used to evaluate gutter flow hydraulics:

$$Q = \left( \frac{K_u}{n} \right) S_x^{1.67} S_L^{0.5} T^{2.67} \quad 504-1$$

or in terms of width of flow (T)

$$T = \left[ (Q n) / (K_u S_x^{1.67} S_L^{0.5}) \right]^{0.375} \quad 504-2$$

(FHWA, August 2013, HEC-22, Eq. 4-2, p. 4-9)

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

where:

Q	=	gutter flow rate, ft <sup>3</sup> /s
T	=	width of flow or spread, ft
n	=	Manning's roughness coefficient, unitless
S <sub>x</sub>	=	pavement cross slope, ft/ft
S <sub>L</sub>	=	longitudinal slope, ft/ft
K <sub>u</sub>	=	0.56 (in English units)

Width of flow (T) or spread on the pavement and flow depth (d) at the curb are often used as criteria for spacing pavement drainage inlets. **Equation 504-3** presents the formula to compute flow depth for a given pavement cross slope (S<sub>x</sub>) and flow width (T).

$$d = T S_x \quad 504-3$$

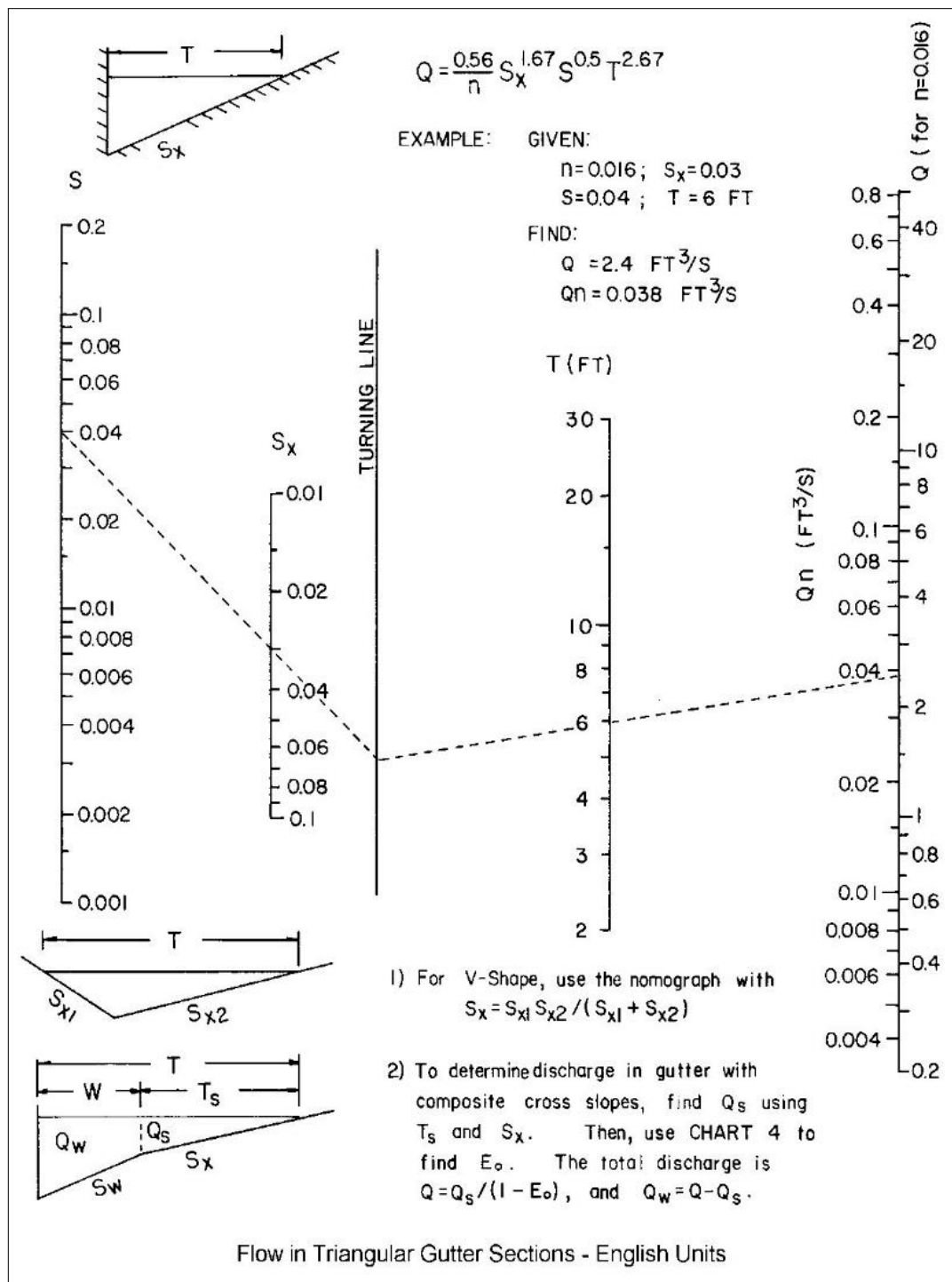
(FHWA, August 2013, HEC-22, Eq. 4-3, p. 4-9)

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

where:

d	=	depth of flow, ft
T	=	width of flow or spread, ft
S <sub>x</sub>	=	pavement cross slope, ft/ft

Common Manning's "n" values for various pavement surfaces are presented in **Table 502-5**. **Figure 504-2** is a nomograph for solving **Equation 504-1**.



Source: FHWA, August 2013, HEC-22, Chart 1B, p. A-3.

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

**Figure 504-2 Flow in Triangular Gutter Sections**

### Conventional Curb and Gutter Sections

Procedures for computing the capacity of a composite curb and gutter follow. For capacity computations of other types of curb and gutter, including shallow swale (v-gutter) sections and circular sections, refer to HEC-22.

To find the gutter flow in a cross section with a composite cross slope, first use **Figure 504-3** or **Equation 504-4** to find the ratio of flow ( $E_o$ ) in a chosen width, that relates to the width ( $W$ ) of the gutter (usually grate width) to total flow width ( $T$ ).

$$E_o = 1 / \left\{ 1 + \frac{S_w / S_x}{\left[ 1 + \frac{S_w / S_x}{\frac{T}{W} - 1} \right]^{2.67} - 1} \right\} \quad 504-4$$

(FHWA, August 2013, HEC-22, Eq. 4-4, p. 4-11)

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

where:

$E_o$	=	ratio of flow in a chosen width (usually grate width) to total gutter flow ( $Q_w/Q$ )
$S_w$	=	gutter cross slope, ft/ft
$S_x$	=	$S_x + a / W$ , ( <b>Figure 504-1 a.2</b> )
$S_x$	=	pavement cross slope, ft/ft
$W$	=	gutter width (usually grate width), ft
$T$	=	width of flow or spread, ft

Then

$$Q_w = Q - Q_s \quad 504-5$$

and

$$Q = \frac{Q_s}{1 - E_o} \quad 504-6$$

(FHWA, August 2013, HEC-22, Eq. 4-4 – 4-6, p. 4-11)

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

where:

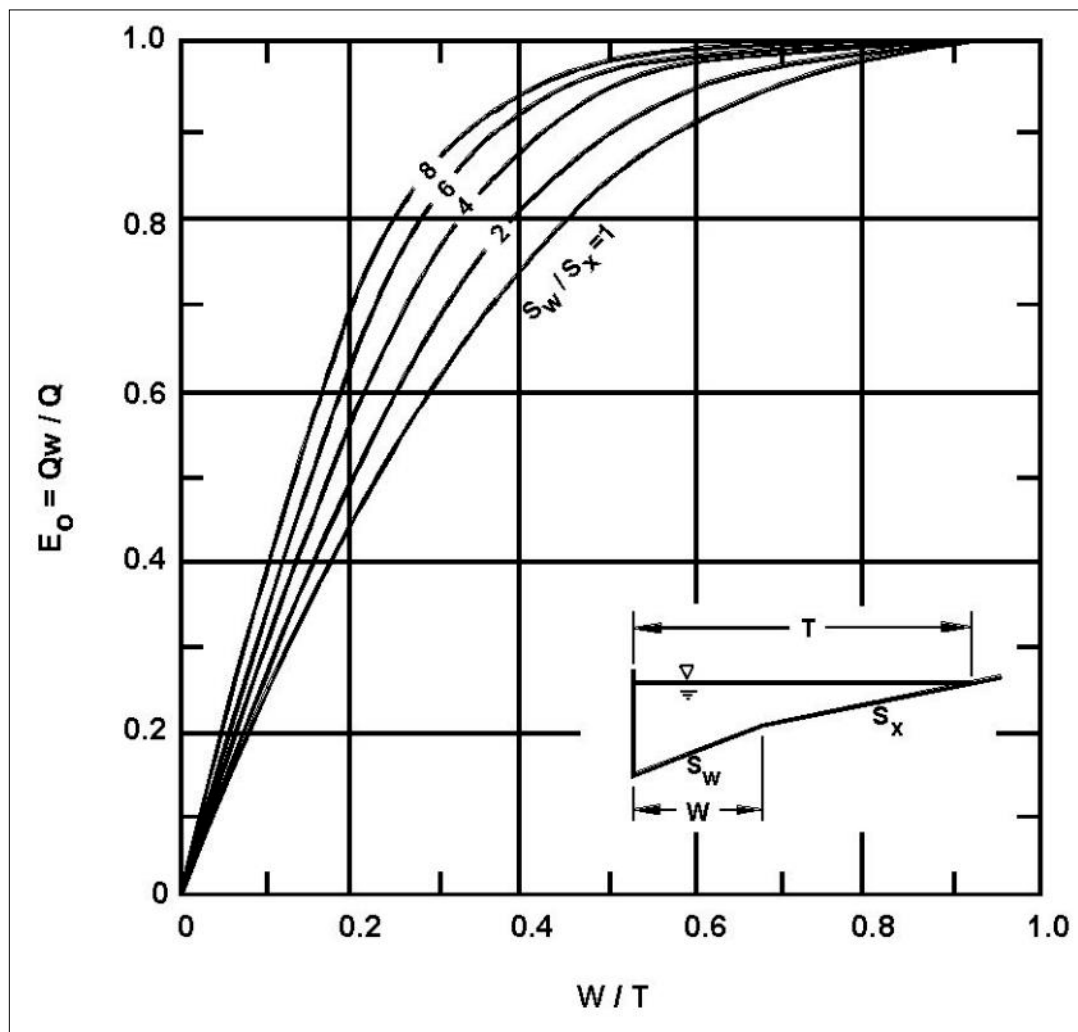
$Q$	=	gutter flow rate, ft <sup>3</sup> /s
$Q_s$	=	flow capacity of the gutter section above the depressed section, ft <sup>3</sup> /s
$Q_w$	=	flow rate in the depressed section of the gutter, ft <sup>3</sup> /s
$E_o$	=	ratio of flow in a chosen width, (usually grate width) to total gutter flow ( $Q_w/Q$ )

Be sure the spread (T) is less than the allowable spread in accordance with the Drainage Design Criteria (**Section 200**) for the specific roadway classification.

Refer to HEC-22 (FHWA, August 2013) for a more in-depth discussion and step-by-step procedures for calculating gutter flow. Refer to HEC-22 (FHWA, August 2013, p. 4-10) for an example problem to solve for capacity of a conventional gutter section.

FHWA, August 2013, "HEC-22, Urban Drainage Manual, Third Edition".

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>



Source: FHWA, August 2013, HEC-22, Chart 2B, p. A-5.

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

**Figure 504-3 Ratio of Frontal Flow to Gutter Flow**

### 504.2.2 Concrete Wall Barriers and Concrete Barrier Railings

Roadway runoff may be directed towards concrete wall barriers and concrete barrier railings. Therefore, flow spread must be evaluated similarly to a curb and gutter section. Depending on the location and contributing drainage area, the concrete wall barriers and concrete barrier railings may need drainage slots. Refer to the NMDOT Standard Drawings for details and also the associated drainage slot sizes (concrete wall barriers).

NMDOT, Website, “Standard Drawings”, “Standard Specifications for Highway and Bridge Construction”.

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

The drainage slot capacity may be computed as a weir for shallower flows, and as an orifice for deeper flows. The Engineer must evaluate both analysis methods for a range of flow depths. For a given flow depth, select the lesser of the two calculated flows as the applicable flow rate. The slots will most likely free discharge. Refer to **Table 205-1** for clogging factors.

### 504.2.3 Roadside and Median Channels

Roadside and median channels are commonly used with uncurbed roadway sections to convey runoff from the highway pavement and from areas which drain toward the highway. Due to right-of-way limitations, roadside channels cannot be used on most urban arterials. They can be used in cut sections, depressed sections, and other locations where sufficient right-of-way is available and driveways or intersections are infrequent. Channel flow depth and corresponding spread must be less than the spread as defined in the Drainage Criteria (**Table 204-1** and **Table 204-2**). Channels may have a vegetative lining based on the soils, vegetation and permissible velocities. Other channel linings shall be used where a vegetation lining will not control erosion. Channel capacity shall be computed with Manning’s Equation and flow area as presented in **Section 502.2**.

### 504.2.4 Bridge Deck Surface Drainage

Bridge deck drainage is similar to that of curbed roadway sections. Effective bridge deck drainage is important for the following reasons:

- Deck structural and reinforcing steel is susceptible to corrosion from deicing salts
- Moisture on bridge decks freezes before surface roadways
- Hydroplaning often occurs at shallower depths on bridges due to the reduced surface texture of concrete bridge decks

Bridge deck drainage is often less efficient than roadway sections because cross slopes are flatter, parapets collect large amounts of debris, drainage inlets or typical bridge scuppers are less hydraulically efficient and more easily clogged by debris, and bridges lack clear zones. Because of the difficulties in providing for and maintaining adequate deck drainage systems, gutter flow from approach roadways should be intercepted before it reaches a bridge. For similar reasons, zero gradients and sag vertical curves should be avoided on bridges. Additionally, runoff from bridge decks should be collected immediately after it flows onto the subsequent roadway section where larger grates and inlet structures can be used. The design of bridge deck drains should prevent the discharge of stormwater onto any portion of the structure or on moving traffic below and prevent erosion at the outlet of the downspout. HEC-21, “Design of Bridge Deck Drainage” (FHWA, May 1993) includes detailed coverage of bridge deck drainage systems.



FHWA, May 1993, “HEC-21, Design of Bridge Deck Drainage”.  
<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hec/hec21.pdf>

Federal, state, or local regulations may prohibit bridge deck drains from discharging directly into the watercourse below.

### 504.3 Inlets

Inlets are drainage structures utilized to collect surface water through grate or curb openings and convey it to storm drains or culverts. Inlet capacity governs both the rate of water removal from the gutter and the amount of water that can enter the storm drainage system.

Inlets are sized and located to limit the spread of stormwater runoff onto traffic lanes to tolerable widths. The inlet analysis must include assumed clogging factors. See **Table 204-1** and **Table 204-2** for spread limits; and **Table 205-1** for clogging factors.

Because grates may become blocked by trash accumulation, curb openings or combination inlets with both grate and curb openings are advantageous for urban conditions. Grate inlets and depressions of curb-opening inlets should be located outside the through-traffic lanes to minimize the shifting of vehicles attempting to avoid them. Inlet grates must safely accommodate bicycle and pedestrian traffic where appropriate.

The preferred configuration for multiple inlets on grade is to utilize clusters of single inlets spaced with a minimum of 20 feet between them. The inclusion of the intervening space allows the bypass flow to fall back down to the curb for increased capture efficiency by the next downstream inlet. Additionally, single inlets, or an inlet cluster, should capture 70% or more of the initial gutter flow. Coordinate with the Drainage Engineer for unusual situations.

Inlets at sag vertical curves in the roadway grade, or within medians, should also be capable of limiting the spread to tolerable limits (**Table 204-1** and **Table 204-2**). At high discharges, this can be accomplished by the use of inlets just upstream of the sag inlet on either or both sides of the sag. These additional inlets, often referred to as flanking inlets, serve to pick up the runoff before it reaches the sag and also limit the spread in the event that the sag inlet is clogged by debris. At least one flanking inlet is required for inlets in a sag and possibly more than one flanking inlet may be required to meet the drainage criteria.

Inlets should be located so that concentrated flow and sheet flow will not cross traffic lanes. Where pavement surfaces are superelevated or warped, as at cross streets or ramps, surface water should be intercepted just before the change in cross slope. Also, inlets should be located just upgrade of pedestrian crossings.

Special care should be given to inlet placement to ensure adequate capacity at bridge approaches and at sag vertical curves where ponding deeper than the curb height could occur.

The NMDOT Standard Drawings refer to all inlet types as “drop inlets” of various types.

#### 504.3.1 Inlet Types

Inlets used for the drainage of highway surfaces can be divided into four major classes. These classes are:

Curb-Opening Inlets are most effective on flatter slopes, in sags, and with flows which typically carry significant amounts of floating debris. The interception capacity of curb-opening inlets

decreases as the gutter grade steepens. Consequently, the use of curb-opening inlets is recommended in sags and on grades less than 3%.

Grate Inlets consist of an opening in the gutter covered by one or more grates. Grate inlets generally lose capacity with increase in grade, but to a lesser degree than curb opening inlets. The principal advantage of grate inlets is that they are installed along the roadway where the water is flowing. Grate inlets principal disadvantage is that they may be clogged by floating trash or debris. Where bicycle traffic occurs, grates should be bicycle safe.

Combination Inlets usually consist of both a curb-opening inlet and a grate inlet placed in a side-by-side configuration. When the curb opening precedes the grate in a "Sweeper" configuration, the curb-opening inlet acts as a trash interceptor during the initial storm runoff or hydrograph rising limb.

Median Drop Inlets consist of a grate inlet set on grade located in medians. These are most often located in a sag vertical curve. These inlets are susceptible to debris and sediment clogging, and consequently the interception capacity will be reduced. Therefore, clogging factors are required to design for adequate capacity.

Slotted Drain Inlets consist of a slotted opening along the curb with bars perpendicular to the opening. Slotted inlets function in essentially the same manner as curb-opening inlets, i.e., as weirs with flow entering from the side. However, slotted inlets are very susceptible to clogging from sediments and debris and are not recommended for use in environments where significant sediment or debris loads may be present.

In addition, where significant ponding can occur in locations such as underpasses and in sag vertical curves in depressed sections, at least one flanking inlet is required at the low point in the sag. The flanking inlets should be placed so that they will limit spread on low gradient approaches to the sag, and act in relief of the inlet at the low point if it should become clogged or if the design spread is exceeded. Refer to the NMDOT standard drawings for NMDOT "drop inlets" and grates.

NMDOT, Website, "Standard Drawings", "Standard Specifications for Highway and Bridge Construction".

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

### 504.3.2 Inlet Capacity

The capacity of an inlet depends on its geometry and characteristics of the gutter flow. Inlet interception capacity has been investigated by various agencies and manufacturers of grates. Where debris is a problem, consideration should be given to debris handling efficiency of grate inlets. Refer to **Table 205-1** for minimum required clogging factors. For inlet efficiency data for various sizes refer to Federal Highway Administration's HEC-22 (FHWA, August 2013) and inlet grate capacity charts prepared by the grate manufacturers.

Inlet interception capacity ( $Q_i$ ) is the flow intercepted by an inlet under a given set of conditions. The efficiency of an inlet ( $E$ ) is the percent of total flow ( $Q$ ) that the inlet will intercept for those conditions. The efficiency of an inlet changes with changes in cross slope, longitudinal slope, total gutter flow, and, to a lesser extent, pavement roughness. In mathematical form, efficiency ( $E$ ) is defined by the following equation:

$$E = Q_i / Q$$

**504-7**

(FHWA, August 2013, HEC-22, Eq. 4-14, p. 4-31)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

where:

E	=	inlet efficiency
Q	=	total gutter flow, ft <sup>3</sup> /s
Q <sub>i</sub>	=	intercepted flow, ft <sup>3</sup> /s

Flow that is not intercepted by an inlet is termed carryover or bypass and is defined as follows:

$$Q_b = Q - Q_i$$

**504-8**

(FHWA, August 2013, HEC-22, Eq. 4-15, p. 4-31)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

where:

Q <sub>b</sub>	=	bypass flow, ft <sup>3</sup> /s
Q	=	total gutter flow, ft <sup>3</sup> /s
Q <sub>i</sub>	=	intercepted flow, ft <sup>3</sup> /s

The interception capacity of all inlet configurations increases with increasing flow rates, and inlet efficiency generally decreases with increasing flow rates. In other words, as the flow rate increases, inlets are able to intercept more flow, but the amount intercepted (Q<sub>i</sub>) represents a smaller percentage of the total flow (Q).

### **504.3.3 Grate Inlets**

#### **Grate Inlets on Grade**

The interception capacity of a grate inlet depends on the amount and depth of water flowing over the grate, the size and configuration of the grate and the velocity of flow in the gutter. The efficiency of a grate is dependent on the same factors and total flow in the gutter. At low velocities, almost all of the water flowing in the section of gutter occupied by the grate, called frontal flow, is intercepted by grate inlets, and a small portion of the flow along the length of the grate, termed side flow, is intercepted. When the longitudinal slope and velocity is increased, water begins to skip or splash over the grate and the efficiency in accepting frontal flow is diminished. The velocity at which the gutter flow begins to splash over depends on the type and size of grate.

A parallel bar grate is the most efficient type of gutter inlet; however, when crossbars are added for bicycle safety, the efficiency is greatly reduced. Where bicycle traffic is a design consideration, the curved-vane grate and the tilt-bar grate are recommended for both hydraulic capacity and bicycle safety features. In certain locations where debris may create constant maintenance problems, the parallel-bar grate may be used more efficiently if bicycle traffic is

prohibited. Refer to the NMDOT standard drawings for grate types available, and to Chapter 4 of HEC-22 (FHWA, August 2013) for further discussion on grate types and factors affecting efficiency.

NMDOT, Website, “Standard Drawings”, “Standard Specifications for Highway and Bridge Construction”.

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

FHWA, August 2013, “HEC-22, Urban Drainage Manual, Third Edition”.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

The ratio ( $E_o$ ) of frontal flow ( $Q_w$ ), to total gutter flow ( $Q$ ), for a straight cross slope, is expressed by the following equation:

$$E_o = \frac{Q_w}{Q} = 1 - \left( 1 - \frac{W}{T} \right)^{2.67} \quad 504-9$$

(FHWA, August 2013, HEC-22, Eq. 4-16, p. 4-42)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

where:

$E_o$	=	ratio of frontal flow $Q_w$ to total gutter flow $Q$
$Q$	=	total gutter flow, $\text{ft}^3/\text{s}$
$Q_w$	=	frontal flow in width $W$ , $\text{ft}^3/\text{s}$
$W$	=	width of depressed gutter or grate, ft
$T$	=	total spread of water in the gutter, ft
$Q_s$	=	total side flow, $\text{ft}^3/\text{s}$

**Figure 504-3** provides a graphical solution of  $E_o$  for either straight cross slopes or depressed gutter sections.

The ratio of side flow ( $Q_s$ ) to total gutter flow ( $Q$ ) is:

$$\frac{Q_s}{Q} = 1 - \frac{Q_w}{Q} = 1 - E_o \quad 504-10$$

(FHWA, August 2013, HEC-22, Eq. 4-17, p. 4-42)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

The ratio of frontal flow intercepted to total frontal flow ( $R_f$ ) is expressed by the following equation:

$$R_f = 1 - K_u (V - V_o) \quad 504-11$$

(FHWA, August 2013, HEC-22, Eq. 4-18, p. 4-42)

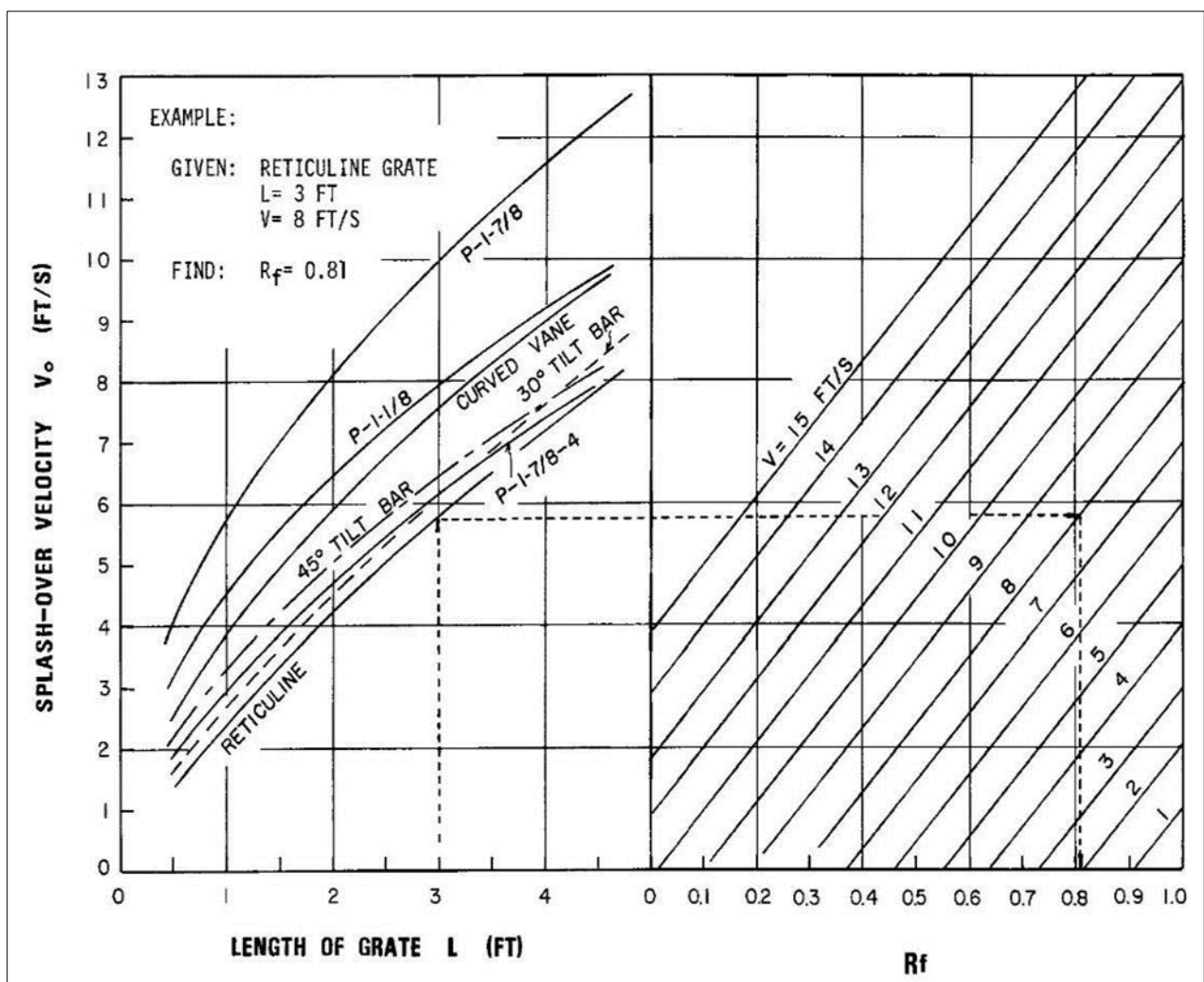
<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

where:

- $R_f$  = ratio of frontal flow intercepted to total frontal flow  
 $V$  = velocity of flow in the gutter, ft/s  
 $V_o$  = gutter velocity where splash-over first occurs, ft/s  
 $K_u$  = 0.09 (English units)

Note that  $R_f$  cannot exceed 1.0.

This ratio ( $R_f$ ) is equivalent to frontal flow interception efficiency. **Figure 504-4** gives values for  $R_f$  which takes into account grate length, bar configuration, and gutter velocity ( $V$ ) at which splash over occurs. The gutter velocity ( $V$ ) in **Figure 504-4** is total gutter flow ( $Q$ ) divided by the area of flow ( $A$ ).



Source: FHWA, August 2013, HEC-22, Chart 5B, p. A-11.

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

**Figure 504-4 Grate Inlet Frontal Flow Interception Efficiency**

The ratio of side flow intercepted to total side flow, or side flow interception efficiency ( $R_s$ ) is:

$$R_s = 1 / \left[ 1 + \left( \frac{K_u V^{1.8}}{S_x L^{2.3}} \right) \right] \quad \text{504-12}$$

(FHWA, August 2013, HEC-22, Eq. 4-19, p. 4-42)

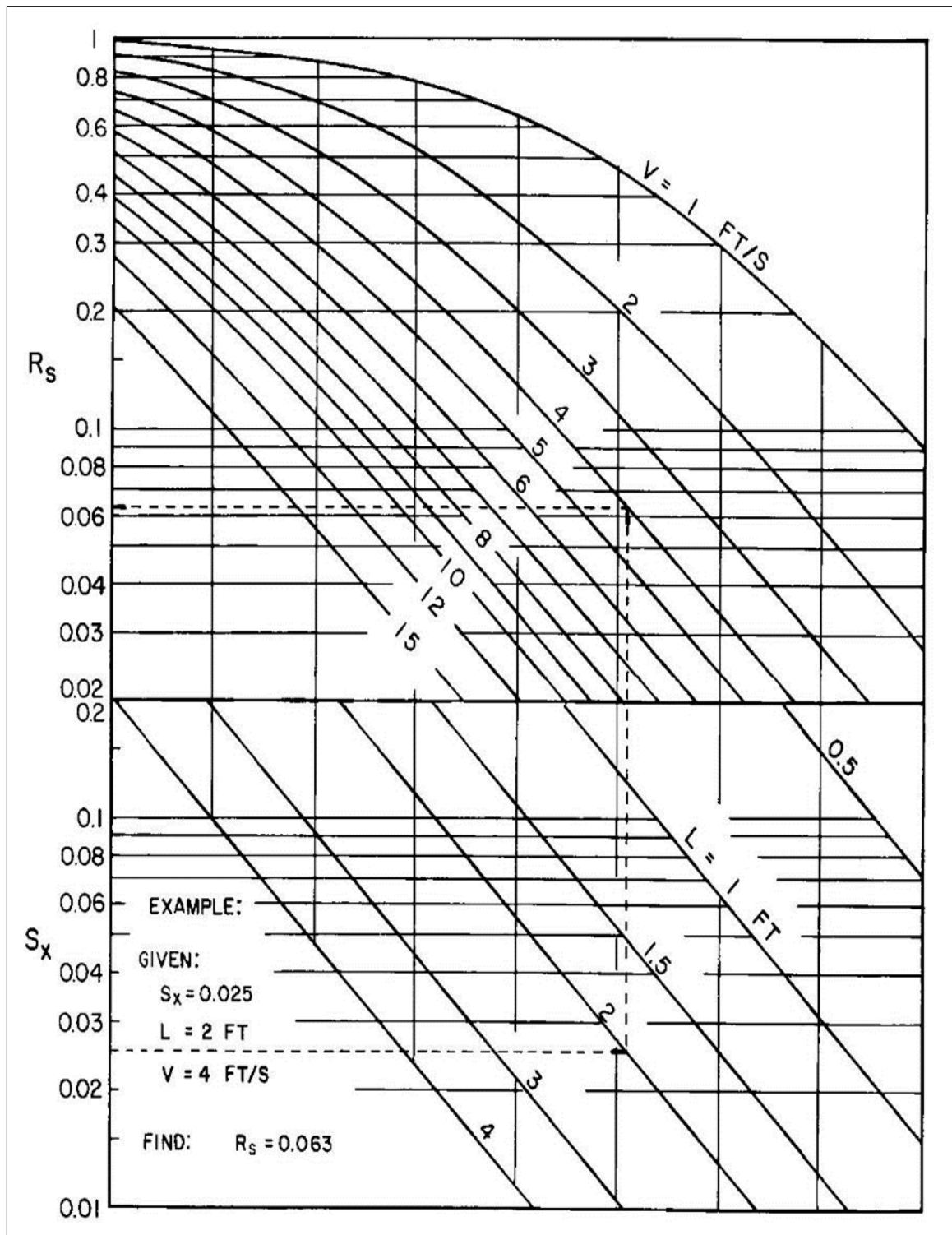
<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

where:

$R_s$	=	side flow interception efficiency
$K_u$	=	0.15 (English units)
$V$	=	velocity of flow in the gutter, ft/s
$S_x$	=	pavement cross slope, ft/ft
$L$	=	length of the grate, ft

**Figure 504-5** provides a solution to **Equation 504-12**.





Source: FHWA, August 2013, HEC-22, Chart 6B, p. A-13.

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

**Figure 504-5 Grate Inlet Side Flow Interception Capacity**

The efficiency (E) of a grate is expressed as:

$$E = R_f E_o + R_s (1 - E_o) \quad 504-13$$

(FHWA, August 2013, HEC-22, Eq. 4-20, p. 4-43)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

where:

E	=	the efficiency of a grate
$R_f$	=	ratio of frontal flow intercepted to total frontal flow
$E_o$	=	ratio of frontal flow ( $Q_w$ ) to total gutter flow (Q)
$R_s$	=	side flow interception efficiency

The first term on the right side of **Equation 504-13** is the ratio of intercepted frontal flow to total gutter flow, and the second term is the ratio of intercepted side flow to total side flow. The second term is insignificant with high velocities and short grates.

It is important to recognize that the frontal flow to total gutter flow ratio ( $E_o$ ) for composite gutter sections assumes, by definition, a frontal flow width equal to the depressed gutter section width. The use of this ratio when determining a grate's efficiency requires that the grate width be equal to the width of the depressed gutter section (W). If a grate having a width less than W is specified, the gutter flow ratio ( $E_o$ ) must be modified to accurately evaluate the grate's efficiency. Because an average velocity has been assumed for the entire width of gutter flow, the grate's frontal flow ratio ( $E'_o$ ) can be calculated by multiplying  $E_o$  by a flow area ratio. The area ratio is defined as the gutter flow area in a width equal to the grate width divided by the total flow area in the depressed gutter section. This adjustment is represented in the following equations:

$$E'_o = E_o (A'_w / A_w) \quad 504-14$$

(FHWA, August 2013, HEC-22, Eq. 4-20a, p. 4-43)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

where:

$E'_o$	=	adjusted frontal flow area ratio for grates in composite cross sections
$E_o$	=	ratio of frontal flow ( $Q_w$ ) to total gutter flow (Q)
$A'_w$	=	gutter flow area in a width equal to the grate width, ft <sup>2</sup>
$A_w$	=	flow area in depressed gutter width, ft <sup>2</sup>

The interception capacity ( $Q_i$ ) of a grate inlet on grade is equal to the efficiency (E) of the grate multiplied by the total gutter flow (Q):

$$Q_i = E Q = Q [ R_f E_o + R_s (1 - E_o) ] \quad 504-15$$

(FHWA, August 2013, HEC-22, Eq. 4-21, p. 4-43)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

Note that  $E'_o$  should be used in place of  $E_o$  when appropriate.

where:

all terms previously defined



### Grate Inlets in Sag

A grate inlet in a sag operates as a weir up to a certain depth, depending on the bar configuration and size of the grate, and as an orifice at greater depths. For a standard gutter inlet grate, weir operation continues to a depth of about 0.33 ft above the top of grate and when depth of water exceeds approximately 1.3 ft, the grate begins to operate as an orifice.

Between the approximate depths of 0.33 ft and 1.3 ft, a transition from weir to orifice flow occurs.

The capacity ( $Q_i$ ) of a grate inlet operating as a weir is:

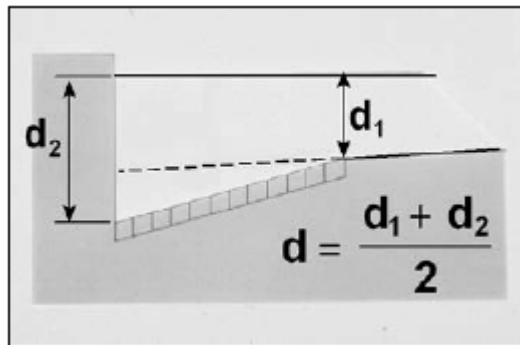
$$Q_i = C_W P d^{1.5} \quad 504-16$$

(FHWA, August 2013, HEC-22, Eq. 4-26, p. 4-57)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

where:

$Q_i$	=	grate inlet capacity, ft <sup>3</sup> /s
$P$	=	perimeter of grate excluding bar widths and the side against the curb, ft
$C_W$	=	3.0 (weir coefficient)
$d$	=	average depth across the grate; $0.5 (d_1 + d_2)$ , ft (see <b>Figure 504-6</b> )



Source: FHWA, August 2013, HEC-22, Figure 4-17, p. 4-57.

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

**Figure 504-6 Definition of Depth**

The capacity of a grate inlet operating as an orifice ( $Q_i$ ) is:

$$Q_i = C_o A_g (2 g d)^{0.5} \quad 504-17$$

(FHWA, August 2013, HEC-22, Eq. 4-27, p. 4-57)

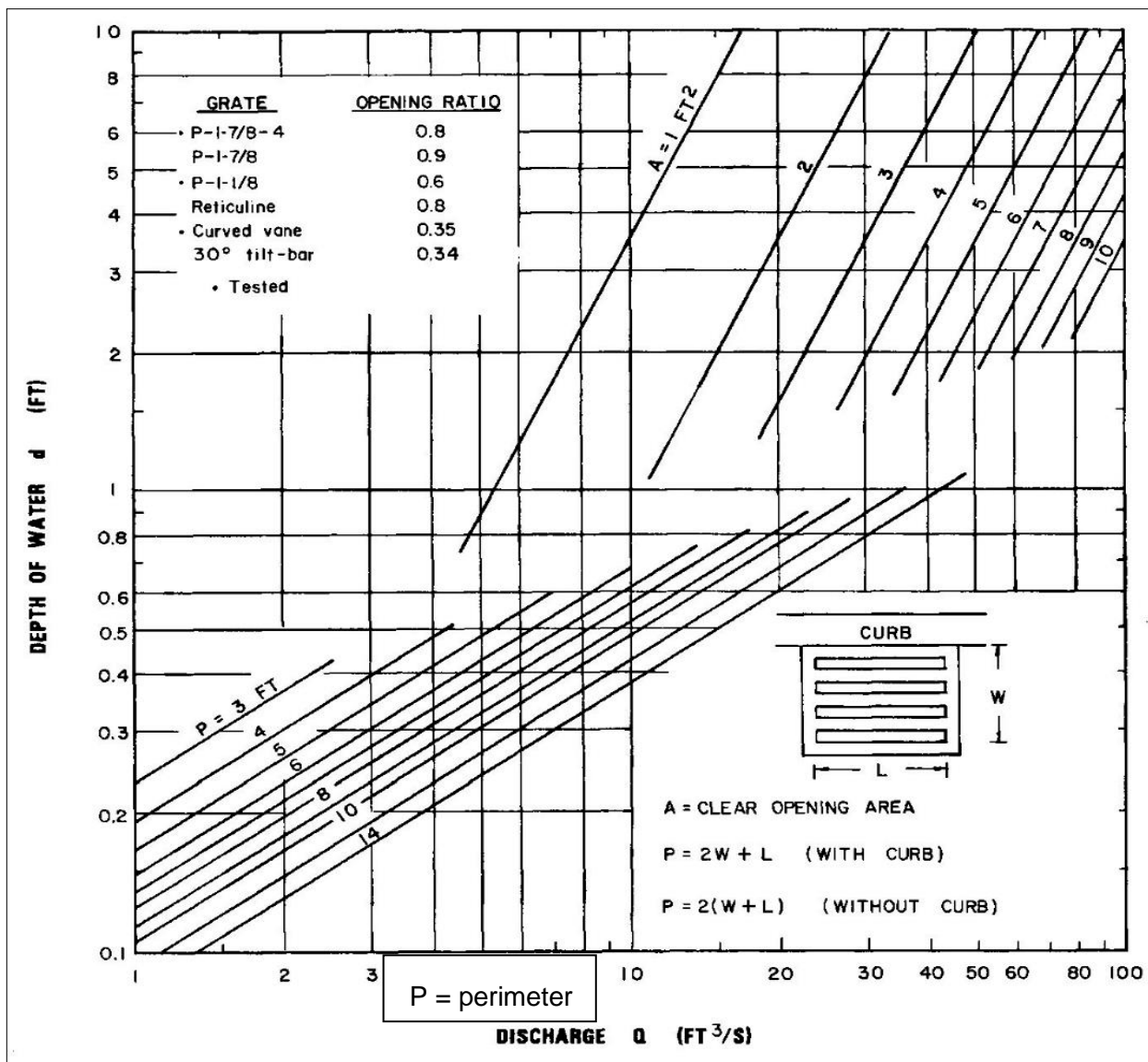
<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

where:

$Q_i$	=	grate inlet capacity, ft <sup>3</sup> /s
$C_o$	=	0.67 (orifice coefficient)
$A_g$	=	clear opening area of the grate, ft <sup>2</sup>
$g$	=	gravitational acceleration, 32.2 ft/s <sup>2</sup>
$d$	=	average depth across the grate; 0.5 ( $d_1$ + $d_2$ ), ft

**Figure 504-7** is a plot of **Equations 504-16** and **504-17** for various grate sizes. The effects of grate size on the depth at which a grate operates as an orifice is apparent from the figure. Transition from weir to orifice flow results in interception capacity less than that computed by either the weir or the orifice equation. This capacity can be approximated by drawing in a curve between the lines representing the perimeter and net area of the grate to be used. Refer to p. 4-58 of HEC-22, (FHWA, August 2013) for an example problem that illustrates use of **Figure 504-7** to determine grate inlet capacity in a sag condition.

FHWA, August 2013, "HEC-22, Urban Drainage Manual, Third Edition".  
<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>



Source: FHWA, August 2013, HEC-22, Chart 9B, p. A-19.

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

**Figure 504-7 Grate Inlet Capacity in Sag Conditions**

### 504.3.4 Curb-Opening Inlets

#### Curb Opening Inlets on Grade

Interception capacity of a curb-opening inlet is largely dependent on flow depth at the curb and curb opening length. Curb-opening inlets are effective in the drainage of highway pavements where flow depth at the curb is sufficient for the inlet to perform efficiently. Curb-opening inlets are relatively free of clogging tendencies and offer little interference to traffic operation. They are a viable alternative to grates in many locations where grates would be in traffic lanes or would be hazardous for pedestrians or bicyclists.

The length of curb opening inlet (L) required for total interception of gutter flow on a pavement section with a straight cross slope is expressed by:

$$L_T = K_u Q^{0.42} S_L^{0.3} [1 / (n S_x)]^{0.6} \quad \text{504-18}$$

(FHWA, August 2013, HEC-22, Eq. 4-22, p. 4-46)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

where:

$L_T$	=	curb opening length required to intercept 100% of the gutter flow, ft
$K_u$	=	0.6
$Q$	=	gutter flow, ft <sup>3</sup> /s
$S_L$	=	longitudinal slope, ft/ft
$n$	=	Manning's roughness coefficient of pavement
$S_x$	=	pavement cross slope, ft/ft

The efficiency (E) of curb-opening inlets shorter than the length required for total interception is expressed by:

$$E = 1 - (1 - L / L_T)^{1.8} \quad \text{504-19}$$

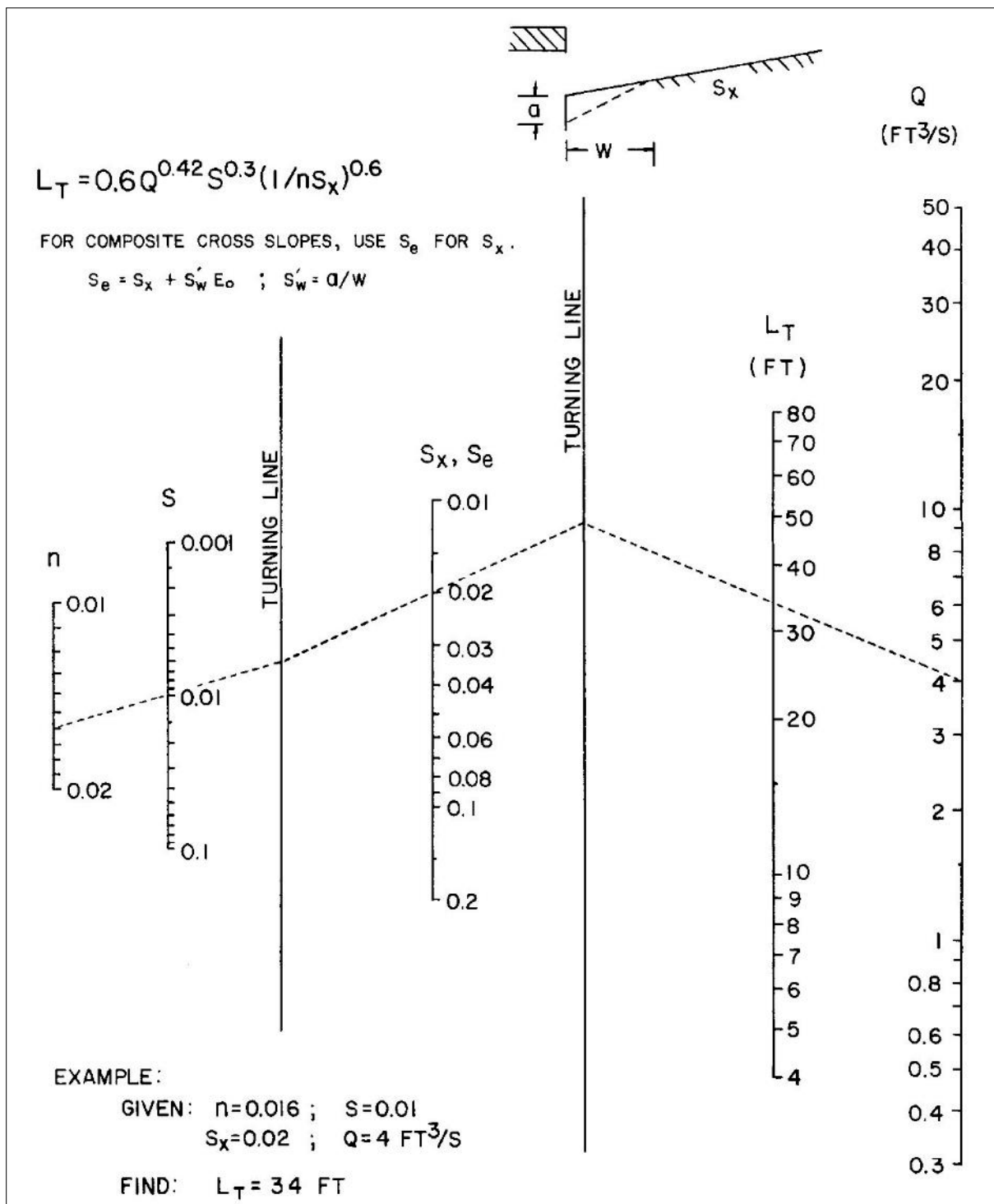
(FHWA, August 2013, HEC-22, Eq. 4-23, p. 4-46)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

where:

$E$	=	curb opening efficiency
$L$	=	curb opening length, ft
$L_T$	=	curb opening length required to intercept 100% of the gutter flow, ft

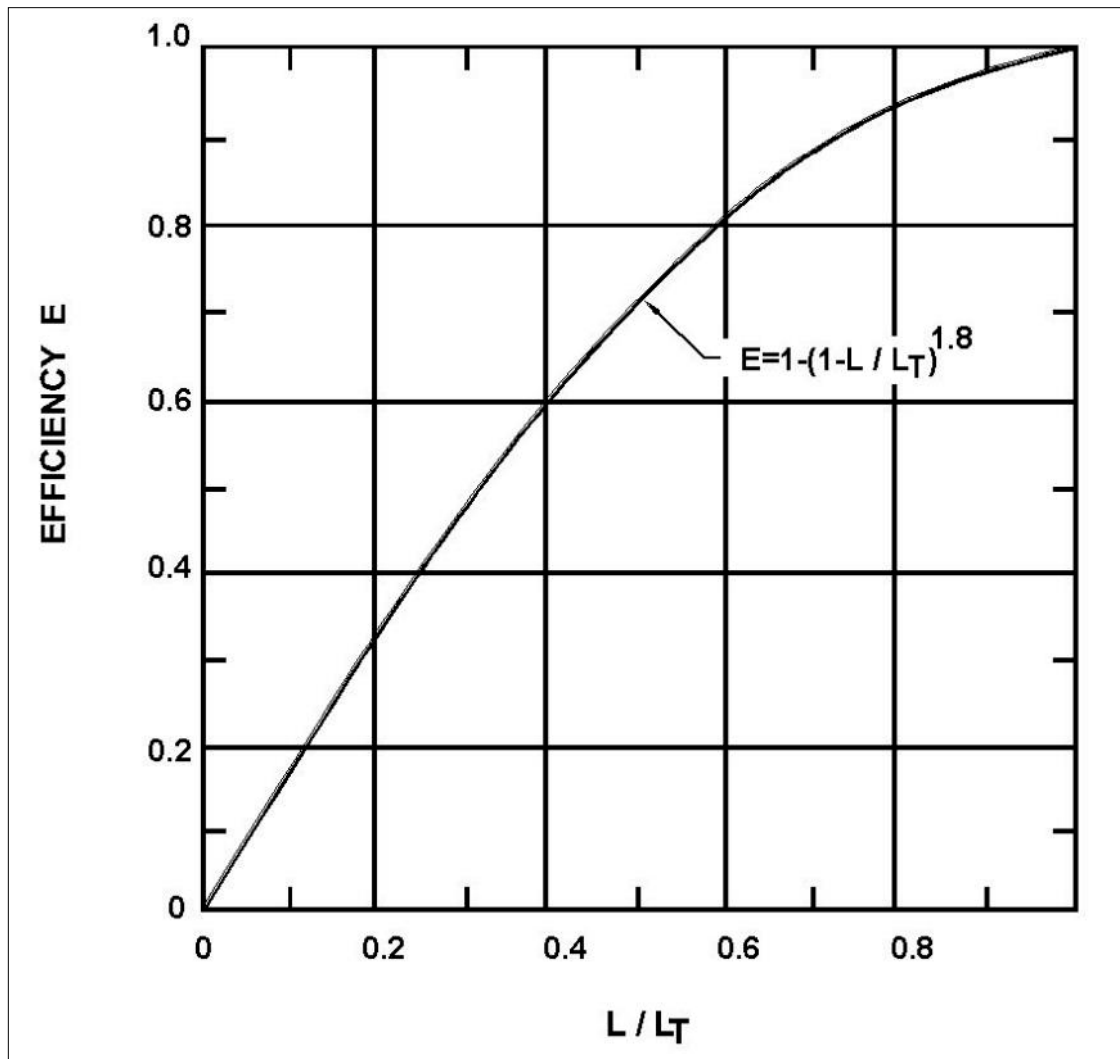
**Figure 504-8** is a nomograph for the solution of **Equation 504-18** and **Figure 504-9** provides a solution of **Equation 504-19**.



Source: FHWA, August 2013, HEC-22, Chart 7B, p. A-15.

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

Figure 504-8 Curb-Opening and Slotted Drain Inlet Length for Total Interception



Source: FHWA, August 2013, HEC-22, Chart 8B, A-17.

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

**Figure 504-9 Curb-Opening and Slotted Drain Inlet Interception Efficiency**

The length of inlet required for total interception by depressed curb opening inlets or curb openings in depressed gutter sections can be found by the use of an equivalent cross slope,  $S_e$ , in **Equation 504-20**.

$$S_e = S_x + S'_w E_o$$

**504-20**

(FHWA, August 2013, HEC-22, Eq. 4-24, p. 4-46)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

where:

$S_e$	=	equivalent pavement cross slope, ft/ft
$S_x$	=	pavement cross slope, ft/ft
$S'_w$	=	cross slope of the gutter measured from the cross slope of the pavement, $S_x$ , ft/ft $S'_w = (a / [12 W])$ (ft/ft) or $S_w - S_x$
$a$	=	gutter depression, inches
$W$	=	width of depressed gutter, ft
$E_o$	=	ratio of flow in the depressed section to total gutter flow determined by the gutter configuration upstream of the inlet

NOTE:  $S_e$  can be used to calculate the length of curb opening by substituting for  $S_x$  in **Equation 504-18**.

### Curb Opening Inlets in a Sag

The capacity of a curb-opening inlet in a sag depends on water depth at the curb, the curb opening length, and the height of the curb opening. The inlet operates as a weir at depths equal to the curb opening height and as an orifice at depths greater than 1.4 times the opening height. At depths between 1.0 and 1.4 times the opening height, flow is in a transition stage.

The equation for the interception capacity ( $Q_i$ ) of a depressed curb-opening inlet operating as a weir is:

$$Q_i = C_w (L + 1.8 W) d^{1.5} \quad \text{504-21}$$

(FHWA, August 2013, HEC-22, Eq. 4-28, p. 4-59)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

where:

$Q_i$	=	grate inlet capacity, ft <sup>3</sup> /s
$C_w$	=	2.3 (weir coefficient)
$L$	=	length of the curb opening, ft
$W$	=	lateral width of depression, ft
$d$	=	depth of water at curb measured from the normal cross slope gutter flow line, ft, i.e., $d = T S_x$

The weir equation is applicable to depths at the curb approximately equal to the height of the opening plus the depth of the depression ( $d$ ). Thus, the limitation on the use of **Equation 504-21** for a depressed curb-opening inlet is:

$$d \leq h + a/12 \quad \text{504-22}$$

(FHWA, August 2013, HEC-22, Eq. 4-29, p. 4-60)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

where:

d	=	depth of water at curb measured from the normal cross slope gutter flow line, ft, i.e., $d = T S_x$
h	=	height of curb-opening inlet, ft
a	=	depth of depression, ft

Experiments have not been conducted for curb-opening inlets with a continuously depressed gutter, but it is reasonable to expect that the effective weir length would be as great as that for an inlet in a local depression. Use of **Equation 504-21** will yield conservative estimates of the interception capacity.

The weir equation for curb opening inlets without depression becomes:

$$Q_i = C_w L d^{1.5} \quad 504-23$$

(FHWA, August 2013, HEC-22, Eq. 4-30, p. 4-60)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

where:

all terms previously defined

Without depression of the gutter section, the weir coefficient ( $C_w$ ) becomes 3.0. The depth limitation for operation as a weir becomes  $d \leq h$ .

At curb-opening lengths greater than 12 ft, **Equation 504-23** for non-depressed inlet produces intercepted flows which exceed the values for depressed inlets computed using **Equation 504-21**. Since depressed inlets will perform at least as well as non-depressed inlets of the same length, **Equation 504-23** should be used for all curb opening inlets having lengths greater than 12 ft.

Curb-opening inlets operate as orifices at depths greater than approximately 1.4 times the opening height. The interception capacity ( $Q_i$ ) can be computed by **Equation 504-24** and **Equation 504-25**.

$$Q_i = C_o h L (2 g d_o)^{0.5} \quad 504-24$$

(FHWA, August 2013, HEC-22, Eq. 4-31a, p. 4-60)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

or

$$Q_i = C_o A_g [2 g (d_i - h / 2)]^{0.5} \quad 504-25$$

(FHWA, August 2013, HEC-22, Eq. 4-31b, p. 4-60)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>



where:

$Q_i$	=	grate inlet capacity, ft <sup>3</sup> /s
$C_o$	=	0.67 (orifice coefficient)
$d_o$	=	effective head on the center of the orifice throat, ft
$L$	=	length of orifice opening, ft
$A_g$	=	clear area of opening, ft <sup>2</sup>
$d_i$	=	depth at lip of curb opening, ft
$h$	=	height of curb-opening orifice, ft
$g$	=	gravitational acceleration, 32.2 ft/s <sup>2</sup>

NOTE: **Equations 504-24** and **504-25** are applicable to depressed and undepressed curb opening inlets. The depth at the inlet includes any gutter depression.

Nomographs also provide solutions for **Equations 504-21, 504-23, 504-24** and **504-25** as included in Appendix A of HEC-22 (FHWA, August 2013).

FHWA, August 2013, "HEC-22, Urban Drainage Manual, Third Edition".  
<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

### 504.3.5 Slotted-Drain Inlets

Slotted inlets function in essentially the same manner as curb opening inlets, i.e., as weirs with flow entering from the side. Interception capacity is dependent on flow depth and inlet length. Efficiency is dependent on flow depth, inlet length, and total gutter flow. Deposition in the pipe is the problem most commonly encountered. The inlet is accessible for cleaning only with a high-pressure water jet. They can be used on curbed or uncurbed sections and offer little interference to traffic operations. Due to maintenance difficulties, slotted drain inlets can be used on NMDOT projects only with Drainage Design Bureau consultation and approval.

#### Slotted-Drain Inlets On Grade

Flow interception by slotted-drain inlets and curb-opening inlets is similar in that each is a side weir and the flow is subjected to lateral acceleration due to the cross slope of the pavement.

Thus, the equations and charts used for curb-opening inlets on grade can be used for the design and analysis of slotted-drain inlets on grade.

#### Slotted-Drain Inlets In Sag

Slotted-drain inlets in sag locations perform as weirs at approximately 0.2 ft, dependent on slot width and length. Generally, at depths greater than 0.4 ft, they perform as orifices. Between these depths, flow is in a transition stage.

The interception capacity ( $Q_i$ ) of a slotted-drain inlet operating as a weir can be computed by the following equation:

$$Q_i = C_w L d^{1.5} \quad 504-26$$

(FHWA, August 2013, HEC-22, Eq. 4-32, p. 4-63)

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

where:

$Q_i$	=	grate inlet capacity, ft <sup>3</sup> /s
$C_w$	=	weir coefficient; varies with flow depth and slot length; a typical value is approximately 2.48
$L$	=	length of slot, ft
$d$	=	depth at curb measured from the normal cross slope, ft

**Figure 504-10** provides solutions for weir flow and a plot representing data at depths between weir and orifice flow.

The interception capacity ( $Q_i$ ) of a slotted-drain inlet operating as an orifice can be computed by the following equation:

$$Q_i = 0.8 L W (2 g d)^{0.5} \quad \text{504-27}$$

(FHWA, August 2013, HEC-22, Eq. 4-33, p. 4-63)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

where:

$Q_i$	=	grate inlet capacity, ft <sup>3</sup> /s
$W$	=	lateral width of slot, ft
$L$	=	length of slot, ft
$d$	=	depth of water at slot for $d > 0.4$ ft, ft
$g$	=	gravitational acceleration, 32.2 ft/s <sup>2</sup>

For a slot width of 1.75 inches, **Equation 504-27** becomes:

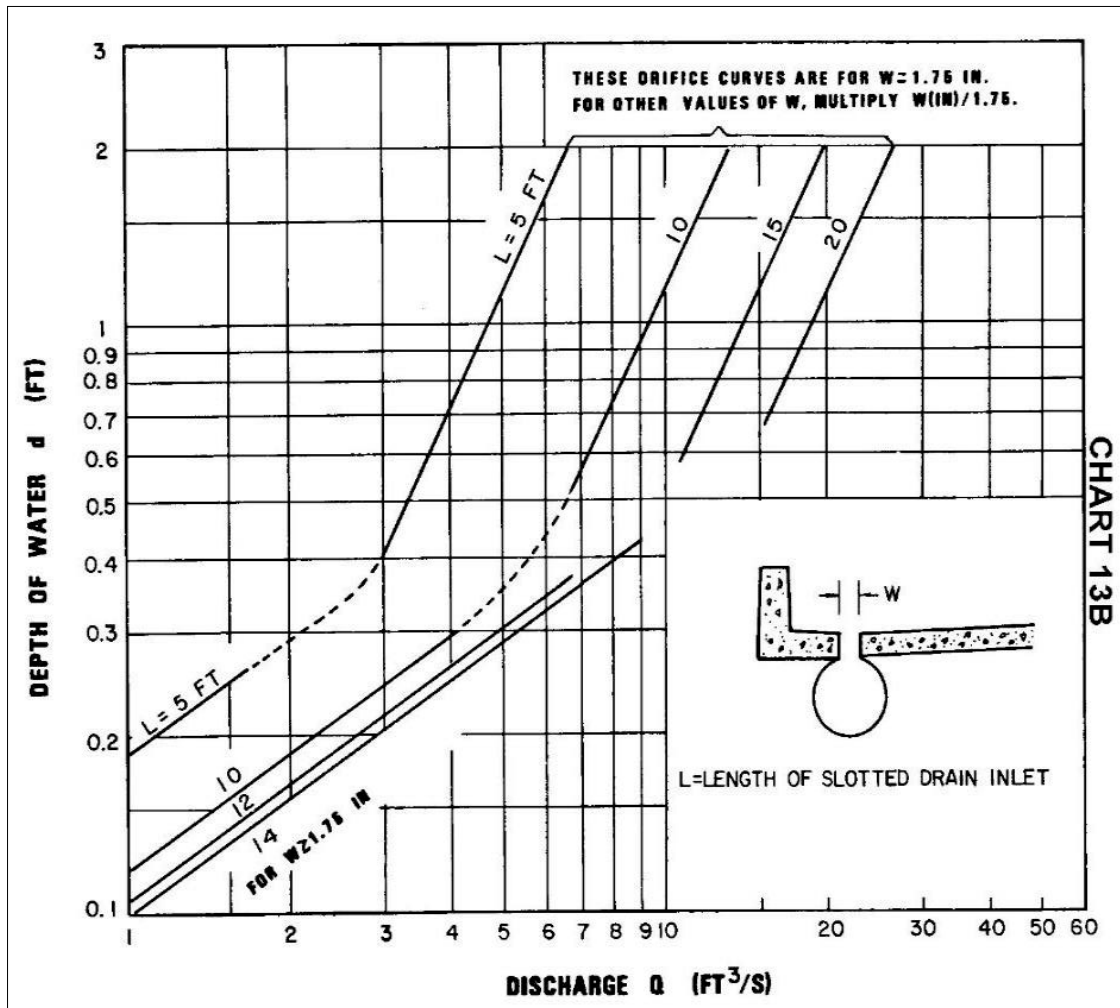
$$Q_i = C_D L d^{0.5} \quad \text{504-28}$$

(FHWA, August 2013, HEC-22, Eq. 4-34, p. 4-64)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

where:

$Q_i$	=	grate inlet capacity, ft <sup>3</sup> /s
$C_D$	=	0.94 for English units
$L$	=	length of slot, ft
$d$	=	depth of water at slot for $d > 0.4$ ft, ft



Source: FHWA, August 2013, HEC-22, Chart 13B, p. A-27.

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

**Figure 504-10 Slotted-Drain Inlet Capacity in Sump Locations**

### 504.3.6 Combination Inlet

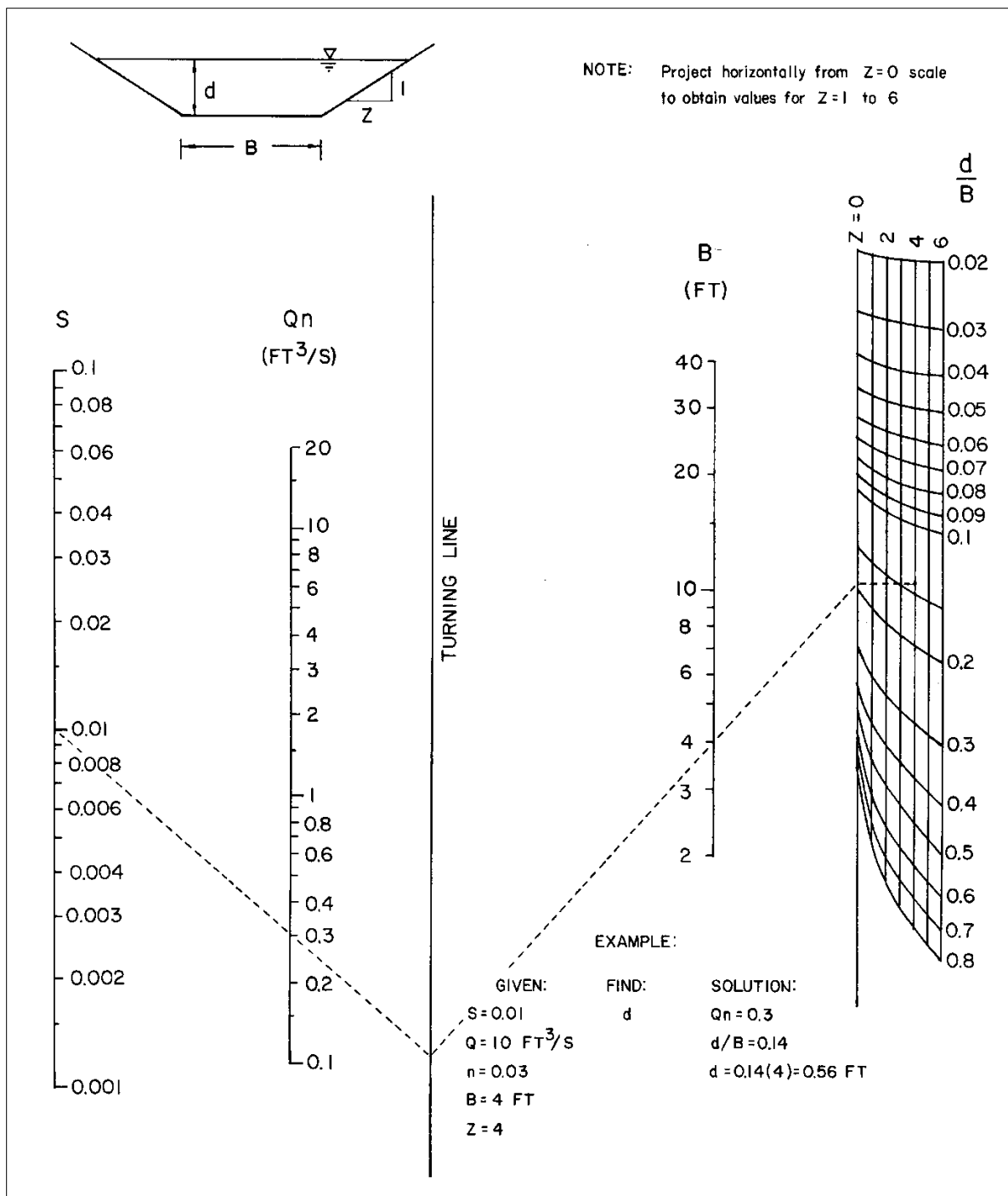
The interception capacity of a combination inlet on a continuous grade is not substantially greater than that of a grate alone. In computing the inlet capacity, the curb opening is neglected and only grate opening is considered. The use of a combination inlet in a sag is desirable. The curb opening provides a relief opening if the grate should become clogged. The capacity of a combination inlet in a sag, is essentially the same as the grate alone in weir flow conditions unless the grate opening becomes clogged. In orifice flow, the capacity is equal to the total capacity of grate and curb opening.

### 504.3.7 Median Drop Inlet

Median ditches may be drained by drop inlets similar to those used for pavement drainage, by pipe culverts under one roadway, or by cross drainage culverts which are not continuous

across the median. Drop inlets should be flush with the ditch bottom and traffic-safe bar grates should be placed on the ends of pipes used to drain medians that would be a hazard to errant vehicles, although this may cause a plugging potential. Cross drainage structures should be continuous across the median unless the median width makes this impractical. Ditches tend to erode at drop inlets; paving around the inlets helps to prevent erosion and may increase the interception capacity of the inlet marginally by acceleration of the flow.

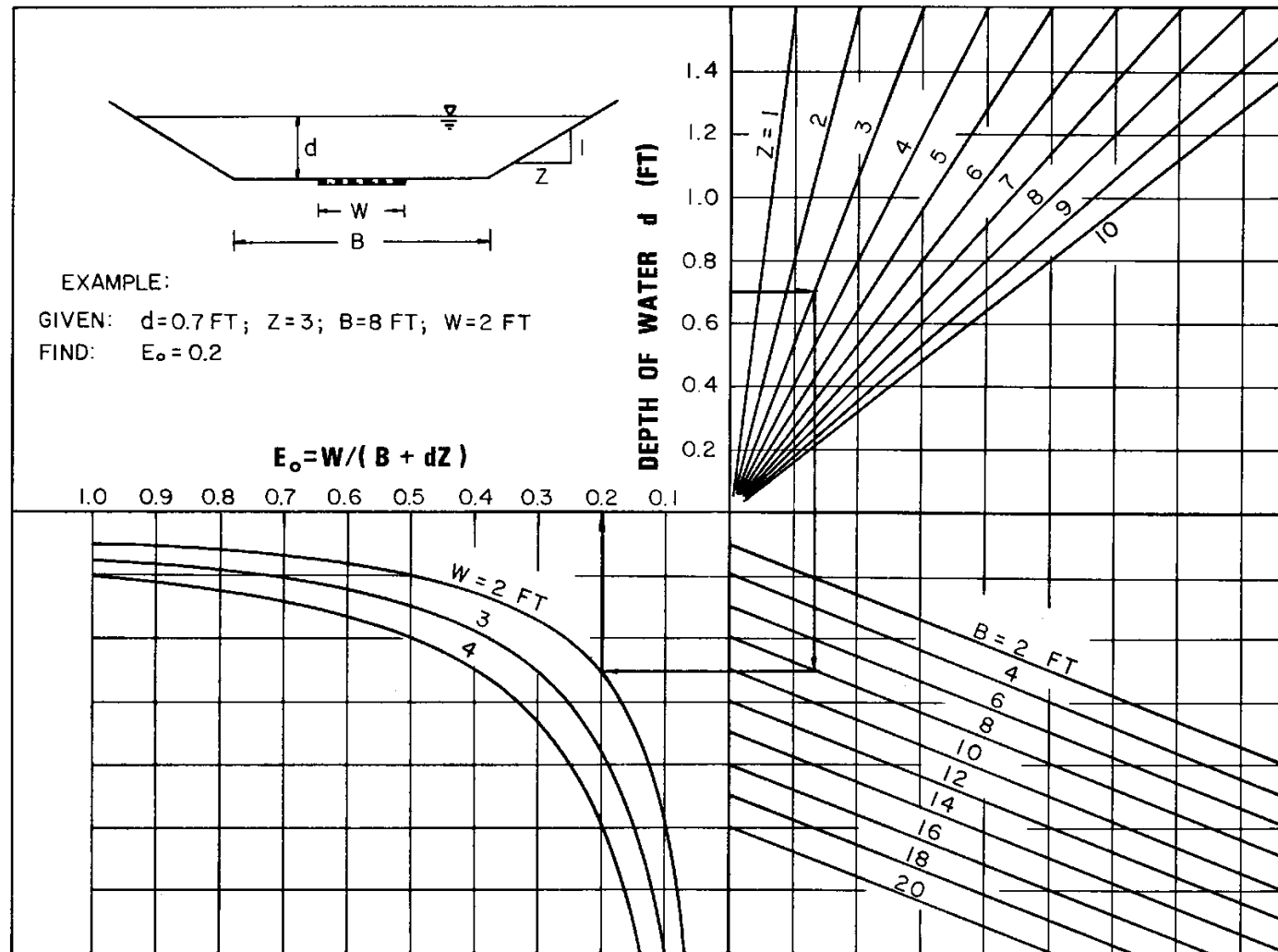
The interception capacity of drop inlets in median ditches on continuous grades can be estimated by use of **Figure 504-11** and **Figure 504-12** (to estimate flow depth and the ratio of frontal flow to total flow in the ditch).



(FHWA, August 2013, HEC-22, Chart 14B, p. A-29)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

**Figure 504-11 Solution to Manning's Equation for Channels of Various Side Slopes**



(FHWA, August 2013, HEC-22, Chart 15B, p. A-31) <https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

**Figure 504-12 Ratio of Frontal Flow to Total Flow in a Trapezoidal Channel**

**Figure 504-11** is the solution to the Manning's Equation for channels of various side slopes. Manning's Equation for open channels is:

$$Q = (K_u / n) A R^{0.67} S_L^{0.5} \quad 504-29$$

(FHWA, August 2013, HEC-22, Eq. 4-38, p. 4-83)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

where:

Q	=	discharge rate, ft <sup>3</sup> /s
K <sub>u</sub>	=	coefficient 1.486
n	=	hydraulic resistance variable
A	=	cross sectional area of flow, ft <sup>2</sup>
R	=	hydraulic radius = area (A)/wetted perimeter (P), ft
S <sub>L</sub>	=	bed slope, ft/ft

For the trapezoidal channel cross section shown on **Figure 504-11**, Manning's Equation becomes:

$$Q = (K_u / n) (Bd + Zd^2) \left\{ (Bd + Zd^2) / [B + 2d (Z^2 + 1)^{0.5}] \right\}^{0.67} S_L^{0.5} \quad 504-30$$

(FHWA, August 2013, HEC-22, Eq. 4-39, p. 4-83)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

where:

Q	=	discharge rate, ft <sup>3</sup> /s
K <sub>u</sub>	=	coefficient 1.486
n	=	hydraulic coefficient
B	=	ditch bottom width, ft
d	=	flow depth, ft
Z	=	horizontal distance of side slope to a rise of 1 ft vertical
S <sub>L</sub>	=	bed slope, ft/ft

**Equation 504-30** is a trial and error solution to **Figure 504-11**.

**Figure 504-12** is the ratio (E<sub>0</sub>) of frontal flow to total flow in a trapezoidal channel. This is expressed as:

$$E_o = W / (B + d Z)$$

**504-31**

(FHWA, August 2013, HEC-22, Eq. 4-40, p. 4-83)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

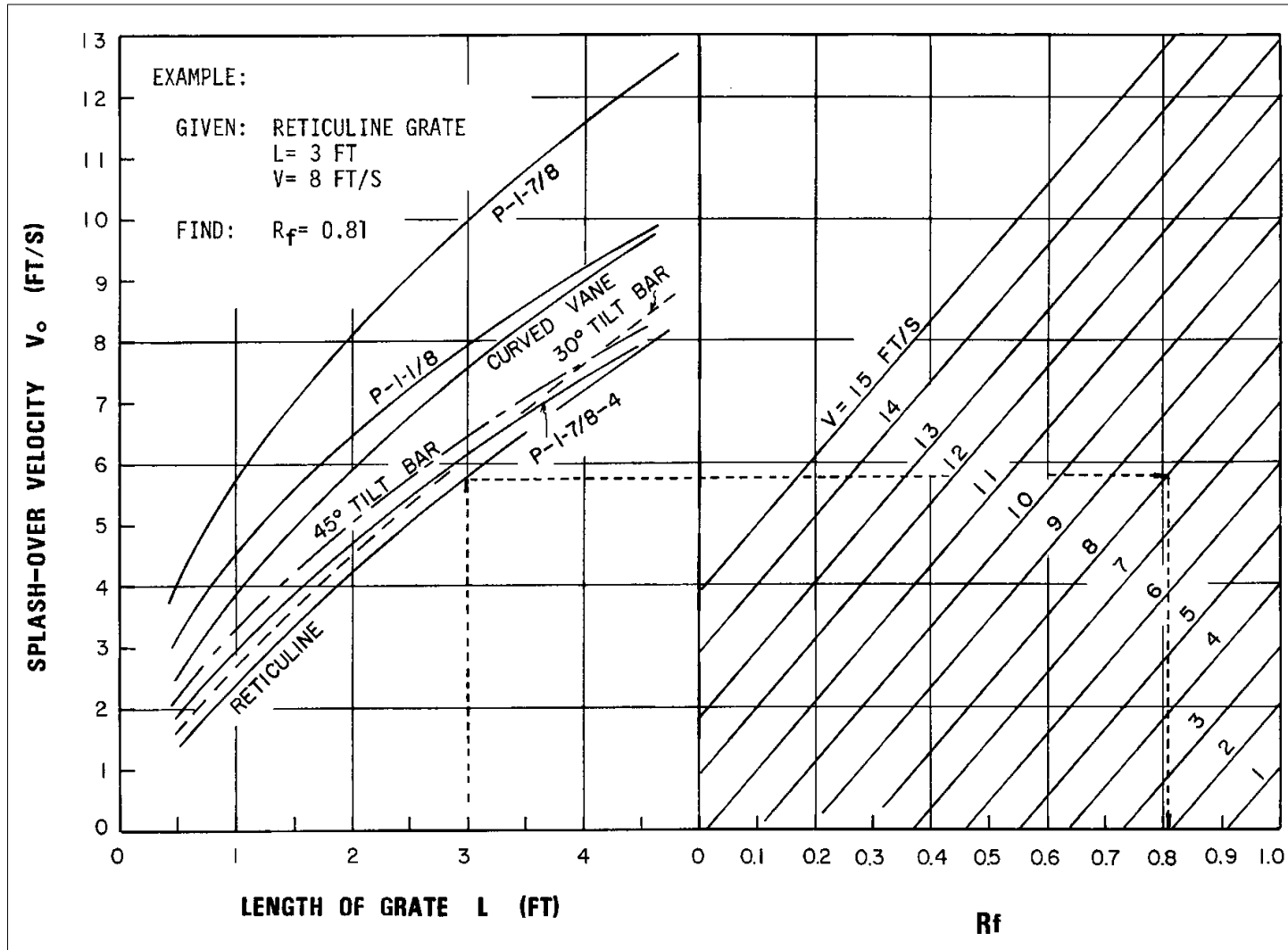
where:

$E_o$	=	ratio of grate frontal flow (W) to total flow (B + d Z)
$W$	=	grate width in flow direction, ft
$B$	=	ditch bottom width, ft
$d$	=	flow depth, ft
$Z$	=	horizontal distance of side slope to a rise of 1 ft vertical

**Figure 504-13** and **Figure 504-14** are used to estimate the ratios of frontal and side flow intercepted by the grate to total flow.

Small dikes downstream of drop inlets can be provided to impede bypass flow in an attempt to cause complete interception of the approach flow. The dikes usually need not be more than a few inches high and should have traffic safe slopes. The height of dike required for complete interception on continuous grades or the depth of ponding in sag vertical curves can be computed by use of **Figure 504-15**. The effective perimeter of a grate in an open channel with a dike should be taken as  $2(L + W)$  since one side of the grate is not adjacent to a curb.

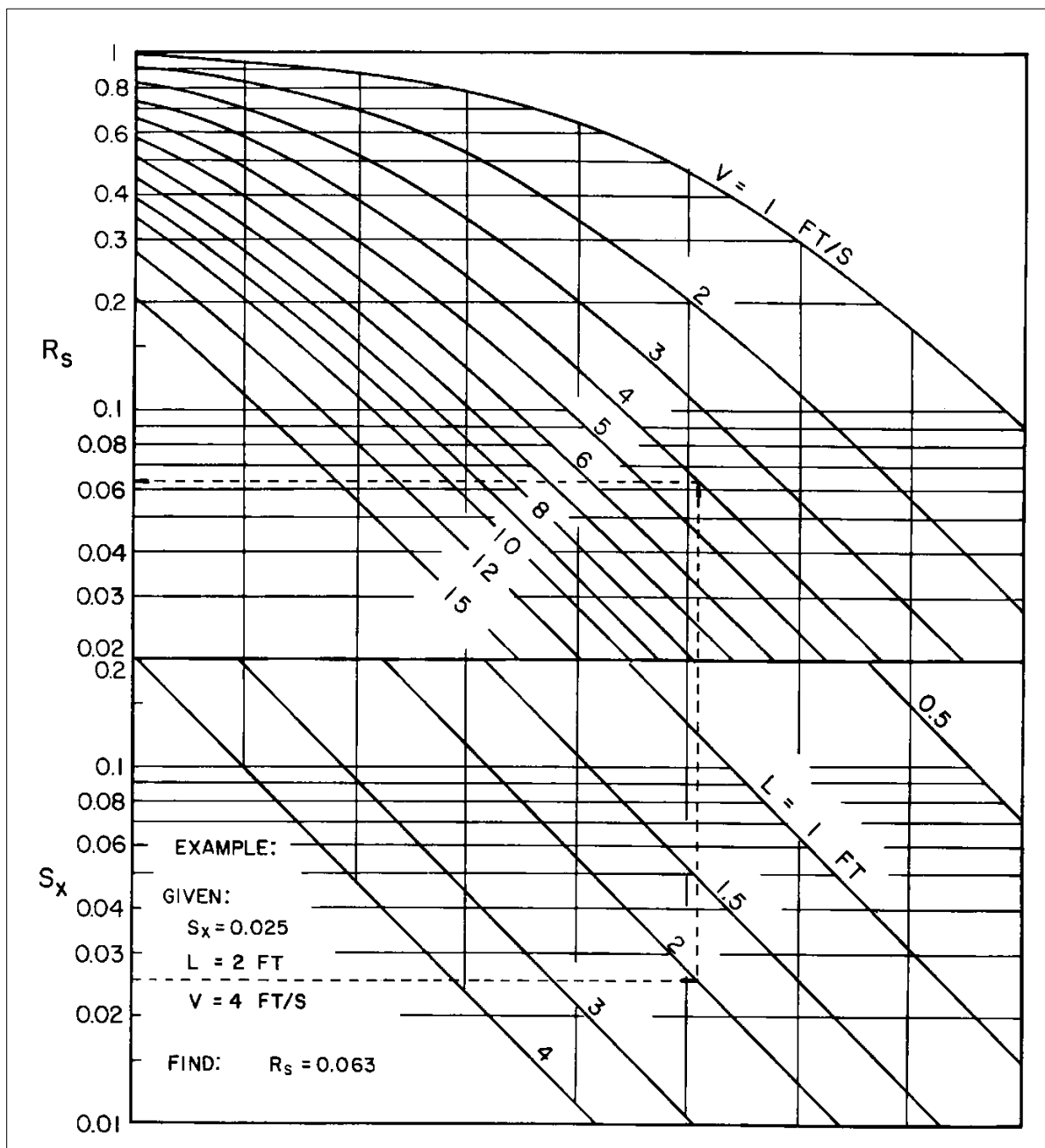




(FHWA, August 2013, HEC-22, Chart 5B, p. A-11)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

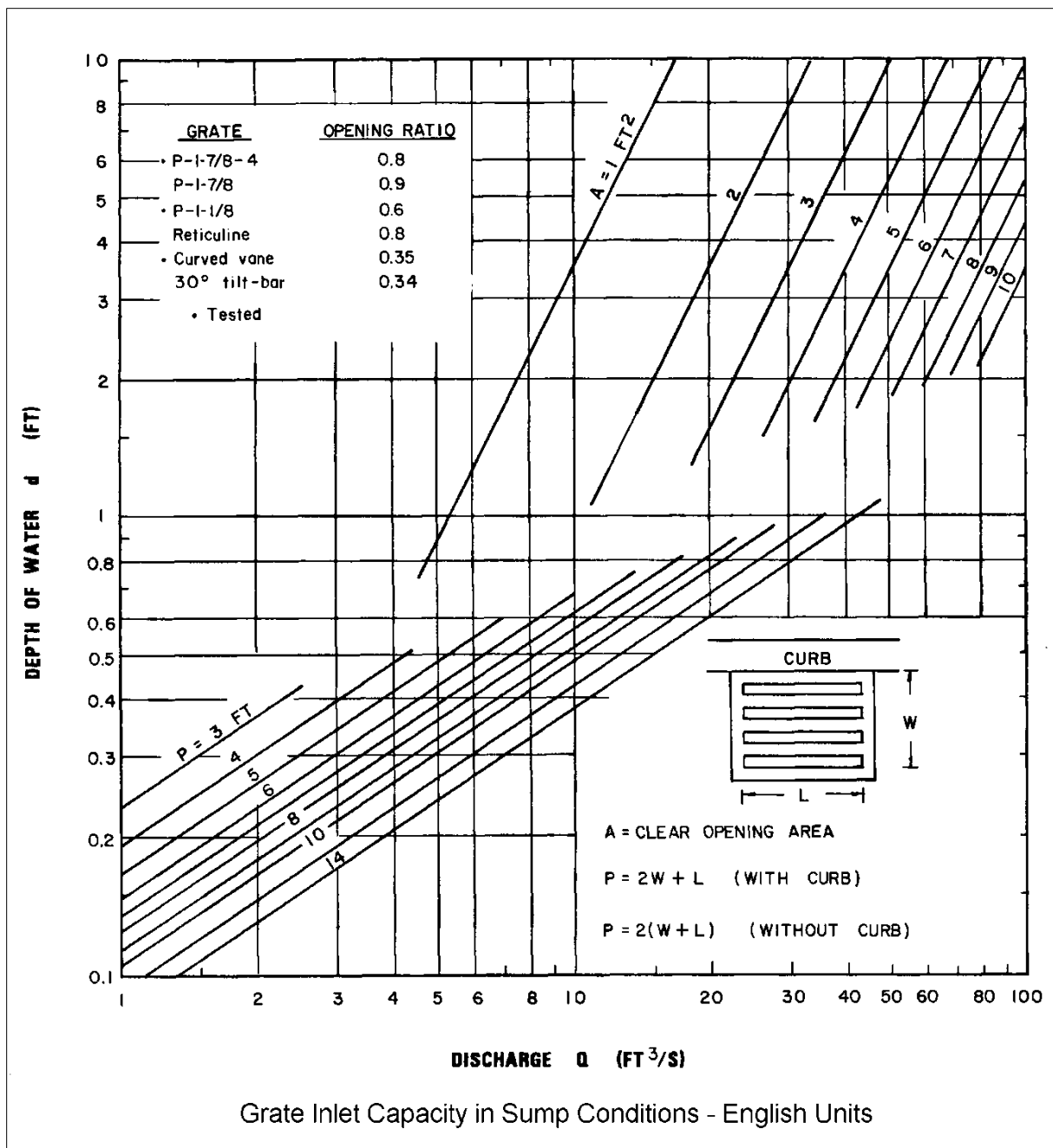
**Figure 504-13 Grate Inlet Frontal Flow Interception Efficiency**



(FHWA, August 2013, HEC-22, Chart 6B, p. A-13)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

**Figure 504-14 Grate Inlet Side Flow Intercept Efficiency**



(FHWA, August 2013, HEC-22, Chart 9B, p. A-19)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

**Figure 504-15 Grate Inlet Capacity in Sump Conditions**

Appendix 7 contains **Example Problem 7-2** that presents street flow and inlet capacity simulated with the FlowMaster program. FlowMaster is commercially available and provides an acceptable analysis for NMDOT drainage analysis, though not required, by the NMDOT Drainage Design Bureau.

#### **504.4 Catch Basins**

Catch basins are designed to collect surface water. However, they differ from inlets in that a sump for collection of silt and debris is provided. Catch basin sumps require periodic cleaning to be effective and may become an odor and/or mosquito nuisance if not properly maintained. A properly designed storm drainage system should be self-cleaning. Therefore, the use of catch basins should be avoided. Refer to the NMDOT standard drawings or the standards of other agencies having jurisdiction for the various catch basin types available for the specific project location.

#### **504.5 Storm Drains**

A storm drain is that portion of the highway drainage system that receives surface water through inlets and conveys the water through conduits to an outfall. Cross drain pipes or culverts, "hydraulically designed" to function as a culvert, often collect water from just one drop inlet or no inlets.

Storm drains shall be designed to maintain the hydraulic grade line at or below the criteria as defined in **Table 204-1** and **Table 204-2**. Storm drains should be designed to avoid existing utilities. Storm drain outfalls must be designed to dissipate energy and minimize erosion.

Detention ponds may provide a means to reduce discharges, promote sediment deposition and retention, and improve stormwater quality for the outflows. Detention ponds also provide a means to reduce storm drain size providing several benefits:

- Cost savings
- Utility conflict avoidance due to smaller pipe size
- Reduced storm drain pipe maintenance due to sediment deposition in the pond

#### **Curved Storm Drains**

Curved storm drains are permitted in special cases. Requirements for curved storm drains are:

- Location: Curved alignments should follow the general street alignment
- Curve Type: Only simple curve design is acceptable
- Radius of Curvature: The minimum allowable radius of curvature is 325 feet
- Manholes: Manholes or inlets are required at the beginning and end of all curves
- Velocity: The minimum velocity shall be 3 feet per second for full-flow condition and the maximum velocity should not exceed 10 feet per second
- Joints: Compression joints are required. The ASTM maximum allowable pipe joint deflection shall not be exceeded.

If curved storm drain alignments are contemplated, the local pipe manufacturer should be consulted regarding manufacturing, installation feasibility, and cleaning equipment access for storm drain maintenance. Many manufacturers have joint configurations and deflections for specific radii which conform to the ASTM and AWWA specifications.

##### **504.5.1 Hydraulic Analysis and Design Procedure**

Prepare a plan view map indicating contours, spot elevations, utility types and locations, roadway intersections, roadway limits, sidewalk, curb and gutter locations, and other features if present.

Prepare a roadway profile with slopes and critical elevations. The profile must also include all utilities (label type, annotate material, draw diameter, annotate elevations, etc.).

The storm drain conduit system and the analysis must meet spread and hydraulic grade line (HGL) criteria as presented in **Table 204-1** and **Table 204-2**.

The analysis must be completed for the design and checked for peak discharges. The steps required to plan and prepare a storm drain system design are:

- Step 1 Plot the off-site subbasin inflow locations, and annotate the peak discharges.
- Step 2 Develop roadway subbasins and analysis points along the roadway in plan view. Annotate the analysis point peak discharges.
- Step 3 Compute pavement spread and ensure spread is less than the allowable spread criteria.
- Step 4 Determine inlet locations, number, and inlet types required to reduce gutter flow in order to meet spread criteria (3).
- Step 5 Prepare a flow accounting table to document the inlet bypass flows, roadway subbasin flows, and/or additional offsite subbasin inflows, and compute the next downstream analysis point peak discharges.
- Step 6 Repeat steps 3, 4 and 5 as required to meet criteria.
- Step 7 Plot manhole locations required to meet criteria.
- Step 8 Determine storm drain diameters required to avoid utility conflicts.
- Step 9 Determine storm drain size required to meet the hydraulic grade line criteria. Final sizes will be determined by a trial and error process based on the hydraulic grade line analysis and elevation results, and utility issues.
- Step 10 Prepare the final storm drain plan and profile sheet and annotate the HGL elevations at all inlets and all manholes. Also annotate all grate elevations, manhole rim and, invert elevations. See **Section 307** for all required information to present on the plan and profile sheets.

### **504.5.2 Access Holes (Manholes)**

Manholes are utilized to:

- Provide access to storm drains for inspection and maintenance, as well as allow connection of two or more storm drains to an outlet storm drain
- Provide a junction for long storm drain segments

Typical locations where manholes should be specified are:

- At intermediate points along tangent sections
- Where pipe sizes change, and multiple pipes must junction
- Where an abrupt change in grade occurs
- Where an abrupt change in alignment occurs

Manholes may be inadequate where the following situations may exist:

- Three or more storm drains combine
- Where high velocities exist

- Where excessive pressures exist (hydraulic grade line excessively high above the manhole rim)
- Where large diameter storm drains combine, if so, a junction box may be required

Refer to **Section 200** for additional storm drain design requirements.

Where feasible, curb inlets should be used in lieu of manholes. The inlet can provide access to the system and the inlet also provides for stormwater interception. Avoid locating manholes in traffic lanes. However, if required, try and avoid the normal vehicle path.

### 504.5.3 Capacity

Manning's Equation is the most widely used formula for determining velocity and subsequently discharge and hydraulic capacity. Manning's Equation follows:

$$V = 1.49 ( R^{2/3} S^{1/2} ) / n \quad 504-32$$

where:

V	=	mean velocity, ft/s
R	=	hydraulic radius (A / P), ft
A	=	cross sectional area, ft <sup>2</sup>
P	=	wetted perimeter, ft
S	=	the slope of the channel, ft/ft
n	=	Manning's roughness coefficient (see <b>Table 502-1</b> through <b>Table 502-5</b> ).

Discharge (Q) is computed as the product of velocity (V) and area (A), so the equation becomes:

$$Q = 1.49 ( A R^{2/3} S^{1/2} ) / n \quad 504-33$$

where:

Q	=	flow rate, ft <sup>3</sup> /s
A	=	cross sectional area, ft <sup>2</sup>
	=	all other terms previously defined

For circular shaped storm drains flowing full, the hydraulic radius is D/4 and **Equations 504-32** and **504-33** can be written as:

$$V = ( 0.59 / n ) D^{0.67} S_o^{0.5} \quad 504-34$$

(FHWA, August 2013, HEC-22, Eq. 7-1, p. 7-2)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

$$Q = ( 0.46 / n ) D^{2.67} S_o^{0.5} \quad 504-35$$

(FHWA, August 2013, HEC-22, Eq. 7-1, p. 7-2)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

where:

V	=	mean velocity, ft/s
Q	=	flow rate, ft <sup>3</sup> /s
n	=	Manning's roughness coefficient (see <b>Table 502-1</b> and <b>Table 502-4</b> )
D	=	diameter of pipe, ft
S <sub>o</sub>	=	the slope of the energy grade line, ft/ft

There are several nomographs available to solve Manning's Equation for full pipe flow. **Figure 502-3** can be used to solve the equation for pipes flowing partially full. Also, pipe manufacturers' design literature often contains solution nomographs for pipe shapes other than round.

#### 504.5.4 Energy Losses

As described in HEC-22 (FHWA, August 2013), "Urban Design Drainage Manual", prior to computing the hydraulic grade line, all energy losses in pipe runs and junctions must be estimated. In addition to the principal energy involved in overcoming the friction in each conduit run, energy (or head) is required to overcome changes in momentum or turbulence at outlets, inlets, bends, transitions, junctions, and access holes. This section presents relationships for estimating typical energy losses in storm drainage systems.

#### Pipe Friction Losses

The major loss in a storm drainage system is the friction or boundary shear loss. The head loss ( $H_f$ ) due to friction in a pipe is computed as follows:

$$H_f = S_f L \quad 504-36$$

(FHWA, August 2013, HEC-22, Eq. 7-2, p. 7-10)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

where:

$H_f$	=	friction loss, ft
$S_f$	=	friction slope, ft/ft
L	=	length of pipe, ft

The friction slope ( $S_f$ ) in **Equation 504-36** is also the slope of the hydraulic gradient for a particular pipe run. As indicated by **Equation 504-36**, the friction loss is simply the hydraulic gradient multiplied by the length of the run. Pipe friction losses can be determined by combining **Equation 504-36** with Manning's Equation as follows:

$$S_f = \frac{H_f}{L} = \left( \frac{Q n}{0.46 D^{2.67}} \right)^2 \quad 504-37$$

(FHWA, August 2013, HEC-22, Eq. 7-3, p. 7-10)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

where:

all variables previously defined

### Exit Losses

The exit loss ( $H_o$ ) from a storm drain outlet is a function of the change in velocity ( $V_o$ ), at the outlet of the pipe. For a sudden expansion such as at an endwall, the exit loss is:

$$H_o = 1.0 \left( \frac{V_o^2}{2g} - \frac{V_d^2}{2g} \right) \quad 504-38$$

(FHWA, August 2013, HEC-22, Eq. 7-4, p. 7-10)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

where:

$H_o$	=	exit loss, ft
$V_o$	=	average outlet velocity, ft/s
$V_d$	=	channel velocity downstream of outlet, ft/s
$g$	=	gravitational acceleration, 32.2 ft/s <sup>2</sup>

Note that when  $V_d = 0$ , as in a reservoir, the exit loss is the velocity head. For partially full flow where the pipe outlets to a channel with water moving in the same direction as the outlet water, the exit loss may be reduced to virtually zero.

### Bend Losses

The bend loss ( $H_b$ ) coefficient for storm drain design is minor but can be estimated using the following formula:

$$H_b = 0.0033 (\Delta) \left( \frac{V^2}{2g} \right) \quad 504-39$$

(FHWA, August 2013, HEC-22, Eq. 7-5, p. 7-11)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

where:

$H_b$	=	bend loss, ft
$\Delta$	=	angle of curvature, degrees
$H_b$	=	bend loss coefficient
all other variables previously defined		

### Transition Losses

A transition is a location where a conduit or channel changes size. Typically transitions should be avoided and access holes should be used when pipe size increases. However, sometimes transitions are unavoidable. Transitions include expansions, contractions, or both. In small storm



drains, transitions may be confined within access holes. In larger storm drains or when a specific need arises, transitions may occur within pipe runs.

Energy losses in contractions or expansions in non-pressure flow can be expressed in terms of the kinetic energy at the two ends. Contraction ( $H_c$ ) and expansion ( $H_e$ ) losses can be evaluated with **Equations 504-40** and **504-41**, respectively.

$$H_c = K_c \left( \frac{V_2^2}{2g} - \frac{V_1^2}{2g} \right) \quad 504-40$$

(FHWA, August 2013, HEC-22, Eq. 7-6, p. 7-11)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

$$H_e = K_e \left( \frac{V_1^2}{2g} - \frac{V_2^2}{2g} \right) \quad 504-41$$

(FHWA, August 2013, HEC-22, Eq. 7-7, p. 7-11)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

where:

$H_c$	=	contraction loss, ft
$H_e$	=	expansion loss, ft
$K_c$	=	contraction coefficient (0.5 $K_e$ for gradual contractions)
$K_e$	=	expansion coefficient
$V_1$	=	velocity upstream of transition, ft/s
$V_2$	=	velocity downstream of transition, ft/s
$g$	=	acceleration due to gravity, 32.2 ft/s <sup>2</sup>

For gradual contractions, it has been observed that  $K_c = 0.5 K_e$ . Typical values of  $K_c$  for sudden contractions are tabulated in **Table 504-1**. Typical values of  $K_e$  for gradual expansions are tabulated in **Table 504-2**. The angle of the cone that forms the transition is defined in **Figure 504-16**.

**Table 504-1 Typical Values of  $K_c$ , for Sudden Pipe Contractions**

Source: FHWA, August 2013, HEC-22, Table 7-4b, p. 7-12

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

$D_2 / D_1$	$K_c$
0	0.5
0.4	0.4
0.6	0.3
0.8	0.1
1.0	0.0

$D_2 / D_1$  = Ratio of diameter of large pipe to small pipe.

**Table 504-2 Typical Values of  $K_e$  for Gradual Enlargement of Pipes in Non-Pressure Flow**

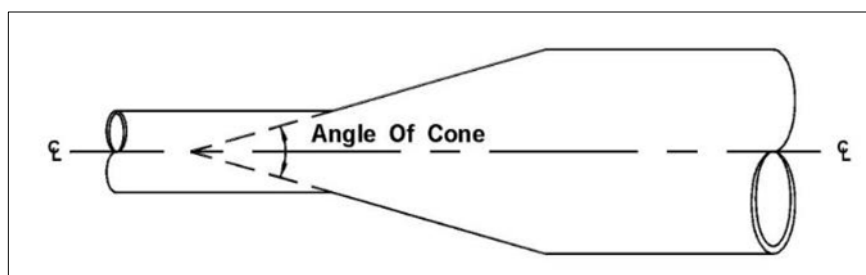
Source: FHWA, August 2013, HEC-22, Table 7-4a, p. 7-12

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

$D_2 / D_1$	Angle of Cone						
	10°	20°	45°	60°	90°	120°	180°
1.5	0.17	0.40	1.06	1.21	1.14	1.07	1.00
3	0.17	0.40	0.86	1.02	1.06	1.04	1.00

 $D_2 / D_1$  = Ratio of diameter of large pipe to small pipe.

Angle of cone is the angle in degrees between the sides of the tapering section.



Source: FHWA, August 2013, HEC-22, Figure 7-3, p. 7-11.

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>**Figure 504-16 Angle of Cone for Pipe Diameter Changes**

For storm drain pipes functioning under pressure flow, the loss coefficients listed in **Table 504-3** and **Table 504-4** can be used with **Equation 504-42** for sudden and gradual expansions, respectively. For sudden contractions in pipes with pressure flow, the loss coefficients listed in **Table 504-5** can be used in conjunction with **Equation 504-43**.

$$H_e = K_e \left( \frac{V_1^2}{2g} \right) \quad 504-42$$

(FHWA, August 2013, HEC-22, Eq. 7-8, p. 7-12)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

$$H_c = K_c \left( \frac{V_2^2}{2g} \right) \quad 504-43$$

(FHWA, August 2013, HEC-22, Eq. 7-9, p. 7-12)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

where:

all variables previously defined

**Table 504-3 Values of  $K_e$  for Determining Loss of Head Due to Sudden Enlargement in Pipes**

Source: FHWA, August 2013, HEC-22, Table 7-4c, p. 7-13.

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

$D_2 / D_1$	Velocity, $V_1$ , in feet per second												
	2	3	4	5	6	7	8	10	12	15	20	30	40
1.2	.11	.10	.10	.10	.10	.10	.10	.09	.09	.09	.09	.09	.08
1.4	.26	.26	.25	.24	.24	.24	.24	.23	.23	.22	.22	.21	.20
1.6	.40	.39	.38	.37	.37	.36	.36	.35	.35	.34	.33	.32	.32
1.8	.51	.49	.48	.47	.47	.46	.46	.45	.44	.43	.42	.41	.40
2.0	.60	.58	.56	.55	.55	.54	.53	.52	.52	.51	.50	.48	.47
2.5	.74	.72	.70	.69	.68	.67	.66	.65	.64	.63	.62	.60	.58
3.0	.83	.80	.78	.77	.76	.75	.74	.73	.72	.70	.69	.67	.65
4.0	.92	.89	.87	.85	.84	.83	.82	.80	.79	.78	.76	.74	.72
5.0	.96	.93	.91	.89	.88	.87	.86	.84	.83	.82	.80	.77	.75
10.0	1.00	.99	.96	.95	.93	.92	.91	.89	.88	.86	.84	.82	.80
$\infty$	1.00	1.00	.98	.96	.95	.94	.93	.91	.90	.88	.86	.83	.81

 $D_2 / D_1$  = ratio of diameter of large pipe to small pipe. $V_1$  = velocity in smaller pipe, ft/sec**Table 504-4 Values of  $K_e$  for Determining Loss of Head Due to Gradual Enlargement in Pipes**

Source: FHWA, August 2013, HEC-22, Table 7-4d, p. 7-13.

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

$D_2 / D_1$	Angle of Cone										
	2°	6°	10°	15°	20°	25°	30°	35°	40°	50°	60°
1.1	0.01	0.01	0.03	0.05	0.10	0.13	0.16	0.18	0.19	0.21	0.23
1.2	0.02	0.02	0.04	0.09	0.16	0.21	0.25	0.29	0.31	0.35	0.37
1.4	0.02	0.03	0.06	0.12	0.23	0.30	0.36	0.41	0.44	0.50	0.53
1.6	0.03	0.04	0.07	0.14	0.26	0.35	0.42	0.47	0.51	0.57	0.61
1.8	0.03	0.04	0.07	0.15	0.28	0.37	0.44	0.50	0.54	0.61	0.65
2.0	0.03	0.04	0.07	0.16	0.29	0.38	0.46	0.52	0.56	0.63	0.68
2.5	0.03	0.04	0.08	0.16	0.30	0.39	0.48	0.54	0.58	0.65	0.70
3.0	0.03	0.04	0.08	0.16	0.31	0.40	0.48	0.55	0.59	0.66	0.71
$\infty$	0.03	0.05	0.08	0.16	0.31	0.40	0.49	0.56	0.60	0.67	0.72

 $D_2 / D_1$  = ratio of diameter of large pipe to small pipe.

**Table 504-5 Values of  $K_c$  for Determining Loss of Head Due to Sudden Contraction**

Source: FHWA, August 2013, HEC-22, Table 7-4e, p. 7-14.

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

$D_2 / D_1$	Velocity, $V_1$ , in feet per second												
	2	3	4	5	6	7	8	10	12	15	20	30	40
1.1	.03	.04	.04	.04	.04	.04	.04	.04	.04	.04	.05	.05	.06
1.2	.07	.07	.07	.07	.07	.07	.07	.08	.08	.08	.09	.10	.11
1.4	.17	.17	.17	.17	.17	.17	.17	.18	.18	.18	.18	.19	.20
1.6	.26	.26	.26	.26	.26	.26	.26	.26	.26	.26	.25	.25	.24
1.8	.34	.34	.34	.34	.34	.34	.33	.33	.32	.32	.32	.29	.27
2.0	.38	.38	.37	.37	.37	.37	.36	.36	.35	.34	.33	.31	.29
2.2	.40	.40	.34	.39	.39	.39	.39	.38	.37	.37	.35	.33	.30
2.5	.42	.42	.42	.41	.41	.41	.40	.40	.39	.38	.37	.34	.31
3.0	.44	.44	.44	.43	.43	.43	.42	.42	.41	.40	.39	.36	.33
4.0	.47	.46	.46	.46	.45	.45	.45	.44	.43	.42	.41	.37	.34
5.0	.48	.48	.47	.47	.47	.46	.46	.45	.45	.44	.42	.38	.35
10.0	.49	.48	.48	.48	.48	.47	.47	.46	.46	.45	.43	.40	.36
$\infty$	.49	.49	.48	.48	.48	.47	.47	.47	.46	.45	.44	.41	.38

 $D_2 / D_1$  = ratio of diameter of large pipe to small pipe. $V_1$  = velocity in smaller pipe, ft/sec**Junction Losses**

A pipe junction is the connection of a lateral pipe to a larger trunk pipe without the use of an access hole structure. The minor loss ( $H_j$ ) equation for a pipe junction is a form of the momentum equation as follows:

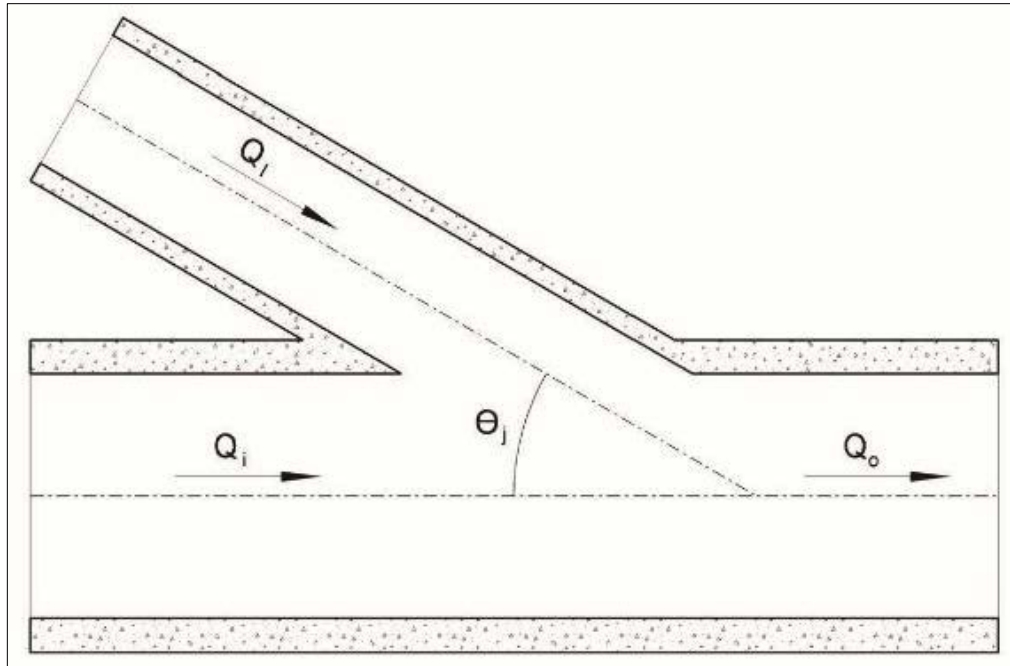
$$H_j = \frac{(Q_o V_o) - (Q_i V_i) - (Q_1 V_1 \cos \theta)}{0.5 g (A_o + A_i)} + h_i - h_o \quad \text{504-44}$$

(FHWA, August 2013, HEC-22, Eq. 7-10, p. 7-15)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

where:

$H_j$	=	junction loss, ft
$Q_o, Q_i, Q_1$	=	outlet, inlet, and lateral flows, respectively, $\text{ft}^3/\text{s}$
$V_o, V_i, V_1$	=	outlet, inlet, and lateral velocities, respectively, $\text{ft}/\text{s}$
$h_o, h_i$	=	outlet and inlet velocity heads, ft
$A_o, A_i$	=	outlet and inlet cross sectional areas, $\text{ft}^2$
$\theta$	=	the angle between the inflow and outflow pipes, degrees (see <b>Figure 504-17</b> )
$g$	=	gravitational acceleration, $32.2 \text{ ft}/\text{s}^2$



Source: FHWA, August 2013, HEC-22, Figure 7-4, p. 7-15.

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

**Figure 504-17 Deflection Angle**

### Access Hole (Manhole) Losses

**Approximate Method** - An approximate method for computing head losses ( $H_{ah}$ ) at access holes (manholes), involves multiplying the velocity head ( $V^2/2g$ ) of the outflow pipe by a coefficient as represented in **Equation 504-45**. Applicable head loss coefficients ( $K_{ah}$ ) are tabulated in **Table 504-6**. This method can be used to estimate the initial pipe crown drop across an access hole or inlet structure to offset energy losses at the structure. The crown drop is then used to establish the appropriate pipe invert elevations. This method is for preliminary or approximate estimation.

$$H_{ah} = K_{ah} \left( \frac{V^2}{2g} \right) \quad 504-45$$

(FHWA, August 2013, HEC-22, Eq. 7-11, p. 7-16)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

where:

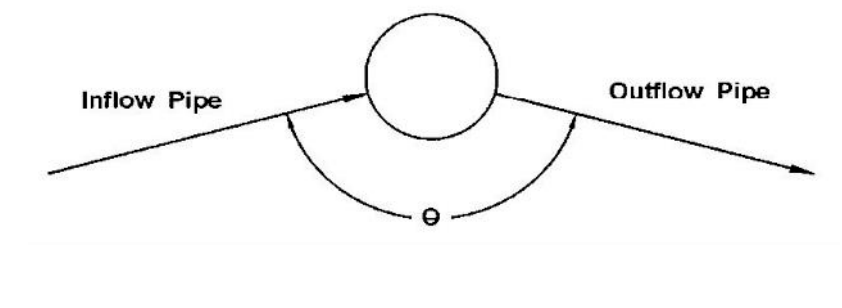
$H_{ah}$  = head loss at access holes (manholes), ft  
 $K_{ah}$  = head loss coefficient at access holes (manholes)  
 all other variables previously defined

**Table 504-6 Access Hole (Manhole) Head Loss Coefficients**

Source: FHWA, August 2013, HEC-22, Table 7-5, p. 7-16.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

Structure Configuration	$K_{ah}$
Inlet – straight run, square edge	0.50
Inlet – angled through 90°	1.50
Manhole – straight run	Min ~ 0.15
Manhole – angled through	
90°	1.00
120°	0.85
135°	0.75
157.5°	0.45



**FHWA Inlet and Access (Manhole) Energy Loss Method.** The following FHWA method for estimating energy losses in manholes and inlets classifies the hydraulic conditions in manholes in a manner analogous to inlet control and full flow for culverts.

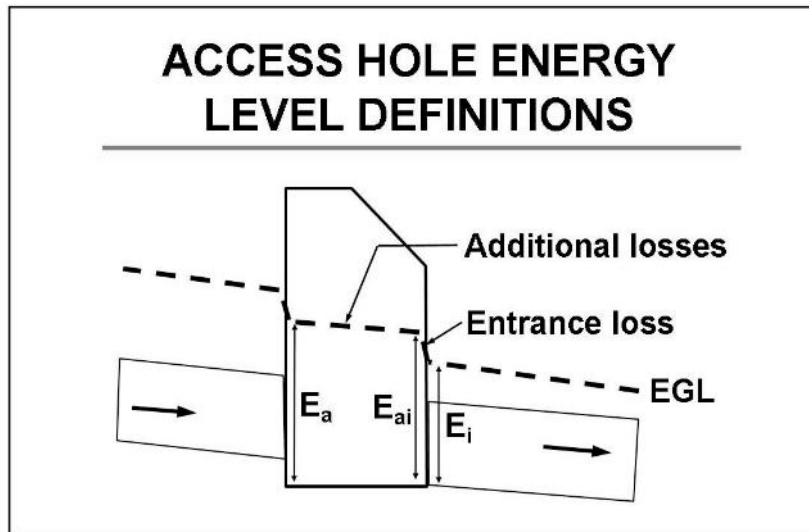
The method follows three fundamental steps (with terms as defined in **Figure 504-18**):

- Step 1 Determine an initial manhole energy level ( $E_i$ ) based on inlet control (weir and orifice) or outlet control (partial and full flow) equations.
- Step 2 Adjust the initial manhole energy level based on benching, inflow angle(s), and plunging flows to compute the final calculated energy level ( $E$ ).
- Step 3 Calculate the exit loss from each inflow pipe and estimate the energy gradeline ( $EGL_o$ ), which will then be used to continue calculations upstream.

Refer to Chapter 7 of HEC-22 (FHWA, August 2013) for detailed procedures of each step.

FHWA, August 2013, "HEC-22, Urban Drainage Manual, Third Edition".

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>



Source: FHWA, August 2013, HEC-22, Figure 7-5, p. 7-17.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

**Figure 504-18 Definition Sketch for FHWA Access Hole (Manhole) Method**

#### 504.5.5 Hydraulic Grade Line Description and Solution Procedure

The hydraulic gradient is the surface or profile of water flowing in an open channel or a pipe flowing partially full. If a pipe is under pressure, the hydraulic grade line is the level water would rise to in a small, vertical tube connected to the pipe. The difference in elevation for the water surfaces in the successive tubes would represent the friction loss for that length of pipe, and the slope of the line between water surfaces would be the friction slope. Therefore, if a pipe run were placed on the calculated friction slope corresponding to a certain flow rate, pipe diameter, and “n” value, the surface of the flow (hydraulic grade line) would be parallel to the top of the pipe and the pipe would not be under pressure. The friction slope can be calculated using **Equation 504-35** solving for  $S_o$ .

If there is a reason to place the pipe run on a slope less than the friction slope, then the hydraulic grade line would be steeper than the pipe slope. Depending on the elevation of the hydraulic grade line at the downstream end of the run in question, it is possible to have the water surface rise to above the top of the pipe which would indicate that the pipe would flow under pressure.

Hydraulic grade line (HGL) and energy grade line (EGL) computations begin at the downstream outlet tailwater elevation and proceed upstream along the entire pipe system. For each pipe segment, friction losses from all sources are computed and applied in the computations to develop HGL and EGL for the segment and for the entire system. The computed HGL elevations at inlets and manholes must be evaluated with respect to **Section 200** (drainage criteria).

#### HGL & EGL Solutions – Computer Programs and Manual Solutions

For most storm drain systems, computer programs provide the most efficient means for computing storm drain the HGL and EGL elevations. However, manual computations in terms of spreadsheet type tables are also acceptable. The tabular procedure explanation is somewhat lengthy and is not presented here. Refer to HEC-22 (FHWA, August 2013) page 7-34 for a

detailed description of the recommended HGL and EGL computation procedure, the required computation tables, and example problems.

FHWA, August 2013, "HEC-22, Urban Drainage Manual, Third Edition".

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

## **504.6 Underdrains**

In certain areas, ground water can be a significant problem as it attacks foundations, substructures, subgrades, and other aspects of highway components. In most soils where ground water is a problem, a system of underdrains, installed for the removal of excess moisture, can be a very useful feature in the overall roadway design. Underdrains may take the form of networks of perforated (or otherwise permeable) pipes, French drains, or collector fields. Where such appurtenances are needed, the additional expense for installation is usually justified in terms of future savings in roadway and structure maintenance costs.

Percolation rates for ground water should be determined by field testing at the underdrain proposed location, or as a last resort, estimated with references to support the estimate. Collector pipe sizes and networks may then be established for the removal of that water. French drains can be useful where the unwanted ground water percolation rates are relatively high. Collector fields may be useful where reasonable outfalls for ground water are not available.

Design of underdrains requires knowledge of subsurface hydrology. An in-depth discussion of subsurface hydrology is beyond the scope of this manual. Refer to the NMDOT Geotechnical Section for more information on the design of underdrains.

## **504.7 References**

FHWA, May 1993, "HEC-21, Design of Bridge Deck Drainage".

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hec/hec21.pdf>

FHWA, August 2013, "HEC-22, Urban Drainage Manual, Third Edition".

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

NMDOT, Website, "Standard Drawings", "Standard Specifications for Highway and Bridge Construction".

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



## 505 Hydraulics at Bridges

### 505.1 Introduction

Bridges are structures that transport vehicular traffic over waterways or other obstructions, with openings of 20 feet or more. Bridges should be designed for: the minimum cost subject to criteria, the desired level of hydraulic performance up to an acceptable risk level, and mitigation of impacts to the stream environment.

### 505.2 Information Requirements

The basic information required to analyze bridges follows:

- Design Flood and Check Flood hydrographs
- Peak flood stage information, if available
- Watercourse, bridge, and roadway topographic survey (see **APPENDIX 3** Survey Checklist)
- Roughness coefficients for the channel bed, banks and overbanks
- Sediment samples required to perform gradation analyses of bed, and bank material
- Record drawings and/or proposed plans

### 505.3 Hydraulic Design Procedure Outline

The hydraulic design for a bridge at a stream crossing requires a comprehensive engineering approach that includes data collection, hydraulic analysis, formulation of alternatives, and final alternative selection. The hydraulic analyses must also include an erosion and sedimentation analysis for the watercourse upstream and downstream of the bridge, and must also include an evaluation of contraction scour, abutment and pier scour, as well as total scour.

For erosion and scour analysis methods, refer to **Section 600**. Each project and watercourse will have unique characteristics. The outline provided below lists the general information and analyses that may be required.

#### I. Field Reconnaissance and Data Collection

##### A. Required field information

(Refer to **APPENDIX 3** for Field Observation Checklists)

##### B. Studies by other agencies

1. Federal flood insurance studies (FEMA)
2. Federal floodplain studies by the USACE, NRCS, etc.
3. State and local agency floodplain studies
4. Existing bridge analyses or hydraulic performance studies

##### C. Influences on the site hydraulic performance

1. Other streams, reservoirs, water intakes, diversions, etc.
2. Structures upstream or downstream
3. Natural features of stream and floodplain
4. Channel modifications upstream or downstream
5. Floodplain encroachments
6. Sediment grain size distributions, bed armoring/stone sizes, and bed forms

- D. Environmental impact
  - 1. Existing bed or bank stability
  - 2. Floodplain land use and flow distribution
  - 3. Environmentally sensitive areas (fisheries, wetlands, streams, rivers, etc.)
- E. Site-specific design criteria
  - 1. Preliminary risk assessment
  - 2. Application of NMDOT or other agency criteria
- II. Hydrologic Analysis
  - A. Watershed morphology
    - 1. Drainage area
    - 2. Watershed and stream slope
    - 3. Channel geometry
    - 4. Soil types and associated rainfall loss rates
  - B. Hydrologic computations
    - 1. Discharge calibration based on high water marks/flood return periods
    - 2. Hydrographs for specified storm frequencies
- III. Hydraulic Analysis
  - A. Computer model calibration and verification
  - B. Hydraulic performance for existing conditions
  - C. Hydraulic performance of proposed designs
- IV. Selection of Final Design
  - A. Risk assessment/least-cost alternative (if appropriate)
  - B. Measure of compliance with established hydraulic criteria
  - C. Consideration of environmental and social criteria
  - D. Design details such as riprap, scour abatement, river training, etc.
- V. Documentation
  - A. Complete project records, permit applications, etc.
  - B. Complete correspondence and reports

### 505.3.1 Hydraulic Analyses and Performance

A bridge may have one or more openings, with one or more piers. The flood water surface may flow as free-surface flow (non-pressure) or as pressure flow. In pressure flow, the flood water may overtop the bridge or road embankment, and therefore, weir flow may occur above the roadway or bridge deck. Water surface analyses are complex. They must include computation of the entire watercourse profile, all energy losses, determination of pressure flow (if applicable), and/or weir flow (if applicable). Therefore, watercourse and bridge hydraulic analyses shall be analyzed using a computer program such as HEC-RAS. Most watercourses and bridges may be simulated with a one-dimensional model; however, more complex or dispersed flow situations may require a two-dimensional model. Subsequent sections discuss computer models in further detail.

### Water Surface Profiles

Various water surface profiles are possible near a bridge. Refer to **Figure 505-1** and **Figure 505-2** for schematics that illustrate flow types and hydraulic variables. Refer to the following definitions related to these figures:

- **Backwater** ( $h_1$ ) is measured relative to the normal water surface elevation without the effect of the bridge at the approach cross section (Section 1). It is the result of contraction, re-expansion, and bridge pier head losses. Backwater can also be the result of a "choking condition" in which critical depth is forced to occur in the contracted opening with a resultant increase in depth and specific energy upstream of the contraction. This is illustrated in **Figure 505-1**.
- **Type I** flow consists of subcritical flow throughout the approach, bridge, and exit cross sections and is the most common condition encountered in practice.
- **Type IIA and IIB** both represent subcritical approach flows which have been choked by the contraction resulting in the occurrence of critical depth in the bridge opening. In Type IIA, the critical water surface elevation in the bridge opening is lower than the undisturbed normal water surface elevation. In Type IIB, the critical water surface elevations are higher than the normal water surface elevation and a weak hydraulic jump immediately downstream of the bridge contraction is possible.
- **Type III** flow is supercritical approach flow and remains supercritical through the bridge contraction. Such a flow condition is not subject to backwater unless it chokes and forces the occurrence of a hydraulic jump upstream of the contraction.

### Upstream Contraction, Downstream Expansion and Ineffective Flow

#### Upstream

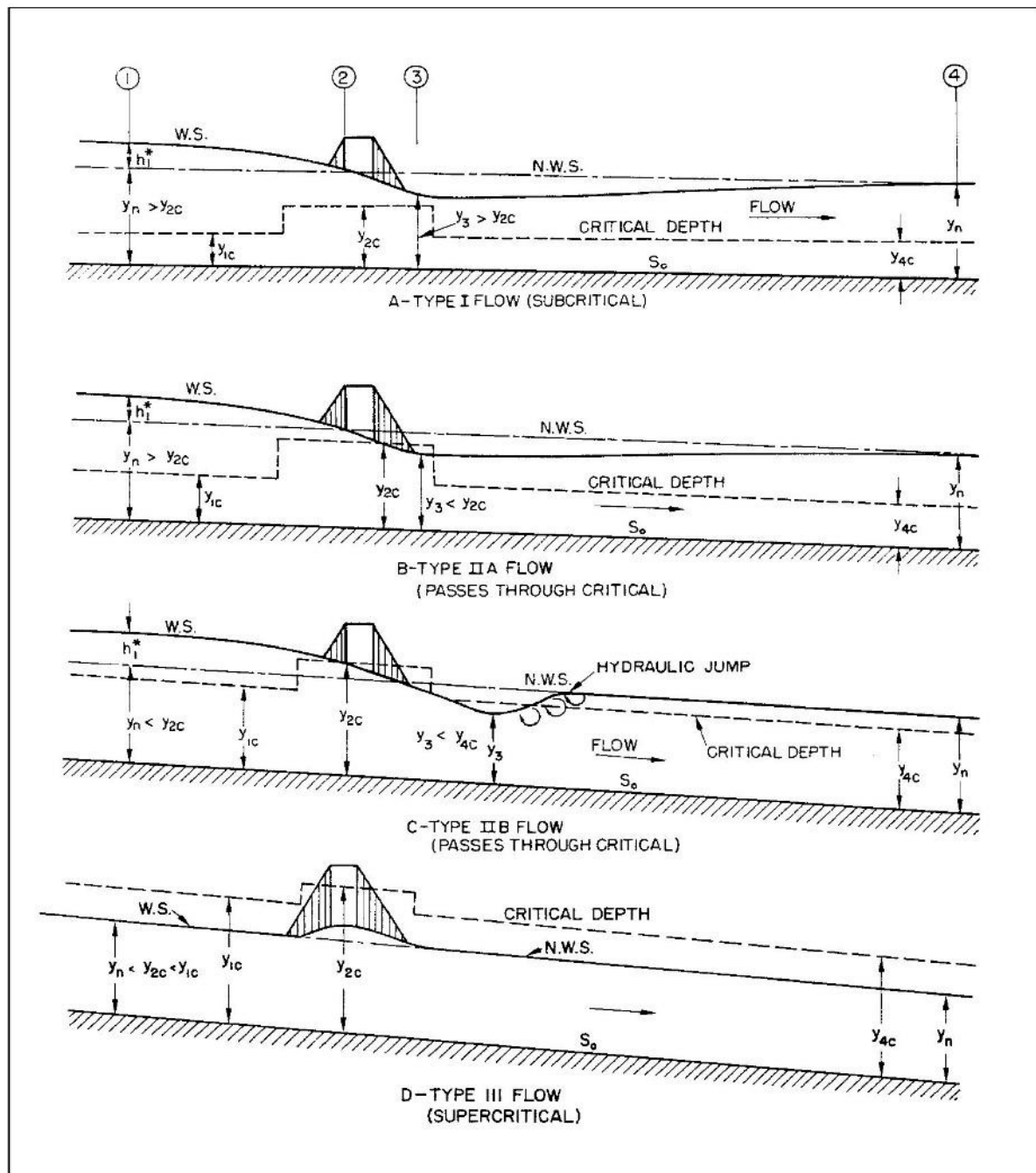
For many watercourses, the flow width for infrequent storms, such as the 50-year or 100-year storms, may extend well past a bridge opening width or the "effective" bridge width for conveyance. Upstream of a bridge for some distance, the full cross section width may be effective in flow conveyance. However, unless the watercourse banks transition gradually into a bridge opening, the flow outside of the effective bridge width or bank width will be ineffective in flow conveyance and must be excluded (simulated as ineffective flow) from the hydraulic computations for at least one or more cross sections. A contraction coefficient is required to account for the head loss due to flow contraction. This coefficient should be applied to one or more cross sections upstream of a bridge. Refer to the HEC-RAS User's Manual for further guidance on cross section locations near a bridge and other modeling guidelines.

#### Downstream

Flow will generally expand as it exits the bridge opening and therefore, ineffective flow must be simulated for one or more sections, and an expansion head loss coefficient must be applied to one or more cross sections downstream, until the first full flow section.

### Bridge Piers

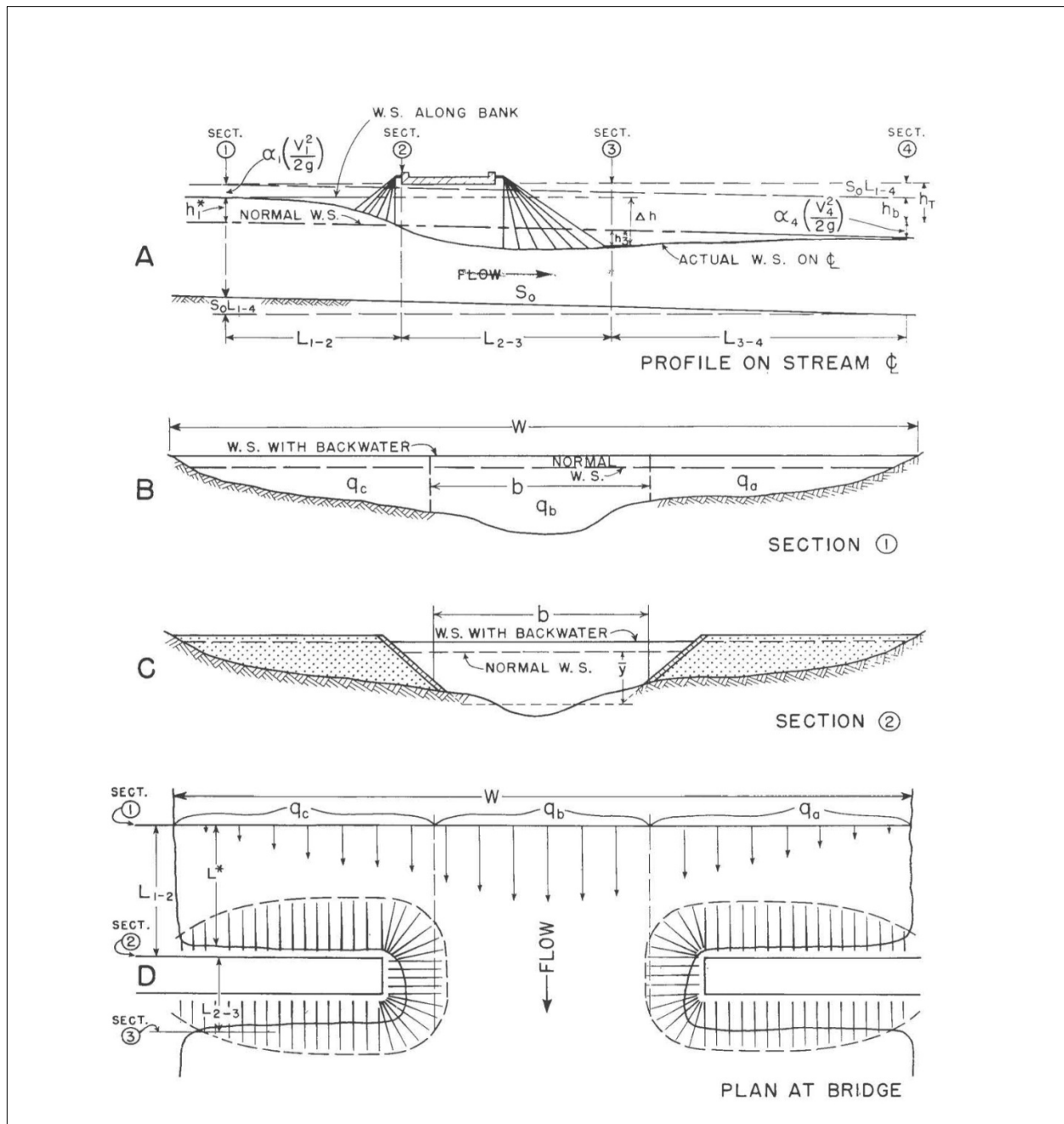
Bridge piers may collect debris and cause head loss, and therefore, must be simulated to predict the water surface profile. Pier debris width and height may be simulated with HEC-RAS. The debris assumption must be based on the watercourse vegetation type and upstream watershed vegetation type. Observations of other bridges on the watercourse or adjacent and similar watercourses may provide guidance as to the pier debris that may be anticipated. In addition, the Patrol Foreman may provide historical information.



Source: Source: FHWA, March 1978, (document dated 1960), HDS-1, Figure 2, p. 3.

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hds1.pdf>

**Figure 505-1 Sketch Illustrating positions of Cross Sections 1 through 4 in HDS 1 Backwater Method (FHWA 1978, 1960)**



Source: FHWA, April 2012, HDS-7, Figure 5.1, p. 5.2.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12018.pdf>

**Figure 505-2 Types of Flow Encountered**

### 505.3.2 Analytical Methods

#### One-Dimensional Modeling

One-dimensional modeling requires that variables (velocity, depth, etc.) change predominantly in one defined direction along the channel, typically along the channel centerline. One-dimensional modeling programs are based on the standard step method for water surface profile computations. The HEC-RAS program provides a one-dimensional analysis that uses a variation of the momentum method in the special bridge routine to determine flow conditions in the presence of bridge piers. The momentum equation between cross sections 1 and 3 is used to detect Type II flow and solve for the upstream depth. In this case, the critical depth is in the bridge contraction. Refer to **Section 505.3.1** for more information.

Many other one-dimensional computer models have been developed to compute water surface profiles. The USGS and the Federal Highway Administration developed WSPRO (developed in 1998) that had components specifically formulated for the analysis of flow through bridge openings. HEC-RAS has incorporated features from the WSPRO bridge routine and has incorporated features from HY-8 for inlet control analyses.

In practice, most watercourse analyses are performed using one-dimensional simulation methods available in HEC-RAS. In general, one-dimensional methods provide reasonable results where lateral velocities are small.

#### Two-Dimensional Modeling

Two-dimensional models compute the horizontal velocity components ( $V_x$  and  $V_y$ ) or, alternatively, velocity vector magnitude and direction throughout the model domain. These models provide detailed determination of the cross-stream water surface elevations, flow velocities, and/or flow distribution.

Two-dimensional models are more complex and require more time to prepare and calibrate. They require essentially the same field data as a one-dimensional model and, depending on complexity, may require more computer run time. Where the flow is essentially two-dimensional in the horizontal plane, a one-dimensional analysis may lead to costly over-design or possibly improper design of hydraulic structures and improvements.

The USACE has updated HEC-RAS to include two-dimensional (2D) open channel flow analyses. The USGS has developed a two-dimensional finite element model for the FHWA (1989) called the Finite Element Surface-Water Modeling System (FESWMS-2DH). This model has been developed to analyze flow at bridge crossings where complicated hydraulic conditions exist. This two-dimensional modeling system is flexible and may be applied to many types of steady and unsteady flow problems including multiple opening bridge crossings, spur dikes, floodplain encroachments, multiple channels, flow around islands, and flow in estuaries.

#### FHWA Current Two-Dimensional Model Preference

The FHWA has now adopted another model developed by the U.S. Bureau of Reclamation (USBR) instead of the previous FESWMS-2DH model. The USBR has developed a two-dimensional model called SRH-2D as presented here (including previous versions and associated capabilities).

SRH-2D Sedimentation and River Hydraulics, is a two-dimensional (2D) hydraulic, sediment, temperature, and vegetation model for river systems under development at the Bureau of Reclamation. Different versions of SRH-2D contain different modules, as listed below:

Note – At the time of this Drainage Design Manual preparation, SRH-2D Version 3.2.0 (March 2017) is the latest version.

Previous versions with additional capabilities are listed here.

SRH-W version 1.1: Watershed Modeling

SRH-2D version 2: Modeling of Flow Hydraulics for Stream/River Systems

SRH-2D version 3: Mobile Bed Sediment Transport Module Added to v.2

SRH-2D version 4: Temperature and Vegetation Module Added to version v.2

Refer to the following USBR Website for more information and program downloads.

<https://www.usbr.gov/tsc/techreferences/computer%20software/models/srh2d/index.html>

#### NMDOT Two-Dimensional Software Preference

Before conducting two-dimensional modeling, obtain direction from the NMDOT Drainage Design Bureau regarding preferred, acceptable or mandated software.

### **Physical Modeling**

Complex hydrodynamic situations defy accurate or practical mathematical modeling. Physical models should be considered when hydraulic performance data is needed that cannot be reliably obtained from mathematical modeling, when risk of failure or excessive over-design is unacceptable, and when research is needed. The constraints on physical modeling are the size (scale), cost, and time required to design, construct, perform, and evaluate the results of the physical models.

## **505.4 Additional Design Considerations**

Any stream is a dynamic natural system which, as a result of the encroachment caused by elements of a stream-crossing system, will respond in a way that may well challenge even an experienced hydraulic engineer. The complexities of the stream response to encroachment demand that hydraulic engineers must be involved from the project inception in the choice of alternative stream crossing locations. At least some of the members of the engineering design team must have extensive experience in the hydraulic design of stream-crossing systems. Hydraulic engineers should also be involved in the solution of stream stability problems at existing structures.

This section qualitatively discusses some of the design issues which contribute to the overall complexity of spanning a stream with a stream-crossing system. A much more thorough discussion of design philosophy and design considerations may be found in “Highways in the River Environment” (FHWA, February 1990).

### **Watershed Land Use Changes**

Land development and human activities in the watershed will change the watershed hydrology in terms of runoff frequency, runoff volume, runoff duration, peak discharge, and sediment load.

Therefore, the watershed condition, land development, and existing uses or anticipated uses (within a reasonable future time period) must be considered.

### **Location of Stream Crossing**

Although many factors, technical and nontechnical, enter into the final location of a stream-crossing system, the hydraulics of the proposed location must have a high priority. Hydraulic considerations in selecting the location include floodplain width and roughness, flow distribution and direction, stream type (braided, straight, or meandering), stream regime (aggrading, degrading, or equilibrium), and stream controls. The hydraulics of a proposed location also affects environmental considerations such as aquatic life, wetlands, sedimentation, and stream stability.

### **Coordination, Permits, and Approvals**

The interests of other government agencies must be considered in the evaluation of a proposed stream-crossing system, and cooperation and coordination with these agencies, especially water resources planning agencies, must be undertaken. Coordination with FEMA is required when, a:

- Proposed crossing encroaches on a regulatory floodway and would require an amendment to the floodway map
- Proposed crossing encroaches on a floodplain where a detailed study has been performed but no floodway has been designated and the allowable increase in the base flood would be exceeded
- The community is expected to enter into the National Flood Insurance Program within a reasonable period and detailed floodplain studies are underway
- The community is participating in the emergency program and the base flood elevation in the vicinity of insurable buildings is increased by more than the allowable limit

Whenever practical, the stream-crossing system should avoid encroachment on the floodway within a floodplain. When this is not feasible, modification of the floodway itself must be considered.

Design engineers of stream-crossing systems must be cognizant of the relevant local, State, and Federal laws and permit requirements. Federal permits are required for construction of bridges over navigable waters. Permits for other construction activities in navigable waters are under the jurisdiction of the USACE. Applications for Federal permits may require environmental impact assessments under the National Environmental Policy Act (NEPA) of 1969. Refer to **Section 700** for a list of regulatory agencies and permits that should be reviewed for design projects.

### **Environmental Considerations**

Environmental criteria which must be met in the design of stream-crossing systems include the preservation of wetlands and protection of the surrounding habitat. Such considerations often require the expertise of a biologist on the design team. Water quality considerations shall also be included in the design process insofar as the stream-crossing system affects water quality relative to beneficial uses. As a practical matter with bridges, the hydraulic design criteria related to scour, degradation, aggradation, flow velocities, and lateral distribution of flow are important criteria for the evaluation of environmental impacts as well as the safety of the stream-crossing structures. Refer to **Section 700** for more information.



## Stream Morphology

The form and shape of the stream path created by its erosion and deposition characteristics comprise its morphology. A stream can be braided, straight, or meandering, or it can be in the process of changing from one form to another as a result of natural or man-made influences. A historical study of the stream morphology at a proposed stream-crossing site is mandatory. This study shall also include an assessment of any long-term trends in aggradation or degradation. Braided streams and alluvial fans should especially be avoided for stream crossing sites whenever possible.

## Surveys

The purpose of surveys is to gather all necessary site information. This should include such information as topography and other physical features, land use, utilities, flood data, basin characteristics, precipitation data, historical high-water marks, existing structures, channel characteristics, and environmental data. The topographic and physically related survey information should be incorporated into the site plan to provide a comprehensive plan to assist in the analyses and design.

## Risk Evaluation

The evaluation of the risk associated with the probability of flooding at a stream-crossing system is a tool by which site-specific design criteria can be developed. This evaluation considers capital cost, traffic service, environmental and property impacts, and hazards to human life.

The evaluation of risk is a two-stage process. The initial step, identified as risk assessment, is more qualitative than a risk analysis and serves to identify threshold values that must be met by the hydraulic design.

In many cases where the risks are low and/or threshold design values can be met, it is unnecessary to pursue a detailed economic analysis. In those cases where the risk is high and/or threshold values cannot be met, a Least Total Expected Cost (LTEC) analysis should be considered. Refer to **Section 408** for further information on risk assessment.

## Scour

Scour at bridges poses an extreme safety hazard due to potential failure. Therefore, bridge scour analyses must be completed. Refer to **Section 607** for scour analysis discussion and computation methods.

## Preventive/Protection Measures

Based on an assessment of potential scour provided by the hydraulic engineer, the structural engineers can incorporate design features that will prevent or mitigate scour damage at piers. In general, circular piers or elongated piers with circular noses and an alignment parallel to the flow direction are possible alternatives. Spread footings should be avoided and used only where the stream bed is extremely stable below the footing and where the spread footing is located at a depth below the maximum scour. Drilled shafts or drilled piers are possible where pilings cannot be driven. Protection against general stream bed degradation can be provided by drop structures or grade-control structures in, or downstream of the bridge.

Rock riprap is often used, where stone of sufficient size is available, to armor abutment fill slopes and the area around the base of piers. For bridge scour countermeasures, refer to HEC-23 (FHWA, September 2009).

FHWA, September 2009, “HEC-23, Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance, Third Edition, Volume 1”.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/09111/09111.pdf>

Whenever possible, vegetation removal upstream and downstream of the bridge location should be avoided. Embankment overtopping may be incorporated into the design but should be located well away from the bridge abutments and superstructure. Spur dikes are recommended to align the approach flow with the bridge opening and to prevent scour around the abutments. Spur dikes, embankments, and abutments shall be protected by dumped or wire-enclosed rock riprap with a gravel filter blanket, or other revetments approved by the NMDOT.

### **Waterway Enlargement**

There are situations where roadway and structural constraints dictate the vertical positioning of a bridge and result in a small vertical clearance between the low chord and the ground. Significant increases in span length provide small increases in effective waterway opening in these cases.

It is possible to increase the effective area by excavating a flood channel through the reach affecting the hydraulic performance of the bridge. There are, however, several factors that must be accommodated when this action is taken including:

- The flood channel must extend far enough upstream and downstream of the bridge to establish the desired flow regime through the affected reach
- The flood channel must be stabilized to prevent erosion and scour

### **Auxiliary Openings**

The need for auxiliary waterway openings, or relief openings as they are commonly termed, arises on streams with wide floodplains. The purpose of openings located on the floodplain is to provide additional capacity in excess of the principal waterway opening when the stream reaches a high stage.

The basic objectives in choosing the location of auxiliary openings include:

- Maintenance of flow distribution and flow patterns
- Accommodation of relatively large flow concentrations on the floodplain
- Avoidance of floodplain flow along the roadway embankment for long distances
- Avoidance of existing or future non-compatible development
- Crossing of significant tributary channels

The technological weakness in modeling auxiliary openings is in the use of one-dimensional models to analyze two-dimensional flow. The development of two-dimensional models is a major step toward more adequate analysis of complex stream-crossing systems.

The most complex factor in designing auxiliary openings is determining the division of flow between two or more structures. If incorrectly proportioned, one or more of the structures may be overtaxed during a flood event. The design of auxiliary openings should evaluate that potential.

## 505.5 References

FHWA, March 1978 (document dated 1960) "HDS-1, Hydraulics of Bridge Waterways".

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hds1.pdf>

FHWA., February 1990, "Highways in the River Environment".

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hire1990.pdf>

FHWA, September 2009, "HEC-23, Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance, Third Edition, Volume 1".

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FHWA, April 2012, "HDS-7, Hydraulic Design of Safe Bridges".

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12018.pdf>

FHWA, July 28, 2016, "HY-8 Version 7.50 – Culvert Hydraulic Analysis Program".

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USACE, Current Program Download, "HEC-RAS River Analysis System".

<http://www.hec.usace.army.mil/software/hec-ras/downloads.aspx>

USACE, Current Download, "HEC-RAS River Analysis System, Users Manual".

<http://www.hec.usace.army.mil/software/hec-ras/documentation.aspx>

USACE, Current Download, "HEC-RAS River Analysis System, Hydraulic Reference Manual".

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<https://www.usbr.gov/tsc/techreferences/computer%20software/models/srh2d/index.html>

USGS, 1989, FESWMS-2DH Software

<http://water.usgs.gov/software/FESWMS-2DH/>

USGS, Current Program, "Water-Surface Profile Computations (WSPRO)".

<http://water.usgs.gov/software/WSPRO/>

USGS, "User's Manual for Water-Surface Profile Computations (WSPRO)".

<http://mctrans.ce.ufl.edu/mct/>

## 506 Stormwater Storage Facilities

### 506.1 General

Development in watersheds increases impervious area which results in increased stormwater runoff rates and volumes as compared to pre-developed or undeveloped conditions. When downstream stormwater infrastructure has limited capacity, or when required by a governmental entity, stormwater storage facilities (ponds) may be required to reduce peak discharge rates. Stormwater storage ponds may either be designed as detention or retention. Detention ponds may reduce the developed condition peak runoff rate to match the pre-developed peak runoff rate or less. Retention ponds retain storm runoff volumes as required in MS4 areas.

### 506.2 Definitions

The terms detention pond and retention pond are sometimes used interchangeably and incorrectly. The following definitions for each pond type will be used in this manual.

Detention Pond – A stormwater storage facility, where stormwater is detained and released by either a controlled or uncontrolled outlet. A controlled outlet has a gate, valve or other means to control the outlet discharge. Uncontrolled outlets have no means of controlling the outlet discharge except for the inherent structure hydraulic capacity. Uncontrolled outlets are required for NMDOT ponds. Detention ponds attenuate the inflow hydrograph such that the outflow hydrograph has a reduced peak discharge and longer duration.

Retention Pond – A stormwater storage facility, where stormwater is retained because the pond does not contain a positive outlet, or the pond may have a regulated or controlled outlet. All or a portion of the inflow hydrograph is retained for a prolonged period. Water loss over time may occur through evaporation and infiltration. Infiltration basins are a type of retention pond.

The engineer should be aware that water rights issues could arise when water is either detained or retained for greater than 96 hours. Refer to Chapter 26 of “Title 19 Natural Resources and Wildlife” (State Statute – Surface Water Rights) from the New Mexico Office of the State Engineer (NMOSE) (January 31, 2005).

NMOSE, January 31, 2005, “Title 19 Natural Resources and Wildlife, Chapter 26, Surface Water, Part 2 Administrations”. (State Statute - Surface Water Rights).

<http://164.64.110.239/nmac/parts/title19/19.026.0002.htm>

### Small and Large Detention Facilities

Stormwater storage ponds can range in size from small parking lot storage areas to large ponds that may require land acquisition. The NMDOT will normally avoid design of large detention ponds that could be considered jurisdictional dams. Refer to the following definitions for jurisdictional dams and non-jurisdictional dams that are from the NMOSE Dam Safety Bureau document as follows.

NMOSE, December 31, 2010, “Rules and Regulations Governing Dam Design, Construction and Dam Safety”, New Mexico Office of the State Engineer Dam Safety Bureau.

<http://www.ose.state.nm.us/DS/Regs/19-25-12-NMAC-2010.pdf>

**Jurisdictional Dam** – A dam that is 25 feet or greater in height which impounds more than 15 acre-feet of water, or a dam that impounds 50 acre-feet or more of water and is 6 feet or greater in height. Dam height is measured from the lowest point on the downstream toe of dam to the lowest dam crest elevation.

**Non-Jurisdictional Dam** – A dam not meeting the height and storage requirements of a jurisdictional dam unless the dam is unsafe and there is a threat to life or property, as determined by the State Engineer. Waters impounded by a non-jurisdictional dam may not be exempt from water right permit requirements; therefore, a separate State Engineer water right permit for the water impounded by the reservoir created by a non-jurisdictional dam may be required. Non-jurisdictional dams have other requirements (refer to NMOSE, December 31, 2010) unless otherwise exempt. The structures below are considered non-jurisdictional dams.

**Levee or Diversion Dike** – A structure where water flows parallel to the length of the levee or diversion dike as determined by the state engineer.

**Roadway Embankment** – A structure designed across a watercourse for the sole purpose of supporting a roadbed or other means of conveyance for transportation as determined by the State Engineer; where the area upstream has not been enlarged to increase flood storage; and where the embankment is provided with an uncontrolled conduit of sufficient capacity to satisfy requirements of the appropriate state or local transportation authority. If no transportation authority has jurisdiction over the structure, the current NMDOT drainage design criteria shall apply.

If a proposed pond volume and embankment height are near the jurisdictional criteria, coordinate with the Office of the State Engineer Dam Safety Bureau (NMOSE) to ensure the facility is non-jurisdictional prior to design.

### **506.3 Information Required**

The basic information and steps required to simulate and design a pond follow:

- Design Flood and Check Flood hydrographs
- Contour map - required to develop the pond and embankment grading plan and to develop the corresponding elevation-storage data. The map must also identify all adjacent properties and ownership, right-of-ways, easements, infrastructure, utilities, etc.
- Sediment load volume – assume/compute/assign a sediment bulking factor to design for the sediment load volume within the clear water volume (refer to **Section 207** and **Section 402.11**).
- Dead storage volume is required (see **Table 207-1**) in the pond. Elevation-storage data must provide for the inflow sediment volume. The dead storage volume provides a factor of safety to ensure the reservoir will function properly based on the clear water hydrograph. The goal is to maintain the elevation-storage-discharge data to ensure the reservoir will function as simulated in the reservoir routing.
- Water storage limits and/or embankment height limits and/or dam regulations
- Downstream channel or drainage infrastructure type and capacity limitations
- Geologic and soils data for the proposed reservoir and embankment locations
- Geotechnical engineering recommendations
- Design stormwater quality features in the pond at the principal spillway to improve the outfall hydrograph stormwater quality

## 506.4 Reservoir Routing and Pond Sizing

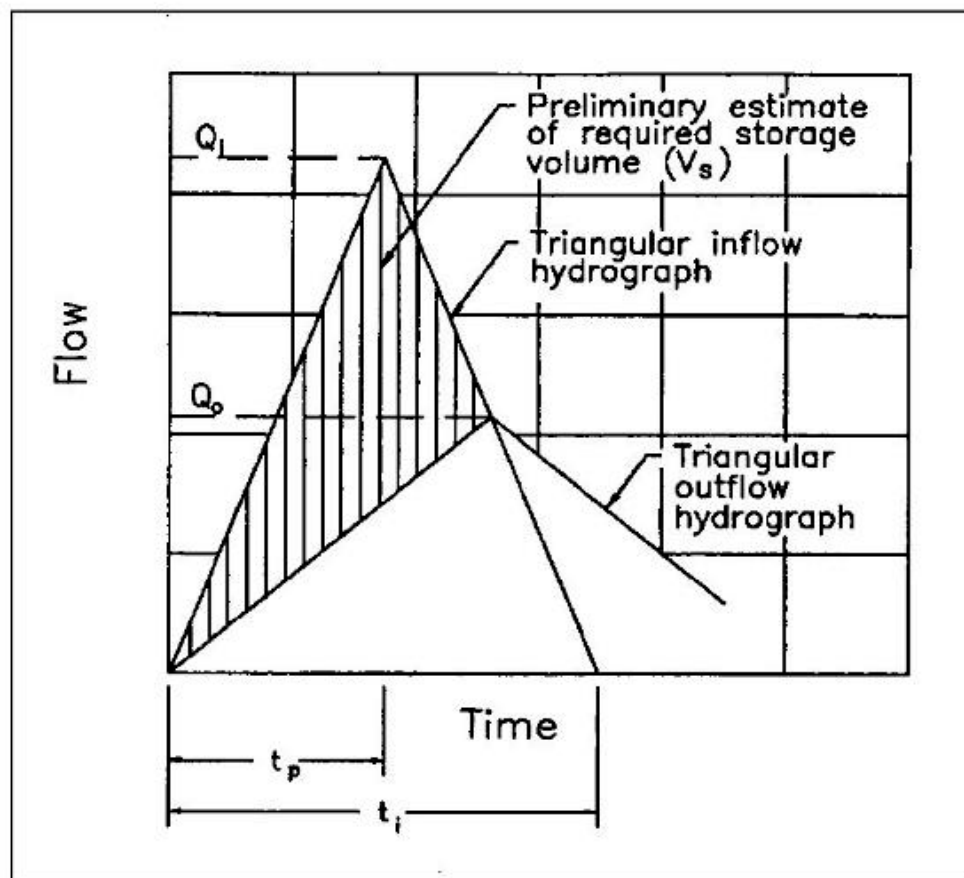
Several methods are available for reservoir routing and sizing, or also called “detention pond” routing and sizing. There are preliminary methods and detailed methods.

### Preliminary Pond Sizing and Routing Methods

#### FHWA Preliminary Triangular Hydrograph Storage Volume Method for Small Watersheds

The HEC-22 manual (FHWA, August 2013) presents a simple triangular hydrograph method to assist in a preliminary storage volume estimation. This method shall only be applied to small watersheds less 150 acres.

This method provides a preliminary estimate of the storage volume required for peak flow attenuation. The inflow and outflow hydrographs are assumed to be triangular shapes as shown in **Figure 506-1**.



Source: FHWA, August 2013, HEC-22, Figure 8-4, p. 8-8.

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

**Figure 506-1 Triangular-Shaped Hydrographs (for Preliminary Estimate of Required Storage Volume)**

The required storage volume ( $V_S$ ) may be estimated from the area above the outflow hydrograph and inside the inflow hydrograph, expressed as:

$$V_S = 0.5 \, t_i (Q_i - Q_o) \quad \text{506-1}$$

(FHWA, August 2013, HEC-22, Eq. 8-1, p. 8-7)

where:

$V_S$	=	storage volume estimate, ft <sup>3</sup>
$Q_i$	=	peak inflow rate, ft <sup>3</sup> /s
$Q_o$	=	peak outflow rate, ft <sup>3</sup> /s
$t_i$	=	duration of basin inflow, s ( <b>Figure 506-1</b> )

Refer to p. 8-7 of HEC-22 (FHWA, August 2013) for additional information. This method will not be acceptable for final routing or pond design.

FHWA, August 2013, "HEC-22, Urban Drainage Manual, Third Edition".

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

#### NRCS Preliminary Storage Volume Method for Small Watersheds

The NRCS has developed a preliminary method to size a pond. This method shall only be applied to small watersheds less than 150 acres. This method will not be acceptable for final routing or pond design and is not presented here. For additional information on this preliminary method refer to:

NRCS, June 1986, "TR-55, Urban Hydrology for Small Watersheds".

[http://www.nrcs.usda.gov/Internet/FSE\\_DOCUMENTS/stelprdb1044171.pdf](http://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/stelprdb1044171.pdf)

### **Detailed Pond Sizing and Routing Methods**

Detention pond routing with detailed data will be required for final routing and pond design. Pond routing simulation with a computer program is required for NMDOT pond design. The standard program required for NMDOT projects (see **Section 405**) follows.

#### HEC-HMS, Hydrologic Modeling System

USACE, Current Program Download, "HEC-HMS Hydrologic Modeling System".

<http://www.hec.usace.army.mil/software/hec-hms/downloads.aspx>

USACE, Current Download, "HEC-HMS Hydrologic Modeling System, User's Manual".

<http://www.hec.usace.army.mil/software/hec-hms/documentation.aspx>

The reservoir routing routines contained within this computer program should be used for sizing detention facilities to ensure they will meet criteria presented in **Section 207**. Refer to the references above for more information.

### **506.5 Elevation – Storage – Discharge Data**

Pond routing will require an elevation–storage–discharge data set that defines the relationship between the depth of water (elevation), the storage volume, and the discharge. For each

elevation, incremental and cumulative storage volume values as well as a discharge value are required. The word “stage” may also be utilized to describe depth or elevation.

Incremental storage may be computed by the average area method. The incremental storage volume is calculated as average of two areas at corresponding pond elevations, multiplied by the depth between the elevations. Cumulative storage is computed as the summation of incremental storage volumes at each elevation.

A typical storage facility has two spillways: principal and emergency (sometimes called auxiliary spillway). The principal spillway capacity is usually designed to convey the Design Flood without allowing flow to enter the emergency spillway refer to **Table 207-1**. The principal spillway or outlet may be a pipe culvert, weir, or other appropriate outlet.

The emergency spillway is sized to protect the pond embankment by providing an outlet for floodwater that exceeds the Design Flood and associated criteria. The emergency spillway peak discharge and outfall location must be designed to consider the potential threat to downstream life and property and must also consider sudden embankment failure due to pond embankment overtopping.

The discharge data must represent the summation of the principal and emergency spillway discharges at each elevation. Appendix 7 contains **Example Problem 7-3** that presents an elevation-storage-discharge data table that illustrates the pond routing data required. In addition, **Example Problem 7-3** contains an example of a reservoir routing summary table that should be submitted with the hydrologic model output. The example problem summary table provides valuable routing results including freeboard with respect to the emergency spillway and to the top of embankment.

## 506.6 Outlet Hydraulics

Weirs, orifices, spillways, and culverts are typically the type of hydraulic structures which may be used as outlets for the stormwater storage facilities. Discharge equations for the weirs, orifices, and spillways are presented in **Section 502.7**. Culvert operation and discharge equations are presented in **Section 503**. Stage-discharge data can be developed using these equations, and may be applied to prepare a graph. If the principal spillway outlet is a pipe, culvert pipe, or box conduit, the discharge rating curve shall be computed with culvert hydraulics and or culvert nomographs for either inlet or outlet control. The tailwater conditions for each water surface elevation in the pond must be considered and documented.

The principal spillway and the emergency spillway each have a discharge data set. This data will be combined for each elevation to develop one elevation-discharge data set for the reservoir routing.

### 506.6.1 Perforated Riser Pipes

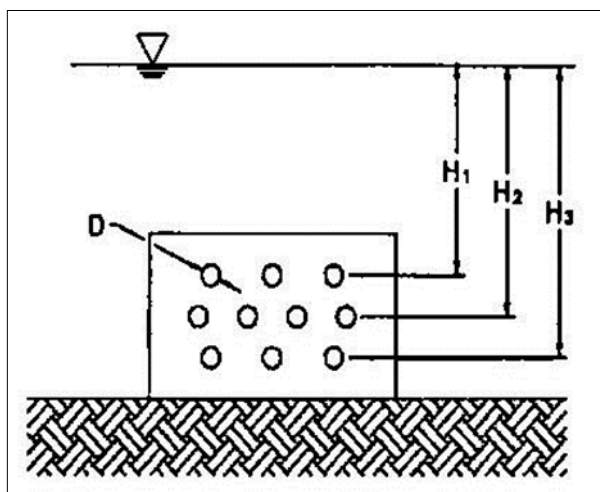
A perforated riser pipe is a commonly used outlet structure for detention ponds. It is simply a perforated pipe set in the vertical position, normally at the low point of the pond, with an open (grated) top (**Figure 506-2**).

The riser pipe open-top flow rate or capacity may be greater than the sum of the perforations, or orifices, on the pipe. In this case, it is safe to assume a free discharge condition, and the flow through each orifice is given by the orifice free discharge **Equation 502-22**. However, the outlet from the riser must be analyzed for backwater effects to verify the assumption of free discharge.



In some cases, some of the perforations may be partially or totally submerged, and the orifice submerged discharge equation (**Equation 502-23**) should be used for these situations.

A perforated riser pipe usually has perforations around the pipe circumference at different elevations. Total flow rate through multiple orifices (**Figure 506-2**) is computed as the summation of all orifices at a given elevation. When flow over the top of the open riser pipe occurs, the opening acts as a weir. This type of outlet, flow over a circular weir, is termed a "morning glory spillway" by the USBR (USBR, 1987).



Source: FHWA, August 2013, HEC-22, Figure 8-12, p. 8-21.

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

**Figure 506-2 Multiple Orifice Flow**

A trash grate should be installed on the top of the riser pipe. This will improve the outfall stormwater quality by debris retention and will reduce outlet conduit clogging from debris. A good guideline is to size the perforations around the riser such that velocities into the riser are less than 1.0 ft/s when the water level is at the top of the riser. This will reduce the maintenance caused by debris accumulation on the surface of the riser.

If the crest profile and transition conform to the shape of the lower nappe of a jet flowing over a sharp-crested circular weir, the discharge for flow over the crest and through the transition can be expressed as  $Q = CLH^{3/2}$  (see **Equation 502-15**); where  $H$  is the head measured either to the apex of the overflow or to some other established point on the overflow. Similarly, the choice of the length ( $L$ ) is related to some specific point of measurement such as the length of the circle at the apex, along the periphery at the upstream face of the crest, or along some other chosen reference line. The coefficient " $C$ " will change with different definitions of  $L$  and Head ( $H$ ). If  $L$  is taken at the outside periphery of the overflow crest (the origin of the coordinates on **Figure 506-3**) and if the head ( $H_o$ ) is measured to the apex of the overflow shape, **Equation 502-15** for weir flow can be written:

$$Q = C_o (2 \pi R_s) H_o^{3/2}$$

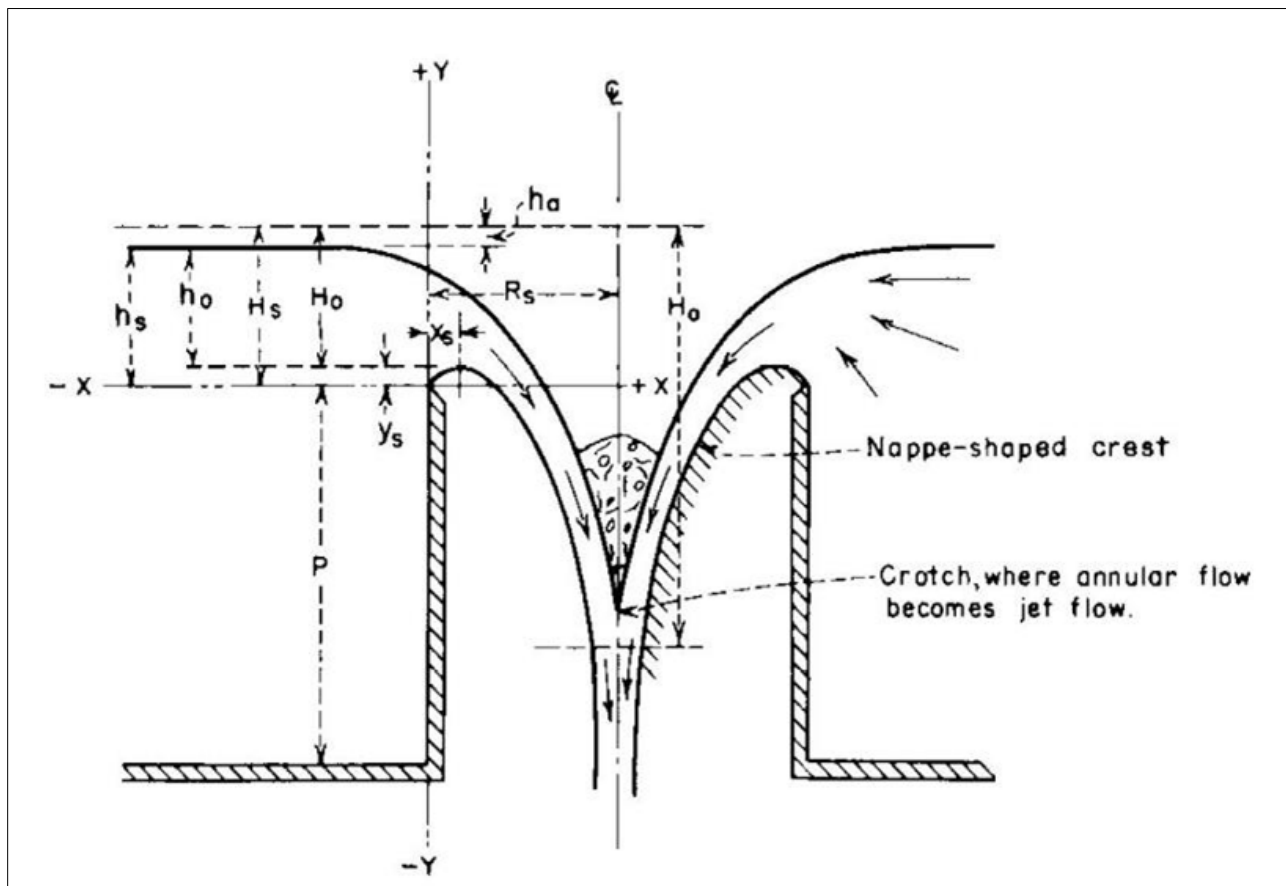
**506-2**

(USBR, 1987, Design of Small Dams, Eq. 28, p. 407)

where:

$Q$	=	discharge, $\text{ft}^3/\text{s}$
$C_o$	=	circular crest coefficient
$\pi$	=	3.14159
$R_s$	=	radius, ft
$H_o$	=	head, ft

Refer to **Figure 506-3** for illustration of  $R_s$  and  $H_o$ .



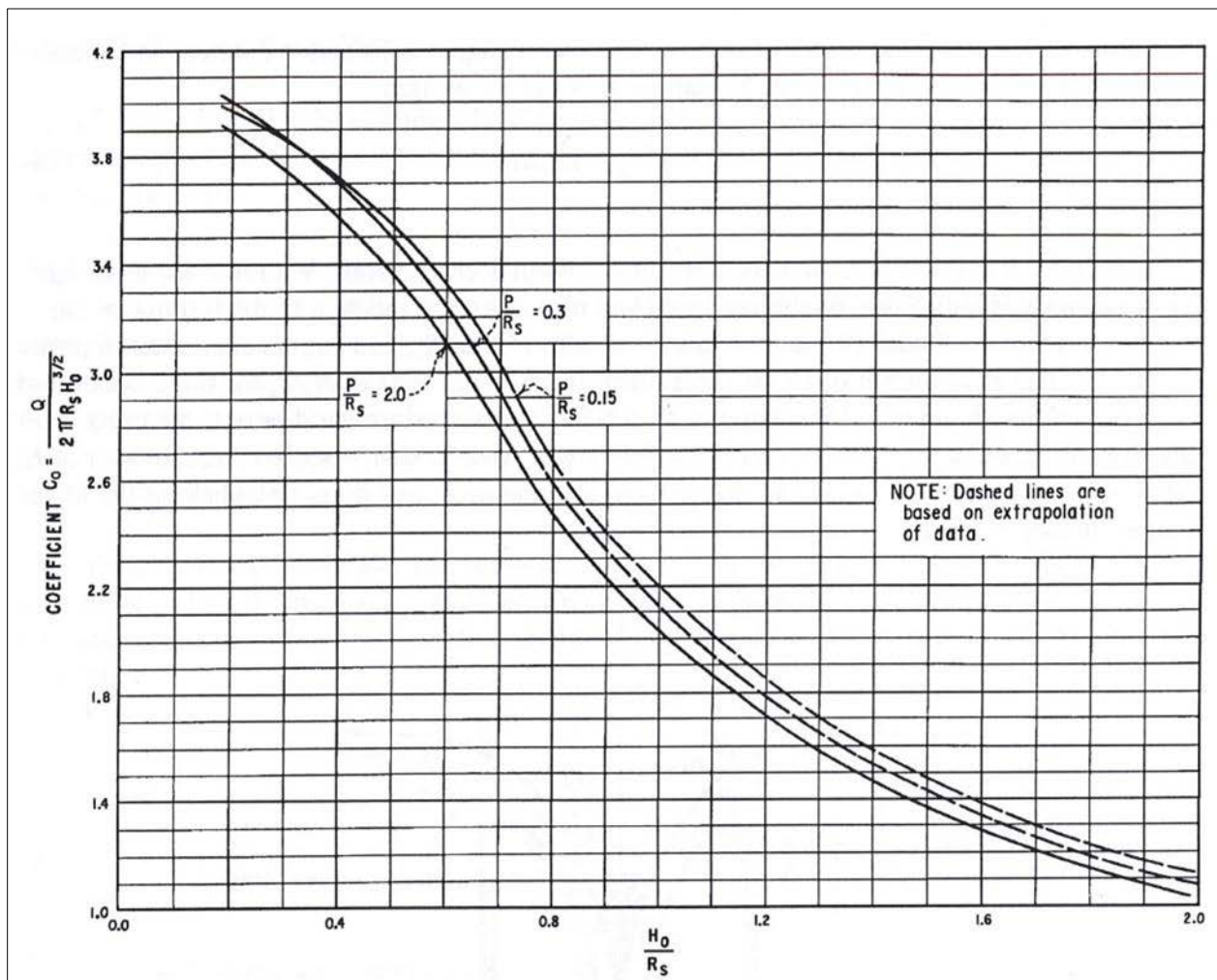
Source: USBR, 1987, Design of Small Dams, Figure 9-56, p. 409.

<http://www.usbr.gov/tsc/techreferences/mands/mands-pdfs/SmallDams.pdf>

**Figure 506-3 Elements of Nappe-shaped Profile for Circular Weir**

It is apparent that the discharge coefficient ( $C_o$ ) for a circular crest differs from that for a straight crest because of the effects of submergence and back pressure incident to the joining of the converging flows. Thus,  $C_o$  must be related to both  $H_o$  and  $R_s$ , and can be expressed in terms of  $H_o / R_s$ . The relationship of  $C_o$ , as determined from model tests to  $H_o / R_s$  for three conditions of approach depth is plotted in **Figure 506-4**. These coefficients are valid only if the crest profile and transition shape conform to that of the jet flowing over a sharp-crested circular weir at head  $H_o$ , and if aeration is provided so that sub-atmospheric pressures do not exist along the lower nappe surface contact.

When the crest outline and transition shape conform to the profile of the nappe shape for an  $H_o$  head over the crest, free flow prevails for  $H_o / R_s$  up to approximately 0.45, and weir control governs. As  $H_o / R_s$  increases above 0.45, the weir partly submerges, and flow showing characteristics of a submerged weir is the controlling condition. When the  $H_o / R_s$  ratio approaches 1.0, the water surface above the weir is completely submerged. For this and higher stages of  $H_o / R_s$  the flow phenomenon is that of orifice flow.



Source: USBR, 1987, Design of Small Dams, Figure 9-57, p. 410.

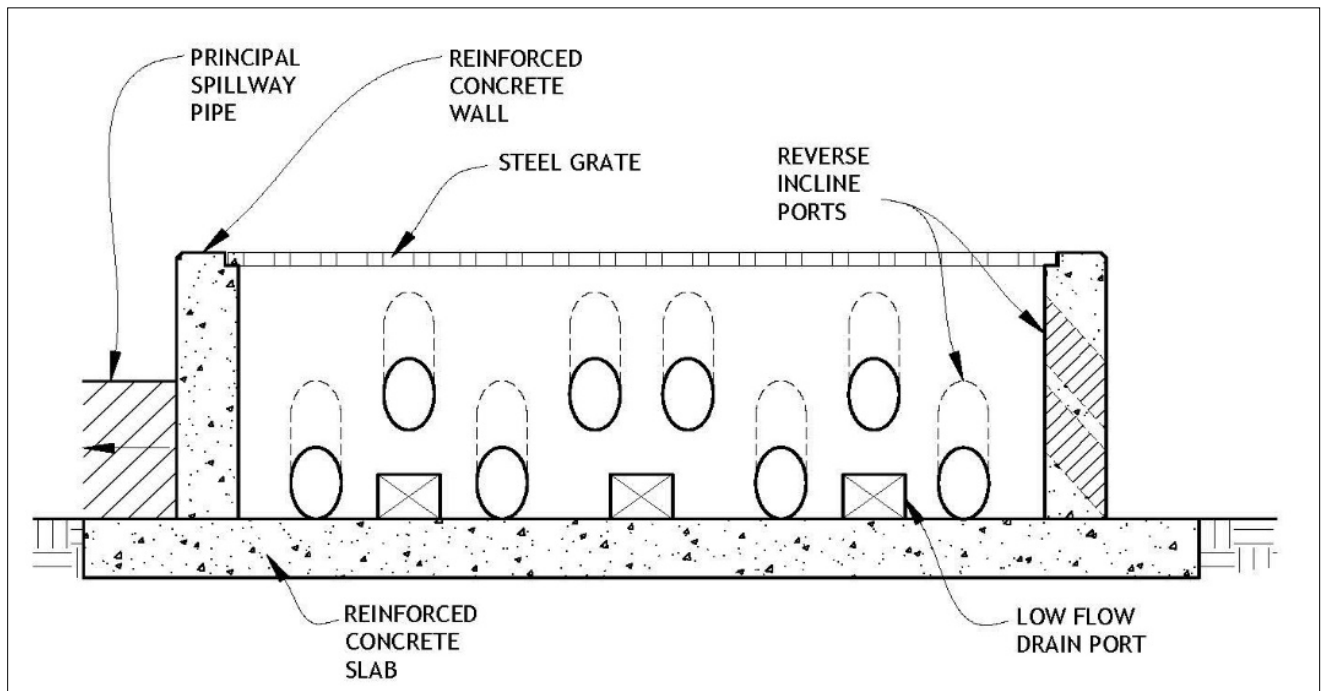
<http://www.usbr.gov/tsc/techreferences/mands/mands-pdfs/SmallDams.pdf>

**Figure 506-4 Relationship of Circular Crest Coefficient  $C_o$  to  $H_o / R_s$  for Different Approach Depths (Aerated Nappe)**

### Reverse Incline Ports

Stormwater quality features must be designed as part of the detention pond outlet structure. Reverse incline ports (pipes) are one way to control floatable debris. This design is applicable to walled structures that have a thickness sufficient to allow the reverse incline ports to be built. The ports are typically 6-inch to 8-inch PVC pipes set at a 45 degree angle with the inside or

upper pipe invert set at an elevation at or higher than the outside or lower pipe soffit elevation. Refer to **Figure 506-5** for an example of a box type of structure.



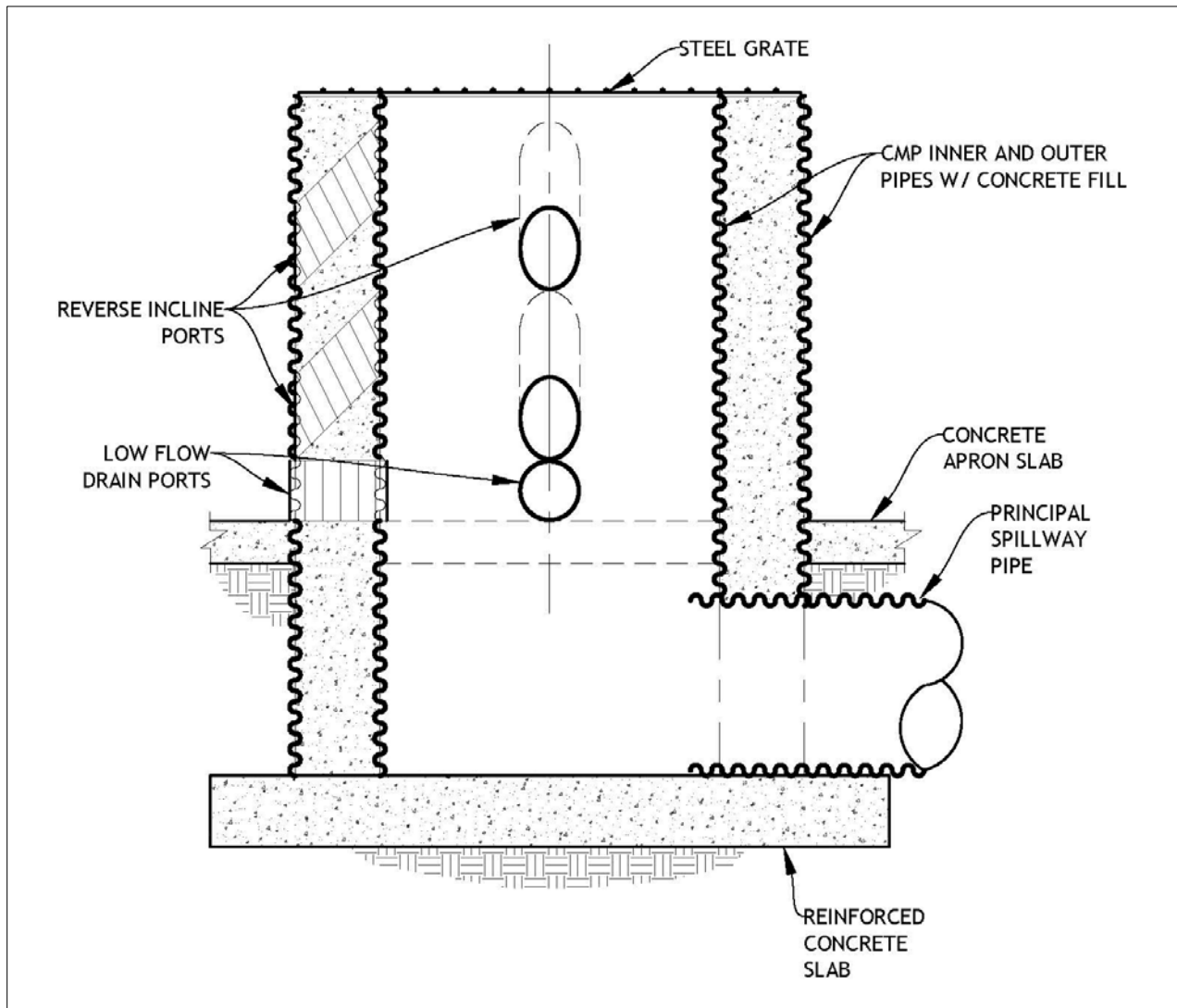
Source: Smith Engineering Company, 2016.

**Figure 506-5 Reinforced Concrete Spillway with Reverse Incline Ports**

Description of the Hydraulic Function of the Vertical Wall Principal Spillway Structure with Reverse Incline Ports

1. Low flow drain ports are located at the invert elevation of the principal spillway box walls. These are included for pond bottom drainage to minimize standing or stagnant water. The small discharge capacity may be ignored considering that sediment and debris are likely to plug these to some degree during a storm runoff event.
2. The reverse incline ports are included as stormwater quality improvement features. This design ensures that floatable debris will not enter the principal spillway box. These will function as orifice flow until submerged within the principal spillway box, and then the ports will be non-functional.
3. The grate located on top of the vertical walls will also prevent most floatable debris from entering the principal spillway box depending on the grate bar spacing. The crest of the vertical walls will function as a weir for a brief time until submerged. Note that if the principal spillway pipe capacity is lower than the weir flow capacity, then the principal spillway pipe will govern the discharge. Then the box will become full of water and will submerge the weir, and the weir will become non-functional.
4. The principal spillway pipe is the terminal outfall from the principal spillway box.

An alternate “double pipe ported riser” outlet structure comprised of two corrugated metal pipes of different diameter set vertically may be less expensive and may be more feasible, particularly for smaller detention facilities. The double ported riser concept is presented in **Figure 506-6**.



Source: Smith Engineering Company and the NMDOT, 2016.

**Figure 506-6 Double Corrugated Metal Pipe Spillway with Reverse Incline Ports**

### **506.7 Retention Storage Facilities**

Retention storage facilities which may have a permanent pool (wet pond) are often discouraged due to several factors including:

- Extensive maintenance and weed or insect control
- Need to avoid anaerobic conditions to assist in odor control
- Safety hazards and controlled access must be provided
- Long term potential for health hazards or nuisance issues

There are times, when retention storage is required due to local topography, physical and/or design constraints, and possibly downstream drainage capacity limitations. Retention ponds will normally be required on NMDOT project within MS4 boundaries (see **Section 701.2**).

Outflow rates through infiltration, percolation, and evapotranspiration for flooded conditions are difficult to calculate. Retention ponds are required primarily to address MS4 requirements and should be designed for the design runoff volume and dead storage volume (account for sediment accumulation). Retention ponds must be designed to drain completely in 96 hours. Therefore, infiltration and percolation field tests to determine rates are required to demonstrate the drain time. See **Table 207-1** for dead storage sediment requirements.

## **506.8      References**

FHWA, August 2013, "HEC-22 – Urban Drainage Design Manual".

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/10009/10009.pdf>

NMOSE, January 31, 2005, "Title 19 Natural Resources and Wildlife, Chapter 26, Surface Water, Part 2 Administrations". (State Statute - Surface Water Rights).

<http://164.64.110.239/nmac/parts/title19/19.026.0002.htm>

NMOSE, December 31, 2010, "Rules and Regulations Governing Dam Design, Construction and Dam Safety", New Mexico Office of the State Engineer Dam Safety Bureau.

<http://www.ose.state.nm.us/DS/Regs/19-25-12-NMAC-2010.pdf>

NRCS, June 1986, "TR-55, Urban Hydrology for Small Watersheds".

[http://www.nrcs.usda.gov/Internet/FSE\\_DOCUMENTS/stelprdb1044171.pdf](http://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/stelprdb1044171.pdf)

Smith Engineering Company & NMDOT, 2016, "Double Corrugated Metal Pipe Spillway with Reverse Incline Ports".

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USACE, Current Program Download, "HEC-HMS Hydrologic Modeling System".

<http://www.hec.usace.army.mil/software/hec-hms/downloads.aspx>

USACE, Current Download, "HEC-HMS Hydrologic Modeling System, Users Manual".

<http://www.hec.usace.army.mil/software/hec-hms/documentation.aspx>

USBR, 1987, "Design of Small Dams".

<http://www.usbr.gov/tsc/techreferences/mands/mands-pdfs/SmallDams.pdf>

## **507      Construction and Maintenance Considerations**

### **507.1      Introduction**

The NMDOT has engineers that design projects and also assist in design coordination with engineering consultants. In addition, the NMDOT has staff that inspects and monitors projects during construction. Design personnel should be aware that there are construction-related

design considerations and construction personnel should be aware that there are design-related construction considerations. Therefore, open communication is important and will lead to fewer design problems and field revisions.

Construction-related hydraulic considerations are a necessary part of the planning and design phases. Those responsible for contract administration and actual construction may need to coordinate their scheduling and construction procedures with the engineer in order to achieve the intended results. Any special or unique construction requirements should be communicated to the engineer prior to the final design phase of the project.

The engineer should meet with the Project Manager/Engineer prior to the preconstruction conference to explain the special features and planned construction phasing where these considerations are necessary to the proper functioning of the design. It may be advisable and necessary to specify certain time limits as to when the work will be completed. Phased construction may be needed to accommodate seasonal variations, floods, fish passage, irrigation, etc. In addition, the need for special considerations related to needed temporary work, detours, and public safety issues can be outlined and discussed. *It should be emphasized by the engineer that proposed construction revisions to drainage designs as contained in construction plans should be discussed with the engineer prior to execution.*

Cost is an important consideration in any design. The primary components of costs are related to materials and construction. Future maintenance costs are an important design consideration. The engineer must achieve the proper balance between material costs, construction costs, and maintenance costs.

Ordinarily, material costs are optimized by using available materials in a consistent manner; recycling materials, and researching programs to identify potential construction materials and how they may be utilized efficiently; using reasonable safety factors in design; and encouraging and allowing alternatives where possible. In some cases, the least expensive material may not be the proper choice because installation costs may be greater than a more expensive material. Also, future maintenance/replacement costs may rule out the less expensive material cost advantage.

Construction costs are affected by:

- The quality of the surveys (topographic, utility, and right-of-way) available to the engineer
- Relative difficulty of construction
- Laws, rules, and/or regulations governing construction procedures
- Degree of competition among contractors
- Construction latitude allowed by the specifications
- Quality of the construction plans
- Existing soil properties
- Field conditions
- Project location

The choice of a more complicated and expensive construction procedure may be prudent if it allows the use of more economical materials, decreases maintenance costs, and eliminates or reduces the need for replacements.

The engineer must be aware of the above relationships and how they affect costs and consider them in design. Additionally, construction plans should reflect these considerations by containing:

- Suggested construction sequences that consider construction costs and environmental considerations as well as public convenience
- Subsurface soil borings
- Utility locations and descriptions
- Consistent plan format which enhances the ability to assimilate and understand the plans

Despite the best efforts, construction changes to the design plans occasionally occur. The engineer should be consulted when these changes may affect the proper functioning of the drainage facilities.

The post-construction inspection following completion of the project should document any deviations from the original plans as well as an initial assessment of the hydraulic performance. Construction personnel should be encouraged to inform the engineer of any design-related difficulties which are encountered and give suggestions to improve future designs. Changes must be incorporated into the record drawings for future reference.

Land use changes in the watershed can modify the hydrology and debris considerations used in the design. New developments along the project could change damage risk considerations for the design. Depending on the time that has elapsed between completion of the design plans and the beginning of construction, changes in land use could significantly affect the validity of design considerations.

Commercial watercourse sand and gravel mining is a common practice that drastically changes the watercourse bed slope, bed width, flow direction and distribution, flow velocities, volume, and character of the bedload. Land clearing for agricultural purposes may create a need to reconsider the location and size of waterway openings and the need for spur dikes. Land development near the site could change damage risk considerations for the crossing. The engineer should be consulted regarding the need to modify the design at any drainage facility site which has changed significantly from the conditions which existed during design.

Changes in stream alignment and profile can result in different flow conditions than those used in the outfall or cross drain design. Drainage area changes due to diversions or site grading can affect inlet and outlet locations and type, as well as storm drain or roadside ditch designs.

Utilities installed after the survey may require extensive redesign of storm drain systems to avoid conflicts. This reinforces the urgent need for good utility surveys prior to design in order to avoid costly redesigns and delays.

Additional changes which may affect drainage designs may include the following items.

### **Possible Errors or Omissions**

- Incorrect existing structure and/or invert elevations
- Inappropriate typical section choice, and/or incorrect grade(s)
- Incorrect drainage area size
- Channel alignment and profile
- Utilities not documented
- Existing structure condition (as related to service life, outlet scour, sedimentation, etc.)  
Note – structure conditions may not be observable due to sediment deposition
- Local flooding not documented
- Existing slope and channel erosion not documented
- Sensitive receiving waters not documented
- Debris problems not documented



- Existing or potential nuisance problems not documented

### **Possible Changes**

- Increased development
- Channel improvements
- Diversion or site grading changes
- New utility
- Loss of outfall due to development

In some cases, a considerable amount of time may elapse between design and construction. In other cases, designs may change during construction. Any changes in the plans, specifications, and estimates should be reflected in the final plans. If questions arise, the construction personnel should check with the engineers to determine if changes have been made and how construction should proceed.

## **507.2 Pre-Bid Conference**

It is important for the design engineer to inform the project manager/engineer of special features of the design and planned construction phasing before the pre-bid conference. This should be done far enough in advance to allow the project manager/engineer the opportunity to visit and evaluate the special feature areas and then convey the information when showing the project area to all prospective bidders. The project manager/engineer can also determine whether any necessary time limits for construction must be imposed within the special feature area.

The purpose of the meeting is to discuss the design and construction aspects of the project, thus affording all parties a common understanding of the proposed work and the problems and possible solutions which may be expected.

Several other concerns should be discussed at the pre-bid conference including drainage maintenance during construction, water pollution, and erosion control.

Drainage work on some projects may be completed several months before total project completion. Immediately following completion of the drainage work, vegetative erosion control measures are not well established. Maintenance to correct erosion and sediment deposition in the newly constructed channels is important to achieving the results intended. Therefore, NMDOT will require the contractor to maintain the newly constructed drainage structures for the duration of the contract and/or to install interim protective measures. The NMDOT may advance its own maintenance schedule to assure that minor damages do not develop into major damages.

During the pre-bid conference, provisions of the contract relating to pollution control should be reviewed. After the contract is awarded, the contractor should submit a Stormwater Pollution Prevention Plan (SWPPP) in accordance with the National Pollutant Discharge Elimination System (NPDES) requirements. The NPDES Handbook (NMDOT, August 2012) includes information and figures illustrating various Best Management Practices (BMPs). Refer to the NMDOT standard drawings (latest version) to assist the contractor in complying with the NPDES regulations.

NMDOT, August 2012, "National Pollutant Discharge Elimination System Manual – Stormwater Management Guidelines for Construction and Industrial Activities, Revision 2".

<http://dot.state.nm.us/content/dam/nmdot/Infrastructure/NPDESM.pdf>

NMDOT, Website, “Standard Drawings”, “Standard Specifications for Highway and Bridge Construction”.

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

### 507.3 Open Channels

Many of the construction considerations for open channels are the same as for culverts and bridges. Thus, **Section 502** should be reviewed as it relates to open channel design. The following discussions will concentrate on those construction considerations that are more appropriate to open channels.

Bank stabilization is an important aspect of open channel construction. In some cases, a considerable length of a stream or channel system may be disturbed by construction, great care should be exercised in scheduling and implementing stabilization measures. Immediately prior to the commencement of construction of bank stabilization measures, the project manager/engineer should inspect the site to ensure that the measures proposed are not inappropriate because of bank movement subsequent to the completion of design surveys. If a problem is discovered, the engineer should be notified.

Channel excavation work on some projects may be completed several months before the total project completion. The time between the completion of channel excavation and the total project completion is usually longer when grading and structure projects are separated from the contract for paving or stabilization. During this period, vegetative erosion and control measures are not established, so maintenance is important to correct erosion and sediment deposition in the newly constructed channels.

Damaged channels can be both expensive to repair and hazardous to traffic. To facilitate repair and maintenance, channels should be designed recognizing that periodic maintenance, inspection, and repair are required. Where possible, access should be incorporated for personnel and equipment during the construction period and afterward. Consideration should be given to the size and type of equipment which will ordinarily be required in assessing the need for access easements, entrance ramps, and gates through right-of-way fences and fee right-of-way.

### 507.4 Culverts

Construction/assembly, bedding, and backfill are as important to culvert service as the hydraulic and structural design. Culverts should be protected from damage during construction operations and should be periodically inspected. A particularly critical time for inspection is upon the completion of grading operations and prior to the start of surfacing operations. It is as important to inspect culverts which are not under the roadway, as it is for those structures that are under the roadway. Prior to the final acceptance of the project, all culverts should be inspected and cleaned as necessary.

Records should be kept for each culvert installation. The plans and record drawings should be annotated with various items including drainage area, peak discharges (with recurrence interval), invert elevations, maximum headwater elevations (with recurrence interval), and slope. This information is useful for evaluating overall performance of the installation and for future maintenance or construction operations. See **Section 307** for information required on construction plans.

The following records should be kept for each installation: location and layout including station, skew(s), location of inlets, outlets, junctions, flowline elevations in inlets, junctions, alignment, and grade.

Culverts shall be designed and constructed to: accommodate the design criteria, provide for outlet energy dissipation and erosion protection, and to minimize hazards to the traveling public by means of slope blankets and related features. Whether within or outside of the clear zone, culvert end treatments and erosion protection measures are applicable. Refer to HEC-14, “Hydraulic Design of Energy Dissipators for Culverts and Channels” (FHWA, July 2006).

FHWA, July 2006, “HEC-14, Hydraulic Design of Energy Dissipators for Culverts and Channels”.  
<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/06086/hec14.pdf>

## **507.5 Bridges**

The responsibility for construction-related hydraulic considerations of bridge construction ordinarily rests with the contractor, but in some cases, the NMDOT may include construction-related details in the plans and specifications in order to mitigate potential environmental effects or to assume or reduce the risk of failure during construction. In addition, other special provisions related to the construction phase of the bridge may be specified in the plans.

A hydrograph of superimposed mean daily flows and a plot of the rating table for a stream gaging station near the crossing site are useful for:

- the design of cofferdams, falsework, and temporary crossings
- selecting the location of work and material storage areas

In the event a gaging station is not located near the stream crossing site, records from upstream or downstream gages may be useful as an indication of the usual magnitude, duration, and time of flood events.

Any site conditions that might impact the foundation design or create unusual scour problems should be discussed with the NMDOT Bridge Section personnel to determine if design changes are necessary. If specific elements of the bridge design are dependent on special foundation or scour considerations, this information should be presented to the project manager/engineer prior to the preconstruction meeting.

Cofferdams, falsework, and occasionally contractor's equipment, constrict the stream channel more than the completed substructure and consequently have greater potential for causing scour and bank caving and for collecting debris. If possible, schedule work to avoid flood seasons.

Temporary stream crossings necessary for the construction of bridges are usually the responsibility of the contractor and should be included as part of the SWPPP prepared by the contractor. Coordinate with the NMDOT Drainage Design Bureau regarding design of such crossings to minimize or mitigate the adverse effects on the stream environment, to facilitate securing permits, or to reduce the risk assumed by the contractor, and thereby, reduce construction costs.

In addition, stream crossings for detours are built to much lesser standards than permanent crossings. Note that the design of detours and associated stream crossings should occur during project development and are part of the project design, not the responsibility of the contractor. The criteria used for the hydraulic design of detour stream crossings should be based on risk factors which should be evaluated considering the probability of flood exceedance during the

anticipated service life of the detour (the construction period for the crossing), the risk to life and property, and traffic service requirements.

As in the case of the design of highway stream crossings, detour designs should accommodate floods larger than the event for which they are designed to avoid damages from excessive backwater and to reduce the probability of losing the detour stream crossing structure during a larger flood. In most instances, the conveyance of floods larger than the detour design flood is provided for by a low roadway profile which allows overflow without creating excessive velocities or backwater.

Minimum disturbance of the banks and bed of a stream during the construction period will reduce erosion damage to the banks, sedimentation, and harm to fish and wildlife. Embankments in or along streams should be constructed of erosion-resistant material and/or protected against erosion to avoid adverse sediment concentrations which contribute to the turbidity of the stream.

Consideration should be given to precluding in-stream operations that would cause turbidity during the spawning season of certain fish species. Detours and construction roads are other sources of turbidity and should either be constructed at a time when fishery activities are not disturbed, or provisions should be made to control any harmful effects of erosion.

Pumping of cofferdams and other dewatering operations may have a discharge of unacceptable quality to the receiving stream. Mitigation measures such as settling basins may be necessary if the ecosystem of the stream would be upset by the temporary water quality degradation.

Often engineers do not have an opportunity to participate in the construction of the projects that they have designed. For this reason, designs that could be improved upon for construction purposes tend to be perpetuated simply because the engineer is not informed of the deficiencies. Engineers are encouraged to visit construction sites to discuss problems with designs and possible improvements in future designs. This is especially important for major projects like bridge construction. Upon completion of a project, a design critique conducted jointly by engineers and field personnel can be a useful learning experience for both. This critique should include difficulties encountered in construction and possible design changes to prevent such difficulties in the future. This will also give the engineer an opportunity to present why some difficulties in construction are necessary because of specific design considerations.

## **507.6 Stormwater Storage Facilities**

Proper stormwater storage facilities design should focus on the design of certain features that will assist maintenance and access. Other items should also be considered as presented here:

- Design a maintenance access road to the pond bottom and to the principal spillway
- Design a hardened surface around the principal spillway structures to provide a solid surface for maintenance vehicles
- Design outlet structures to minimize sedimentation and debris blockage (small pipes tend to block quite easily and should be avoided)
- Locate the facility to provide easy maintenance access to the pond
- A fence will be required around all ponds to minimize public safety hazards
- Weed growth and grass maintenance may be addressed by the design of mild embankment side slopes that may be maintained using available power-driven equipment, such as tractor mowers

- Embankment and slope rill erosion may be minimized with protective lining and/or by the design of mild slopes and/or by the use of small contour ditches/dikes on the slopes that reduce the effective slope lengths, thus minimizing rill erosion
- Sedimentation may be controlled by constructing traps to contain sediment for easy removal or by low-flow channels to reduce erosion and sediment transport by frequent or continuous flows
- Standing water or soggy surfaces may be eliminated by sloping basin bottoms toward the outlet, by constructing low-flow pilot channels across basin bottoms from the inlet to the outlet, or by constructing underdrain facilities to lower water tables
- In general, mosquito or pest control needs may be eliminated or minimized when the above problems are addressed

Refer to **Section 207** for additional pond design criteria and design requirements.

### **507.7 Record Drawings**

Record drawings serve many functions related to the design and construction process including documentation of:

- The final location of all elements of the drainage system and related facilities
- Any changes that were made in the design during the construction process such as slopes, size of facilities, materials used, addition facilities or elimination of facilities
- Any variation between the original plans and specifications and the final installed facilities

Preparation of accurate and complete record drawings are invaluable in documenting:

- Changes that may be incorporated in future designs
- A resource for future investigations of the project if problems are encountered
- Future need to re-analyze the performance
- Documentation in case of future legal action

### **507.8 References**

FHWA, July 2006, "HEC-14, Hydraulic Design of Energy Dissipators for Culverts and Channels".  
<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/06086/hec14.pdf>

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

NMDOT, Website, "Standard Drawings", "Standard Specifications for Highway and Bridge Construction".  
<http://dot.state.nm.us/content/nmdot/en/Standards.html>

# 600 EROSION, SEDIMENTATION, SCOUR AND COUNTERMEASURES

## 601 Introduction

The control of erosion and sedimentation has generated much attention as a result of the National Environmental Policy Act (NEPA) of 1969. As a result, many federal and state regulations have been developed regarding land and watercourse disturbing activities. There are federal and state control requirements through the administration of various permitting requirements enforced by numerous agencies including:

### Federal Agencies

- U.S. Environmental Protection Agency (EPA)
  - National Pollutant Discharge Elimination System (NPDES)
  - Stormwater General Permit for Municipal Separate Storm Sewer Systems (MS4s)
  - Stormwater Pollution Prevention Plan (SWPPP)
- U.S. Army Corps of Engineers (USACE)
  - Sections 402 and 404 of the Federal Water Pollution Control Act (FWPCA) and Sections 9 and 10 of the River and Harbor Act
- U.S. Fish and Wildlife Service (USFWS)
  - Sections 402 and 404 of the Federal Water Pollution Control Act (FWPCA) and Sections 9 and 10 of the River and Harbor Act

### State Agencies

- New Mexico Environment Department (NMED)
  - 401 Clean Water Act Permit

Erosion and sedimentation are natural geologic processes whereby soils are transported from one location and deposited in another, primarily due to rainfall and runoff. Accelerated erosion and sedimentation may occur in conjunction with development, highway construction or transportation facility construction. Accelerated erosion can result in significant affects such as slope instability, watercourse bed and bank instability, ecosystem disruption, safety hazards, and excessive maintenance. The design of drainage facilities must include countermeasures for anticipated erosion and sedimentation.

Countermeasures to control erosion and sedimentation can be either temporary or permanent. Temporary measures are generally employed during the construction phase of the project. The construction contractor is usually responsible for developing a Stormwater Pollution Prevention Plan (SWPPP) as required under the NPDES to address erosion and sediment control during construction. The NMDOT has developed a NPDES Manual (NMDOT, August 2012) to provide assistance regarding the design of temporary measures. Permanent countermeasures are generally more extensive, costly, and require considerably more analysis and design effort.

This section will concentrate primarily on analysis methods to assist in the design of permanent erosion and sediment control measures. Due to the large volume of information on topics

regarding sediment transport, scour, and countermeasure selection and design, this Manual includes references and hotlinks to additional analysis and design guidance that is beyond the scope of this Manual.

### **601.1 References**

NMDOT, August 2012, “National Pollutant Discharge Elimination System Manual (Stormwater Management Guidelines for Construction and Industrial Activities, - Revision 2”.

<http://dot.state.nm.us/content/dam/nmdot/Infrastructure/NPDESM.pdf>

## **602 Stream Restoration**

Highway construction that requires bridges or culverts and associated stream bed and bank stabilization measures, particularly in perennial streams, will affect the natural watercourse. The topic of stream restoration is beyond the scope of this manual; however, the engineer may find guidance through numerous other publications. Some additional references are included in the references section.

Biotechnical Engineering in combination with hydrologic/hydraulic and sediment transport engineering analyses should be considered when developing solutions for stream restoration. Additional Biotechnical Engineering information may be found in Chapter 6 of HEC-23 (FHWA, September 2009). Refer to Chapter 9 of HEC-20 (FHWA, April 2012) for additional general information regarding channel restoration. Hotlinks to these documents are provided in **Section 602.1**.

### **The Rosgen System**

Dave Rosgen has developed a Stream Classification System to assist in stream restoration. This system is not an engineering solution. Soft solutions such as timber, large stones, vegetation, etc. are suggested for stream stabilization techniques. Many federal and state agencies are aware of the Rosgen system. Refer to HEC-20 (FHWA, April 2012, pg. 5.16 and Chapter 9) for more information regarding the Rosgen system of stream classification.

### **602.1 References**

FHWA, September 2009, “HEC-23, Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance, Third Edition, Volume 1”.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/09111/09111.pdf>

FHWA, April 2012, “HEC-20, Stream Stability at Highway Structures, Fourth Edition”.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12004.pdf>

NRCS, August 2007, “Part 654 Stream Restoration Design, National Engineering Handbook”.

<https://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=17807.wba>

Zeedyk, Bill and Clothier, Van, 2009, “Let the Water Do The Work: Induced Meandering, an Evolving Method for Restoring Incised Channels”.

## **603 Factors Influencing Erosion**

The erosion potential of any watershed is determined by several factors that include:

- Soil characteristics
- Vegetative cover
- Topography
- Climate
- Impervious area that generates clear water runoff for even small rainfall events
- Flow redirection or flow concentration
- Construction activities that modify the natural topography and landscape
- Length of unprotected slope

Erosion due to the last four items is a result of human activity. The first four inherent factors are described below.

### **603.1 Soil Characteristics**

The properties of soil that influence erosion by rainfall and runoff affect the infiltration capacity of a soil and affect resistance to detachment and water transport. Soils containing high percentages of fine sands and silt are normally the most erodible. As clay and organic matter content increase, the erodibility will decrease. Clays act as a binder to soil particles, thus reducing erodibility. However, while clays have a tendency to resist erosion, once eroded, they are easily transported by water. Soils high in organic matter have a more stable structure which increases their permeability. Such soils resist raindrop detachment and allow more rainwater infiltration. Clean, well-drained, and well-graded gravels and gravel-sand mixtures are usually the least erodible soils. Soils with high infiltration rates and permeability have lower runoff rate and volume. Fine sandy soils found throughout New Mexico are generally highly erodible.

### **603.2 Vegetative Cover**

Vegetative cover plays an important role in controlling erosion in the following ways:

- Shields the soil surface from rainfall impact
- Holds soil particles in place
- Enhances the soil's capacity to absorb water
- Slows the velocity of runoff
- Removes subsurface water between rainfalls through the process of evapotranspiration

### **603.3 Topography**

The size, shape, and slope characteristics of a watershed influence the runoff volume and rate. As slope, length, and gradient increase, the runoff volume and runoff rate increase, which results in a higher potential for erosion. North or south slope orientation may also be a factor in determining erosion potential. North facing slopes may retain moisture longer due to less sun exposure and therefore may produce more vegetative cover at a faster rate than south-facing slopes.



### **603.4 Climate**

The frequency, intensity, and rainfall duration are fundamental factors in the determination of runoff volume and rate. Detachment and transport of soil particles is a function of rainfall impact, runoff volume, and rate. In New Mexico, where storms are of high intensity, erosion risks are high. Seasonal changes in temperature, as well as variations in rainfall, help to define the high erosion risk periods. When precipitation falls as snow, no erosion will take place. However, during the spring snow melting period, excess runoff will contribute to erosion potential. Also, during the spring the soil may be partially frozen, and its absorptive capacity is therefore reduced. Although frozen soils are relatively erosion-resistant, soils with high moisture content are subject to swelling by freezing action and are usually easily eroded upon thawing.

## **604 Erosion Control Measures**

The following guidelines should be followed during the development and implementation an erosion and sediment control design:

- Design structures that utilize the existing topography, soils, and natural vegetation
- Minimize the duration of exposed bare soil without erosion protection
- Develop erosion control measures to mitigate and control erosion from the site
- Develop sediment control measures to prevent offsite damage
- Develop and monitor a thorough maintenance and inspection program

Control measures such as stabilizing emulsions and vegetation are required for all disturbed areas. Vegetative measures may include retention or provision of strips of vegetation to provide a filtration buffer, permanent seeding, and mulching. Structural control measures may be required for larger sites that generate significant runoff. Structural measures may include sediment traps, sediment basins, diversions, and permanent drainage facilities.

An erosion and sediment control plan should be included in the construction documents, including appropriate construction specifications for all control measures. These plans and specifications should be developed in consultation with the engineer for site-specific conditions. Typical specifications may be obtained from various erosion control references and the NMDOT Standard Drawings and Specifications.

### **604.1 Vegetation**

Vegetative filter strips may be used to remove suspended solids from sheet-flow runoff but are not suitable for controlling erosion and sedimentation from concentrated flows. Filter strips may be used to control suspended soils on areas with slopes up to 10 percent and with slope lengths up to 100 feet.

#### **Vegetation Limitations**

- Permanent seeding with perennial cover is suitable for cases where the life expectancy of temporary plantings is inadequate to protect a site during long periods between disturbance activities. Permanent seeding is also appropriate for final vegetative cover establishment during acceptable growing seasons.

- Mulching should be used with all seeding operations to provide temporary protection during adverse growing seasons. Typical mulch material includes straw, hay, and wood chips. Gravel mulch has also been used successfully in New Mexico.

Seed bed preparation is an important consideration for all vegetative control measures. Soil characteristics, such as the depth to rock, pH, fertility, and moisture should all be evaluated during plant selection. Lime, fertilizer, and irrigation will often be required to establish a vegetative cover that meets local requirements. The amounts of lime and fertilizer required will vary by location. The local National Resource Conservation Service (NRCS) office may also provide guidance. The erosion and sediment control plan should clearly specify soil preparation requirements for the project site. Erosion and sediment control facilities may not be considered complete until suitable vegetative cover is established.

## **604.2 Channel Lining**

One means of reducing erosion during highway construction and operation is through the use of properly designed linings in drainage channels. Lining may be rigid, such as Portland cement concrete, soil cement, and asphaltic concrete; or flexible, such as vegetation, rock riprap, or gabions. Flexible linings of erosion-resistant vegetation, rock riprap, or gabions should be used whenever feasible. When vegetation is chosen as the permanent channel lining (rare in New Mexico), it may be established by seeding. Seeding usually requires protection by one of a variety of temporary lining materials until the vegetation becomes established.

### **Channel Lining Limitations**

Flexible linings are generally less expensive to install than rigid linings and are adjustable to some changes in channel shape, which reduces maintenance costs. Flexible linings also permit infiltration and exfiltration, may have a natural appearance (especially after vegetation is established), and provide a filtering media for runoff contaminants.

Flexible linings have a limited hydraulic performance range (depth, grade, velocity, and discharge). The hydraulic performance range is limited by the magnitude of hydraulic force that the lining can withstand without eroding. As a result, the channel may provide a lower flow capacity for a given cross sectional area when compared to a rigid lining. Limited right-of-way access, limited rock (riprap) availability, or the inability to establish vegetation may preclude the use of flexible linings. In these instances, rigid linings may be the only alternative.

Rigid linings are generally smooth so that they have a high flow capacity for a given cross sectional area due to low hydraulic resistance, and thus produce a high flow velocity. When properly designed and constructed, rigid linings will prevent erosion in steep or difficult channels where other linings cannot be used. Rigid linings may also be used in areas where the channel width is restricted. So long as the rigid lining is intact, the underlying soil is completely protected after construction of the lining.

Rigid linings also have a number of inherent disadvantages. They are expensive to construct and maintain, they may have an unnatural appearance, they prevent or reduce natural infiltration, and they may contribute to scour at the downstream end of the lining unless roughness features and/or energy dissipation features are added to reduce the flow velocity. Rigid linings may be destroyed due to slow undercutting of the lining, channel head cutting, or hydrostatic pressure behind the channel walls or floor.

## Lining Types

The following is a brief description of some channel linings which can be used for erosion control.

Rock Riprap (flexible lining) well graded rock with a properly designed gravel filter blanket can stabilize channel bottoms and side slopes where erosion may be anticipated. Filter blankets are required and serve the purpose of minimizing subgrade soil loss below the rock. A filter blanket consists of crushed gravel and/or one or more layers of either: 1) a graded gravel material placed as a subgrade, or 2) a nonwoven geotechnical fabric that is placed on the soil surface prior to rock placement. The thickness and gradation of the gravel filter blanket or specification of the nonwoven geotechnical fabric must be included in the plans.

Dumped Riprap (flexible lining) consists of stone or broken concrete “dumped” in place to form a well-graded mass with minimum voids.

Wire enclosed Riprap or Gabions (flexible lining) consist of mats or baskets fabricated from wire mesh, filled with stone, connected together, and anchored to the slope. Details of construction may differ depending upon the degree of exposure and service, whether used for revetment or used as a toe protection in conjunction with other types of riprap.

Concrete-slab Riprap (“Slope Paving”) consists of plain or reinforced concrete poured in place, with riprap embedded into the concrete prior to hardening.

Grouted Riprap consists of riprap with all or part of the interstices filled with Portland cement mortar.

Concrete Cloth is a fabric that is imbedded with Portland cement and comes in various thicknesses from ¼ inch to ½ inch. The dry cloth is laid in place, secured to the ground with staples and anchor trenches, and then hydrated with clean water. The cloth then cures in place.

Revetments are either flexible or rigid structures placed longitudinally along a channel to protect a bank line or to establish a new bank line. Flexible revetments include dropped riprap, wire-enclosed riprap, and gabions. Rigid revetments include concrete pavement, grouted riprap, bagged riprap, and soil cement.

## Design Considerations

Rigid Channel Linings - For rigid channel linings such as concrete or soil cement, there is no maximum permissible depth for the flow velocities normally encountered in highway drainage work, since no erosion can occur. Thus, the maximum flow depth is based only on the channel freeboard requirement.

Flexible Channel Linings - For design considerations regarding vegetation, cobble lining, rock riprap, wire-enclosed riprap and other less common linings, the engineer should refer to HEC-15, “Design of Stable Channels with Flexible Linings”, (FHWA, September 2005).

## 604.3 References

FHWA, September 2005, “HEC-15, Design of Roadside Channels with Flexible Linings, Third Edition”.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/05114/05114.pdf>

NMDOT, Website, “Standard Drawings”, “Standard Specifications for Highway and Bridge Construction”.

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

## **605 Stream Stability**

Stream or watercourse stability adjacent to and within a bridge/highway crossing is important to assure the integrity of the bridge and associated structures including piers, abutments, guide banks, etc. An unstable watercourse may lead to bank instability that could result in encroachment into road embankments, cause loss of useable property, cause exposure and/or failure of bridge piers and/or abutment foundations, or cause other scour related issues. This section incorporates information from the following documents by the FHWA. Note that these documents are intended to be used together to provide guidance for stream stability analyses, scour and sediment transport analyses, and scour countermeasure design.

FHWA, April 2012, “HEC-18, Evaluating Scour at Bridges, Fifth Edition”.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12003.pdf>

FHWA, April 2012, “HEC-20, Stream Stability at Highway Structures, Fourth Edition”.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12004.pdf>

FHWA, September 2009, “HEC-23, Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance, Third Edition, Volumes 1 and 2”.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/09111/09111.pdf>

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/09111/09112.pdf>

The engineer is encouraged to refer to and study these manuals for a more in-depth discussion on the following topics.

### **605.1 Factors That Affect Stream Stability**

Understanding the nature of watercourse geomorphology is relevant to evaluating and analyzing long term aggradation/degradation and for planning and designing watercourse structures and bridges. This includes understanding landform evolution and geomorphic factors that affect stream stability. Please refer to Chapter 2 of HEC-20 (FHWA, April 2012) for a comprehensive description of geomorphic factors and principles.

Factors which affect stream stability and potentially bridge stability at highway stream crossings can be classified as geomorphic factors and hydraulic factors. Rapid and unexpected changes can occur in streams in response to human activities in the watershed and/or natural disturbances of the alluvial system, making it important to anticipate changes in channel geomorphology, location, and behavior. Geomorphic characteristics of particular interest to the highway engineer are the alignment, geometry, and form of the stream channel. The behavior of a stream at a highway crossing depends not only on the apparent stability of the stream at the bridge but also on the behavior of the stream system. Upstream and downstream changes may affect future stability at the site. Natural disturbances such as floods, drought, earthquakes, landslides, forest fires, etc., may result in large changes in the watercourse sediment load

causing major channel changes. These changes can result in aggradation/degradation, and/or watercourse lateral migration.

Geomorphic factors that can influence stream stability include stream size, flow habit (i.e., ephemeral or perennial) and characteristics of the channel boundaries. The bed material of a stream can be a cohesive material, sand, gravel, cobbles, boulders, or bedrock. Bank material is also composed of these materials and may be dissimilar from the bed material. The stability and rate of change in a stream are dependent on material in the bed and banks. Other natural factors such as the stream's relationship to its valley, floodplain, and planform characteristics, and features such as natural levees, incision, and riparian vegetation are important indicators of stream stability or instability.

Human-induced changes in the drainage basin and the stream channel, such as alteration of vegetative cover and changes in pervious or impervious areas, can alter the hydrology of a stream, sediment yield, and channel geometry. Channelization, stream channel straightening, streamside levees and dikes, bridges and culverts, reservoirs, gravel mining, and changes in land use can have major effects on streamflow, sediment transport, channel geometry, and location. Geomorphic factors are discussed in detail in Chapter 2 of HEC-20 (FHWA, April 2012).

There are many hydraulic factors which affect stream channel and bridge stability. These factors include bed forms and their effects on sediment transport, resistance to flow, flow velocities, and flow depths. They also include the magnitude and frequency of floods; characteristics of floods (i.e., duration, time to peak, and time of recession); flow classification (e.g., unsteady, nonuniform, turbulent, supercritical or subcritical); ice and other floating debris in the flow; and flow constrictions. Other factors are bridge length, location, orientation, span lengths, pier location and shape, superstructure elevation and design, the location and design of countermeasures, and the effects of natural and human-induced changes which affect the hydrology and hydraulic flow conditions of the stream. In the bridge reach, bridge design and orientation can induce contraction scour and local scour at piers and abutments. Hydraulic factors are discussed in Chapter 3 of HEC-20 (FHWA, April 2012).

## **605.2 Permissible Tractive Force – Shear Stress**

Permissible tractive force is the method recommended by FHWA to determine channel stability (HEC-15, September 2005). The hydrodynamic force of water flowing in a channel is known as the tractive force or shear stress. The basis for stable channel design with flexible lining materials is the flow-induced tractive force should not exceed the permissible or critical shear stress of the lining materials. In a uniform flow, the tractive force is equal to the effective component of the gravitational force acting on the body of water, parallel to the channel bottom.

The average tractive force, or shear stress ( $\tau$ ) on the channel is equal to:

$$\tau = \gamma R S$$

**605-1**

(FHWA, September 2005, HEC-15, Eq. 2.3, p. 2-4)

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/05114/05114.pdf>

where:

$\tau$	=	tractive force or shear stress, lb/ft <sup>2</sup>
$\gamma$	=	unit weight of water, lb/ft <sup>3</sup>
$R$	=	hydraulic radius, ft
$S$	=	average bed slope or energy grade line slope, ft/ft

The maximum shear stress,  $\tau_d$  for a straight channel occurs on the channel bed and is equal to the shear stress at maximum depth. This stress is defined as:

$$\tau_d = \gamma d S$$

**605-2**

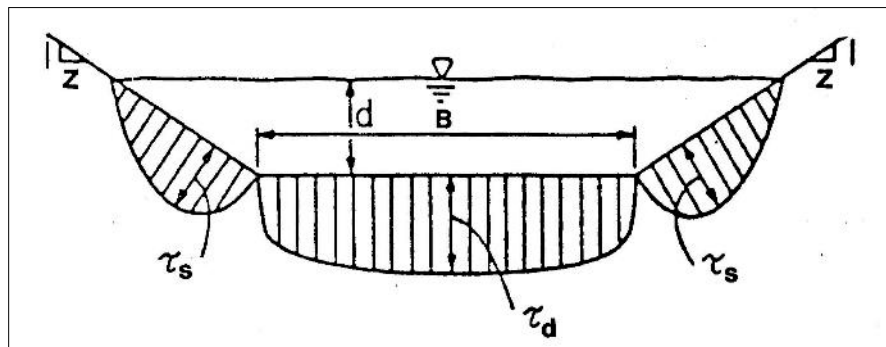
(FHWA, September 2005, HEC-15, Eq. 2.4, p. 2-5)

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/05114/05114.pdf>

where:

$\tau_d$	=	shear stress in channel at maximum depth, lb/ft <sup>2</sup>
$\gamma$	=	unit weight of water, lb/ft <sup>3</sup>
$d$	=	maximum depth of flow in the channel, ft
$S$	=	channel bottom slope or energy grade line slope, ft/ft

Shear stress in channels is not uniformly distributed along the wetted perimeter. A typical distribution of shear stress in a trapezoidal channel is approximately zero at the corners with a maximum on the centerline of the bed, and the maximum for the side slopes ( $\tau_s$ ) occurring at about the lower third of the side slope as shown in **Figure 605-1**.

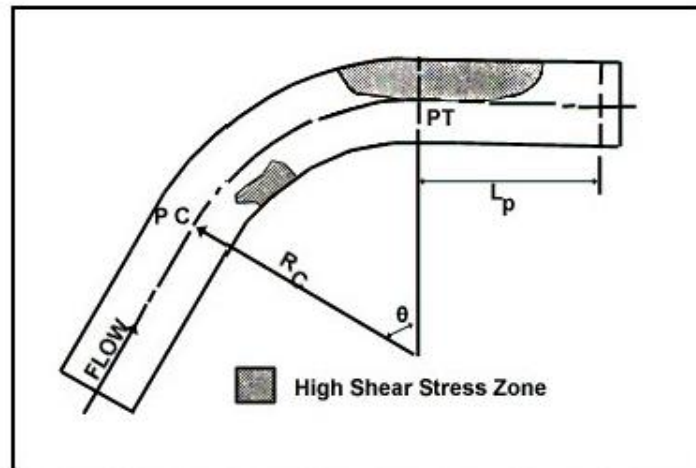


Source: FHWA, April 1988, HEC-15, Figure 18, p. 22.

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hec/hec15si.pdf>

**Figure 605-1 Typical Distribution of Shear Stress**

Flow around a bend creates secondary currents which impose higher shear stresses on the channel sides and bottom compared to a straight reach. At the beginning of the bend, the maximum shear stress is near the inside of the bend and moves towards the outside as the flow leaves the bend. **Figure 605-2** illustrates zones of high shear stress in a channel bend.



Source: FHWA, September 2005, HEC-15, Figure 3.3, p. 3-12.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/05114/05114.pdf>

**Figure 605-2 High Shear Stress Zones in a Channel Bend**

The maximum shear stress in a bend is a function of the ratio of channel curvature ( $R_c$ ) to top width ( $T$ ) (water surface),  $R_c/T$ . As  $R_c/T$  decreases, that is as the bend becomes sharper, the maximum shear stress in the bend tends to increase. The bend shear stress ( $\tau_b$ ) is expressed in the following equation.

$$\tau_b = K_b \tau_d \quad 605-3$$

(FHWA, September 2005, HEC-15, Eq. 3.6, p. 3-12)

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/05114/05114.pdf>

where:

$\tau_b$	=	side shear stress on the channel, lb/ft <sup>2</sup>
$K_b$	=	ratio of channel bend to bottom shear stress (see <b>Equation 605-4</b> )
$\tau_d$	=	shear stress in channel at maximum depth, lb/ft <sup>2</sup>

$K_b$  is found using **Equation 605-4**, **Equation 605-5**, and **Equation 605-6**:

$$K_b = 2.00 \quad \text{for} \quad R_c / T \leq 2 \quad 605-4$$

$$K_b = 2.38 - 0.206 \left( \frac{R_c}{T} \right) + 0.0073 \left( \frac{R_c}{T} \right)^2 \quad \text{for} \quad 2 < R_c / T < 10 \quad 605-5$$

$$K_b = 1.05 \quad \text{for} \quad 10 \leq R_c / T \quad 605-6$$

(FHWA, September 2005, HEC-15, Eq. 3.7, p. 3-13)

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/05114/05114.pdf>

where:

$$\begin{aligned} R_c &= \text{radius of curvature of the bend to the channel centerline, ft} \\ T &= \text{channel top (water surface) width, ft} \end{aligned}$$

The increased shear stress caused by a bend persists downstream of the bend a distance of  $L_p$ . The length of protection ( $L_p$ ) required downstream of a bend is found using **Equation 605-7**.

$$L_p = 0.60 \left( \frac{R^{7/6}}{n} \right) \quad \text{605-7}$$

(FHWA, September 2005, HEC-15, Eq. 3.8, p. 3-13)

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/05114/05114.pdf>

where:

$$\begin{aligned} L_p &= \text{length of protection, ft} \\ R &= \text{hydraulic radius of the channel, ft} \\ n &= \text{Manning's roughness coefficient for lining material in the bend} \end{aligned}$$

A final consideration for channel design at bends is the increase in water surface elevation at the outside of the bend caused by the superelevation of the water surface. Additional freeboard ( $\Delta d$ ), required for bends, can be calculated using the following equation:

$$\Delta d = \frac{V^2 T}{g R_c} \quad \text{605-8}$$

(FHWA, September 2005, HEC-15, Eq. 3.9, p. 3-13)

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/05114/05114.pdf>

where:

$$\begin{aligned} \Delta d &= \text{additional freeboard required due to superelevation, ft} \\ V &= \text{average channel velocity, ft/s} \\ T &= \text{water surface top width, ft} \\ g &= \text{acceleration due to gravity, 32.2 ft/s}^2 \\ R_c &= \text{radius of curvature of the bend to the channel centerline, ft} \end{aligned}$$

**Table 605-1** gives permissible shear stresses for various channel lining materials. The computed maximum or bend shear stress must be less than the permissible shear stress for stable channel design. **Figure 605-3** and **Figure 605-4** show permissible shear stresses for non-cohesive and cohesive soils, respectively, in graph form.



**Table 605-1 Permissible Shear Stresses for Lining Materials**

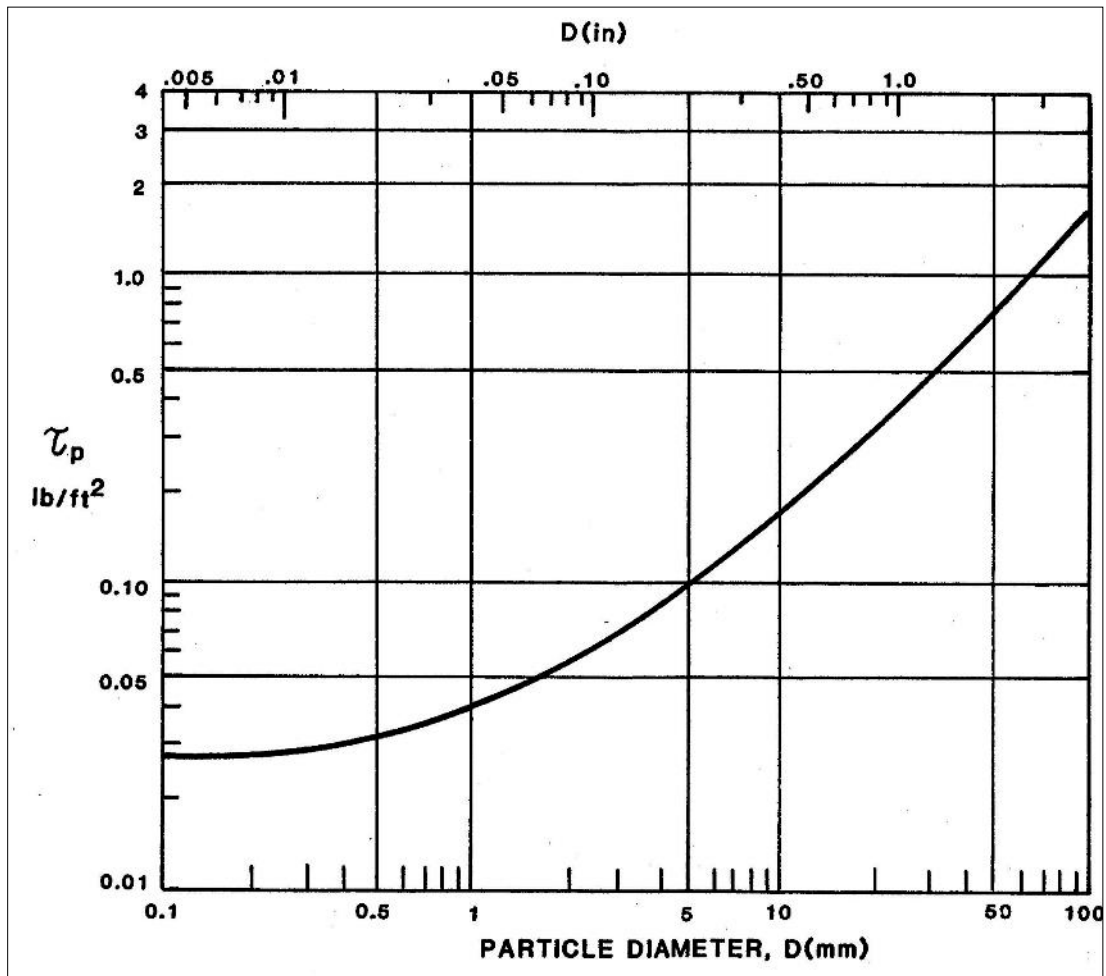
Source: FHWA, September 2005, HEC-15, Table 2.3, p. 2-7.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/05114/05114.pdf>

Lining Category	Lining Type	Permissible Unit Shear Stress (lb/ft <sup>2</sup> )
Bare Soil	Cohesive (PI = 10)	
	Clayey Sands	0.037-0.095
	Inorganic Silts	0.027-0.11
	Silty Sands	0.024-0.072
	Cohesive (PI ≤ 20)	
	Clayey Sands	0.094
	Inorganic Silts	0.083
	Silty Sands	0.072
	Inorganic Clays	0.14
	Non-Cohesive (PI <10)	
	Finer than coarse sand D <sub>75</sub> < 0.05 in (1.33 mm)	0.02
	Fine gravel D <sub>75</sub> = 0.3 in (7.5 mm)	0.12
	Gravel D <sub>75</sub> = 0.6 in (15 mm)	0.24
Gravel Mulch	D <sub>50</sub> = 1-inch (25 mm)	0.4
	D <sub>50</sub> = 2-inch (50 mm)	0.8
Rock Riprap	D <sub>50</sub> = 6-inch (0.15 m)	2.4
	D <sub>50</sub> = 12-inch (0.30 m)	4.8

NOTE: PI = Plasticity Index

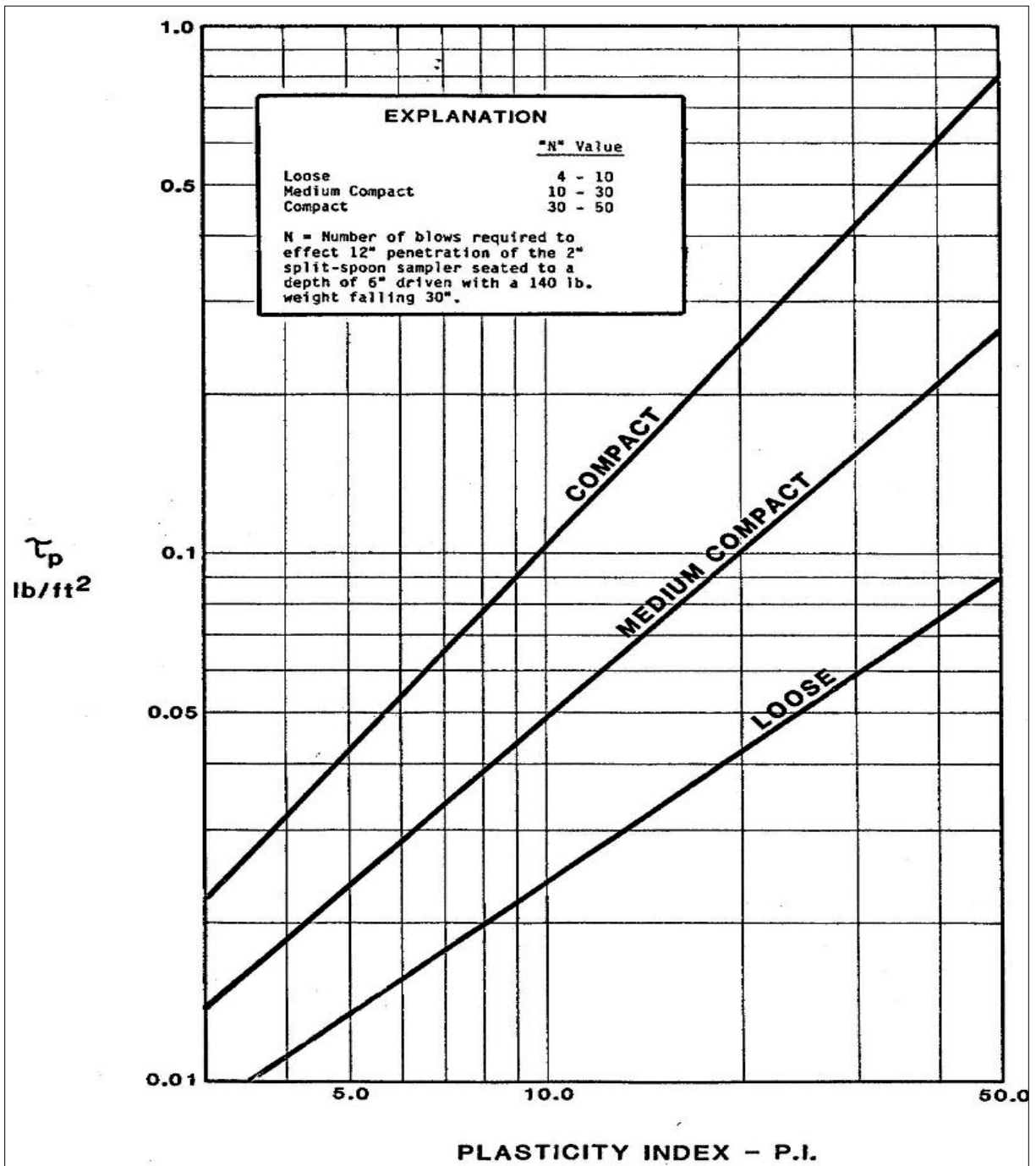
NOTE: Since vegetative and rolled erosion control product (RECP) lining performance relates to how well they protect the underlying soil from shear stresses these linings do not have permissible shear stresses independent of soil types. For information on vegetative and RECP linings, refer to Chapters 4 and 5, respectively, of HEC-15 (link provided above in table title).



Source: FHWA, April 1988, HEC-15, Chart 1, p. 49.

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hec/hec15si.pdf>

**Figure 605-3 Permissible Shear Stress,  $\tau_p$ , for Non-Cohesive Soils**



Source: FHWA, April 1988, HEC-15, Chart 2, p. 50.

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hec/hec15si.pdf>

Figure 605-4 Permissible Shear Stress,  $\tau_p$ , for Cohesive Soils

## Permissible Velocity

The permissible velocity approach is an alternative method to determining channel stability. The method was first developed in the 1920s, based on empirical data.

**Table 605-2** and **Table 605-3** list permissible velocities for grass and earth-lined channels, respectively. The permissible velocities listed in **Table 605-3** were compiled based on data from canals. Therefore, the values provided are applicable only to channels with mild slopes.

**Table 605-2 Permissible Velocities for Grass-Lined Channels**

Source: NRCS, August 2007, Part 654 National Engineering Handbook,  
Stream Restoration Design, Table 8-4, p. 8-7.

<https://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=17784.wba>

Cover	Slope Range Percent	Permissible Velocity (ft/s)	
		Erosion-Resistant Soils	Easily Eroded Soils
Bermuda grass	0-5	8	6
	5-10	7	5
	>10	6	4
Buffalo grass, Kentucky blue grass, smooth brome, blue grama	0-5	7	5
	5-10	6	4
	>10	5	3
Grass mixture	0-5	5	4
	5-10	4	3
	>10	Not recommended	Not recommended
Lespedeza sericea, weeping lovegrass, ischaemum (yellow bluestem), kudzu, alfalfa, crabgrass	0-5	3.5	2.5
	5-10	Not recommended	Not recommended
	>10	Not recommended	Not recommended
Annual – used on mild slopes or as temporary protection until permanent covers are established, common lespedeza, Sudangrass	0-5	3.5	2.5
	5-10	Not recommended	Not recommended
	>10	Not recommended	Not recommended

**Table 605-3 Permissible Velocities in Earth-Lined Channels**

Source: NRCS, August 2007, Part 654 National Engineering Handbook,  
Stream Restoration Design, Table 8-3, p. 8-6.

<https://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=17784.wba>

Material	Mean Velocity, for Straight Canals of Small Slope, After Aging, with Flow Depths Less Than 3 ft, (ft/s)		
	Clear Water, no detritus	Water transporting colloidal silts	Water transporting noncolloidal silts, sands, gravels, or rock fragments
Fine sand (noncolloidal)	1.50	2.50	1.50
Sandy loam (noncolloidal)	1.75	2.50	2.00
Silt loam (noncolloidal)	2.00	3.00	2.00
Alluvial silts (noncolloidal)	2.00	3.50	2.00
Ordinary firm loam	2.50	3.50	2.25
Volcanic ash	2.50	3.50	2.00
Stiff clay (very colloidal)	3.75	5.00	3.00
Alluvial silts (colloidal)	3.75	5.00	3.00
Shales and hardpans	6.00	6.00	5.00
Fine gravel	2.50	5.00	3.75
Graded, loam to cobbles (noncolloidal)	3.75	5.00	5.00
Graded silt to cobbles (when colloidal)	4.00	5.50	5.00
Coarse gravel (noncolloidal)	4.00	6.00	6.75
Cobbles and shingles	5.00	5.50	6.75

HEC-15 also provides an equation for permissible velocity ( $V_p$ ) based on permissible shear stress:

$$V_p = \frac{1.49}{n \sqrt{\gamma d}} R^{1/6} \tau_p^{1/2}$$

**605-9**

(FHWA, September 2005, HEC-15, Eq. 2.6, p. 2-6)

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/05114/05114.pdf>

where:

$V_p$	=	permissible velocity, ft/s
$n$	=	Manning's roughness coefficient
$\gamma$	=	specific weight of water, lb/ft <sup>3</sup>
$d$	=	flow depth, ft

$$\begin{aligned} R &= \text{hydraulic radius (area (A)/wetted perimeter (P)), ft} \\ \tau_p &= \text{permissible shear stress, lb/ft}^2 \end{aligned}$$

### Riprap Gradation and Thickness

Riprap is a common channel lining material used in New Mexico, and as such, special mention is made in the manual regarding the gradation and thickness. The gradation of dumped riprap should follow a smooth size distribution curve. Most riprap gradation ratios will fall in the range of  $D_{100} / D_{50}$  to  $D_{50} / D_{20}$  between 3.0 to 1.5. The most important criterion is a proper distribution of sizes in the gradation so that interstices formed by larger stones are filled with smaller stones in an interlocking fashion, preventing the formation of open pockets. These gradation requirements apply regardless of the type of filter design used. A geosynthetic fabric may be used as a filter rather than a graded gravel. Riprap stone must also meet the requirements of Section 603 Temporary Erosion & Sediment Control, of the NMDOT Standard Specifications for Highway and Bridge Construction (NMDOT, <http://dot.state.nm.us/content/nmdot/en/Standards.html>).

In general, riprap constructed with angular stones have the best performance. Flat slab-like stones should be avoided since they are easily dislodged by flow. Smooth round stones should be avoided if possible as the lack of angularity or fractured faces reduce the stability as compared to interlocking angular stones. An approximate guide to stone shape is that neither the breadth nor thickness of a single stone should be less than one-third its length.

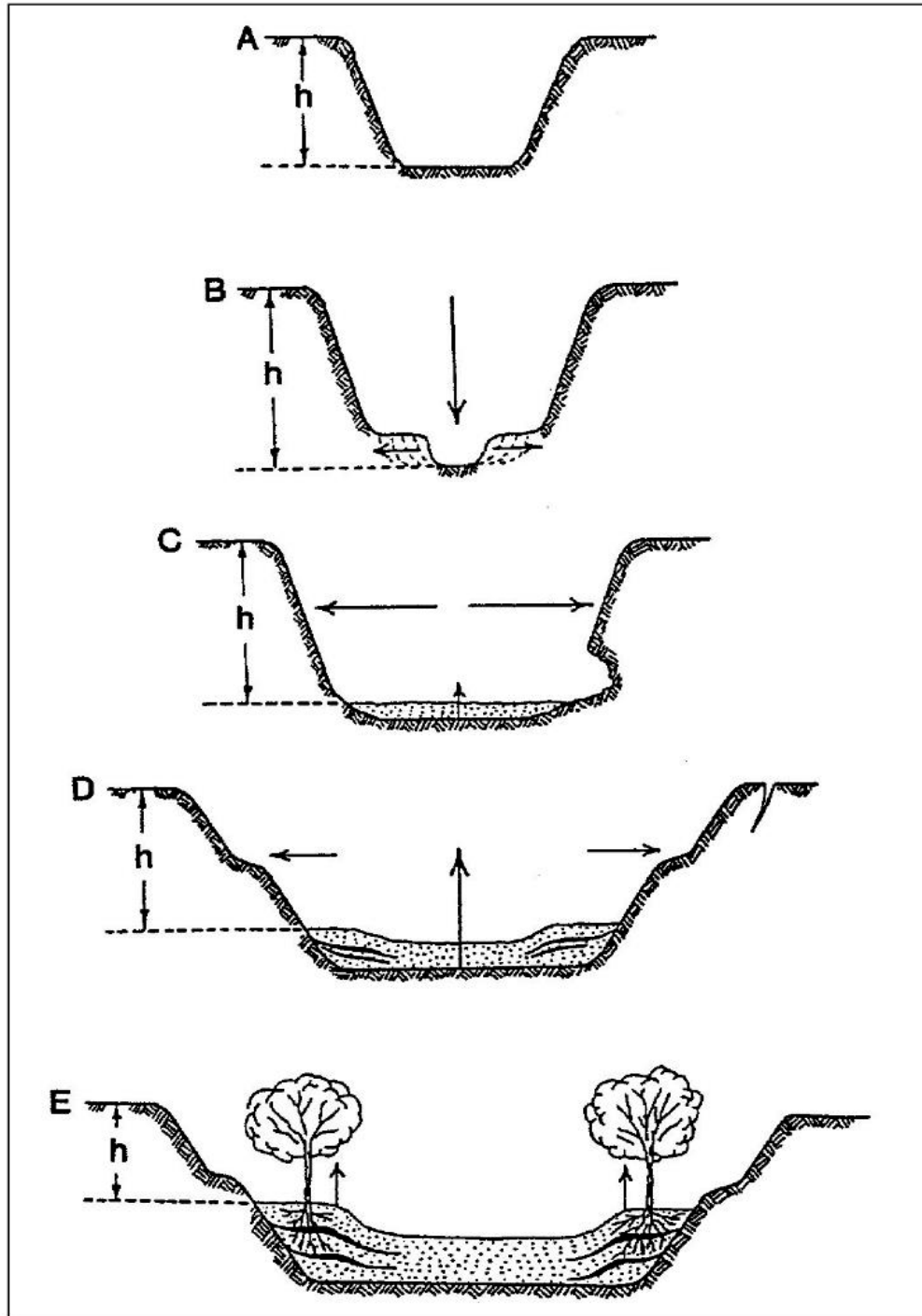
The thickness of a riprap lining should equal the diameter of the largest rock size in the gradation. For most gradations, this will mean a thickness from 1.5 to 3.0 times the mean riprap diameter ( $D_{50}$ ). For additional information on design of riprap lined channels refer to Chapter 6 of HEC-15 (FHWA, September 2005).

### 605.3 Evaluation of Incised Channels

A common technique for describing incised channel evolution is known as “location-for-time substitution”. This technique has led to the development of a five-stage geomorphic model of incised channel evolution for arroyos of the southwest from a state of non-equilibrium to a new state of dynamic equilibrium.

The five evolutionary stages are shown in **Figure 605-5**. Beginning with a given channel (**Figure 605-5A**), a change or disruption of the balance of forces in the channel causes down cutting (**Figure 605-5B**). As the channel begins to undercut the banks (**Figure 605-5C**), the width of the channel increases, and the channel bed accumulates some of the bank material. The channel continues to a point of maximum width where the bank slopes decrease (**Figure 605-5D**), eventually stabilizing over time (**Figure 605-5E**). For a more in-depth description of arroyo evolution described refer to HEC-20 (FHWA, April 2012).

The above process occurs over long periods of time in arid and semi-arid environments. Many arroyos in New Mexico exhibit all of these features somewhere along their length. Beginning at the head of an arroyo and proceeding downstream, each of the five stages shown in **Figure 605-5** may be identified. This is termed location-for-time substitution.



Source: FHWA, April 2012, HEC-20, Figure 2.2, Schumm, et al. (1984), p. 2.3.  
<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12004.pdf>

**Figure 605-5 Five-Stage Arroyo Evolution Model**

To classify a section of arroyo as being in a specific phase of evolution is difficult at best. A recent storm may have provided effects that resemble a certain phase of evolution, but these effects may exist for a relatively short time until the arroyo can readjust to its current evolutionary phase. Therefore, it is important to look at the entire arroyo and even adjacent arroyos for clues about the long-term dynamics of the arroyo in question.

Perhaps it is not as important to know precisely which phase of evolution the arroyo is in, as it is to know whether the reach of arroyo is aggrading or degrading from a sediment standpoint. This knowledge will have an impact on how and which drainage structures are used in the construction of highway projects. For example, a degrading arroyo reach may require grade control structures to prevent undermining or bypassing drainage structures. Similarly, aggrading reaches must account for sediment deposition when designing culvert or channel capacity.

**Figure 605-5** is a generalized guide for identifying aggrading and degrading channels. Detailed analyses are required to determine the actual channel dynamics.

#### **605.4 Analysis Procedures**

In HEC-20 (FHWA, April 2012), a three-level procedure for analyzing stream stability is presented, which will involve at least the first two of the following three levels of analyses:

- Level 1 - Application of simple geomorphic concepts and other qualitative analysis.
- Level 2 - Application of basic hydrologic, hydraulic, and sediment transport engineering concepts.
- Level 3 - Application of mathematical or physical modeling studies.

Refer to p. 4.1 of HEC-20 (FHWA, April 2012) for a detailed discussion on Level 1, 2, and 3 analyses.

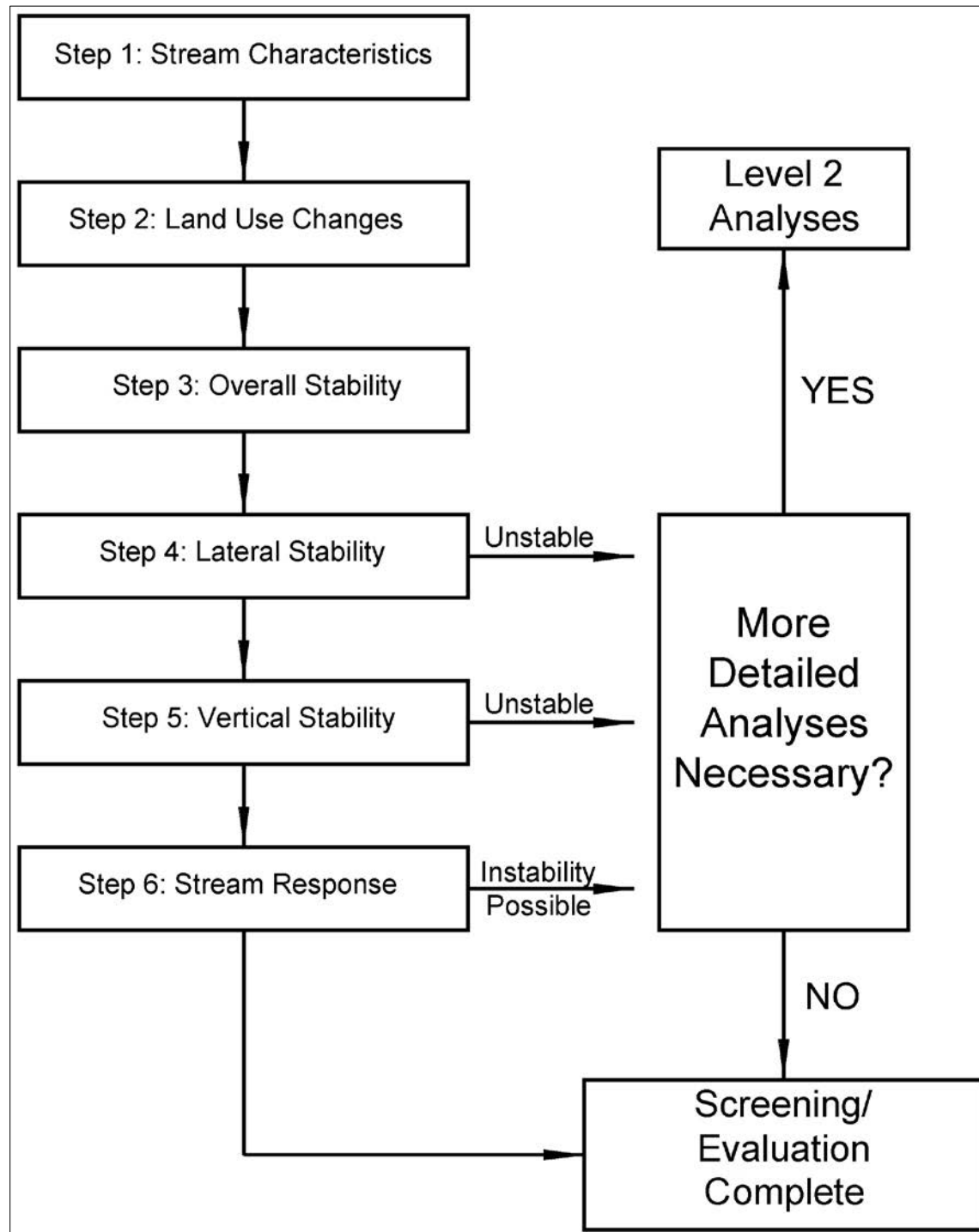
##### **605.4.1 Level 1: Qualitative Geomorphic Analyses**

A flow chart of the typical steps in qualitative geomorphic analyses is provided in **Figure 605-6**. The six steps are generally applicable to most stream stability problems. As shown in **Figure 605-6**, the qualitative evaluation leads to a conclusion regarding the need for more detailed (Level 2) analysis or a decision to complete a screening or evaluation based on the Level 1 analysis. A Level 1 qualitative analysis is a prerequisite for a Level 2 engineering analysis for bridge design, evaluation, and rehabilitation.

Refer to HEC-20 (FHWA, April 2012, p. 4.4 and Chapter 5) for detailed descriptions of the steps in the Level 1 analysis.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12004.pdf>





Source: FHWA, April 2012, HEC-20, Figure 4.1, p. 4.10.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12004.pdf>

**Figure 605-6 Flow Chart for Level 1: Qualitative Geomorphic Analyses**

### 605.4.2 Level 2: Basic Engineering Analyses

Data requirements for basic hydrologic, hydraulic, and sediment transport engineering analyses are dependent on the types of analysis that must be completed. Hydrologic data needed include dominant discharge (or bank full flow), flow duration curves, and flow frequency curves.

Hydraulic data needed include cross sections, channel and bank roughness estimates, channel alignment, and other data for computing channel hydraulics, up to and including water surface profile calculations. Analysis of basic sediment transport conditions requires information on land use, soils, geologic conditions, sediment sizes in the watershed and channel, and available measured sediment transport rates.

More detailed quantitative analyses require data on the properties of bed and bank materials and, at times, field data on bed load and suspended load transport rates. Properties of bed and bank materials that are important to a study of sediment transport include size, shape, fall velocity, cohesion, density, and angle of repose.

In a Level 3 analysis, the application of mathematical and physical model studies requires the same basic data as a Level 2 analysis, but typically in much greater detail. For example, water and sediment routing by mathematical models (e.g., HEC-RAS) and construction of a physical model both require detailed channel cross sectional data. The more extensive data requirements for either mathematical or physical model studies, combined with the additional level of effort needed to complete such studies, results in a relatively large effort.

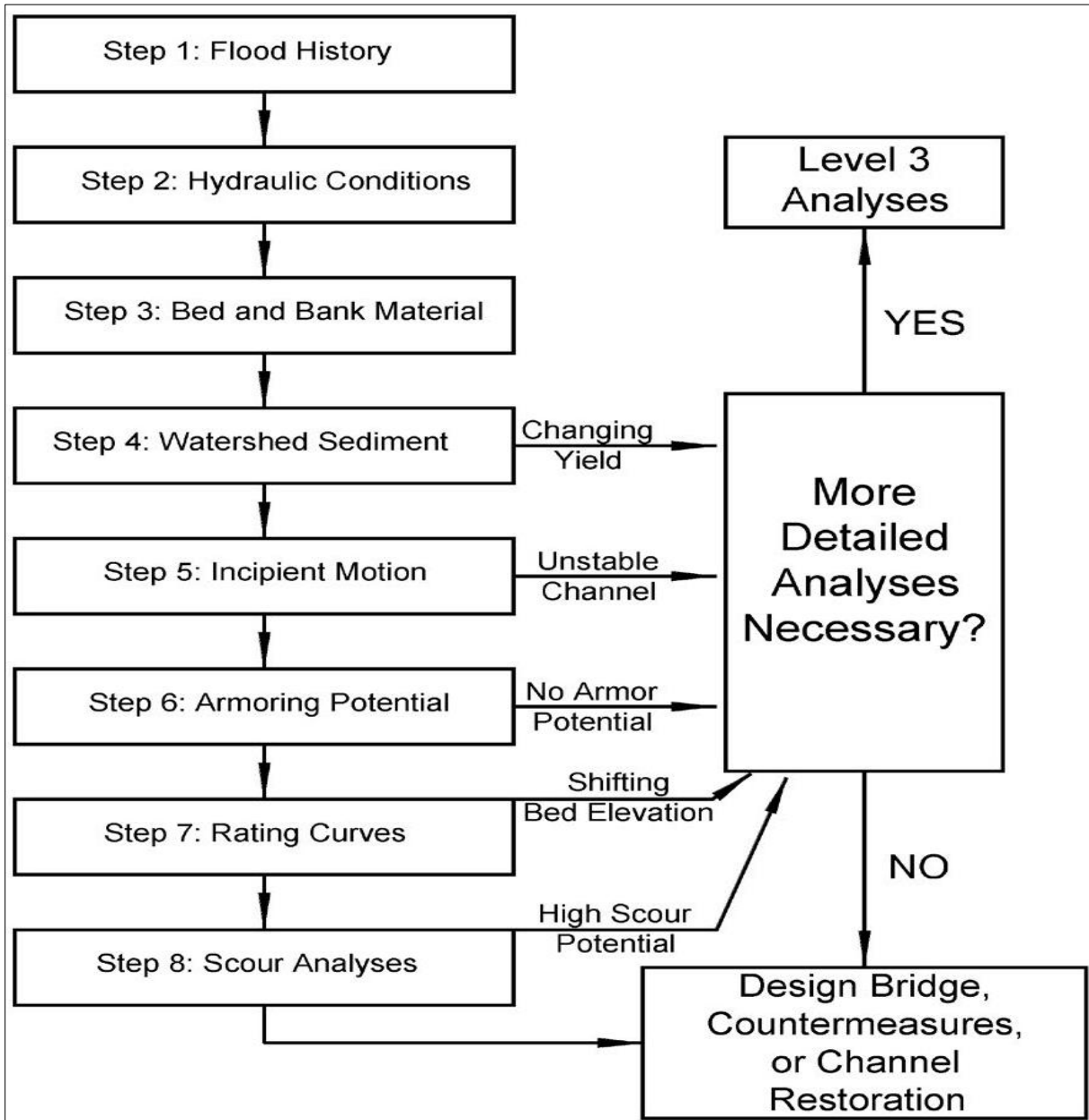
A flow chart of the typical steps required in basic engineering analyses is provided in **Figure 605-7**. The flow chart illustrates the typical steps to be followed if a Level 1 qualitative analysis resulted in a decision that Level 2 analyses are required. The eight basic engineering steps are generally applicable to most stream stability problems and is discussed in more detail in the paragraphs that follow. The basic engineering analysis steps will lead to a conclusion regarding the need for more detailed Level 3 analysis or a decision to proceed to the selection and design of countermeasures without more complex studies. Selection of countermeasures are discussed in **Section 608**. For design of countermeasures refer to HEC-23, "Bridge Scour and Stream Instability Countermeasures" (FHWA, September 2009) for information, design steps, and equations.

FHWA, September 2009, "HEC-23, Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance, Third Edition, Volume 1".

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/09111/09111.pdf>

FHWA, September 2009, "HEC-23, Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance, Third Edition, Volume 2".

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/09111/09112.pdf>



Source: FHWA, April 2012, HEC-20, Figure 4.4, p. 4.17.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12004.pdf>

**Figure 605-7 Flow Chart for Level 2: Basic Engineering Analyses**

### **Step 1 Evaluate Flood History and Rainfall-Runoff Relations**

Consideration of flood history is an integral step in attempting to characterize watershed response and morphologic evolution. Analysis of flood history is of particular importance to understanding arid region stream characteristics. Many dry land streams flow only during the spring and immediately after major storms. For example, Leopold et al. (1966) found that

arroyos near Santa Fe, New Mexico, flow only about three times a year. As a consequence, dryland stream response can be considered to be more hydrologically dependent than streams located in humid environments. Whereas, the simple passage of time may be sufficient to cause change in a stream located in a humid environment, time alone, at least in the short term, may not necessarily cause change in a dry land system due to the infrequency of hydrologically significant events. Thus, the absence of significant morphological changes in a dryland stream or river, even over a period of years, should not necessarily be construed as indicative of system stability.

The occurrence of single large storms can often be directly related to system change in any region of the country, however, that is not always the case. In particular, the succession of morphologic change may be linked to the concept of geomorphic thresholds as proposed by Schumm (1977). Although a single major storm may trigger an erosional event in a system, the occurrence of such an event may be the result of a cumulative process leading to an unstable geomorphic condition.

Where available, the study of flood records and corresponding system responses, as indicated by time-sequenced aerial photography or other physical information, may help determine the relationship between morphological change, flood magnitude, and frequency. Evaluation of wet-dry cycles can also be beneficial to understanding historical system responses. Observable historical changes may be better correlated with the occurrence of a sequence of events during a period of above-average rainfall and runoff than with a single large event. For example, a large storm preceded by a period of above-average precipitation may result in less erosion, (due to better vegetative cover), than a comparable storm occurring under dry antecedent conditions; however, runoff volumes might be greater due to saturated soil conditions.

A good method for evaluating wet-dry cycles is to plot annual rainfall amounts, runoff volumes, and maximum annual mean daily discharge for the period of record. A comparison of these graphs will provide insight to wet-dry cycles and flood occurrences. Additionally, a plot of the ratio of rainfall to runoff is a good indicator of watershed characteristics and historical changes in watershed condition.

## **Step 2      Evaluate Hydraulic Conditions**

Knowledge of basic hydraulic conditions, such as velocity, flow depth, and top width, for a given flood event is essential for the completion of Level 2 stream stability analysis. Incipient motion analysis, scour analysis, and assessment of sediment transport capacity all require basic hydraulic information. Hydraulic information is sometimes required for both the main channel and overbank areas, such as in the analysis of contraction scour.

For many river systems, particularly near urban areas, hydraulic information may be readily available from previous studies, such as flood insurance studies, channel improvement projects, etc., and therefore, complete re-analysis may not be necessary. However, in other areas, hydraulic analyses based on appropriate analytical techniques will be required prior to completing other quantitative analyses in a Level 2 stream stability assessment. The most common computer models for the analysis of water surface profiles and hydraulic conditions are one-dimensional and two-dimensional models.

### **Step 3      Bed and Bank Material Analysis**

Bed material is the sediment mixture of which streambeds are composed. Bed material can range in size from boulders to fine clay particles. The erodibility or stability of a channel largely depends on the size of the particles in the bed. Additionally, knowledge of bed sediment material is necessary for most sediment transport analyses, including evaluation of incipient motion, armoring potential, sediment transport capacity, and scour calculations. Many of these analyses require knowledge of particle size gradation, and not just the median size.

Of the various sediment properties, particle size has the greatest significance to the hydraulic engineer, not only because size is the most readily measured property, but also because other properties, such as shape and fall velocity tend to vary with particle size. A comprehensive discussion of sediment characteristics, including sediment size and its measurement, is provided in "Highways in the River Environment" (FHWA, February 1990). The following information briefly describes sediment sampling considerations.

Important factors to consider in determining where and how many bed and bank material samples to collect include: 1) size and complexity of the study area; 2) number, lengths and drainage areas of tributaries; 3) evidence of or potential for armoring; 4) structural features that can impact or be significantly impacted by sediment transport; 5) bank failure areas; 6) high bank areas; and 7) areas exhibiting significant sediment movement or deposition. Tributary sediment characteristics can be important to channel stability, since a single major tributary or tributary source area can be the predominant supplier of sediment to a system.

The depth of bed material sampling depends on the homogeneity of surface and subsurface materials. Where possible, it is desirable to dig down to some depth to establish bed material characteristics. For example, in sand/gravel bed systems, the potential existence of a thin surface layer of coarser sediments (armor layer) on top of relatively undisturbed subsurface material must be considered in any sediment sampling. A single sample containing material from both layers would contain materials from two populations in unknown proportions, and thus it is typically more appropriate to sample each layer separately. If the purpose of the sampling is to evaluate hydraulic friction or initiation of bed movement, then the surface sample will be of most interest. Conversely, if bed material transport during a large flood (i.e., large enough to disturb the surface layer) is important, then the underlying layer may be more significant.

### **Step 4      Evaluate Watershed Sediment Yield**

Evaluation of watershed sediment yield, and in particular the relative increase in yield as a result of some disturbance, can be an important factor in stream stability assessment. Sediment eroded from the land surface can cause silting problems in stream channels resulting in increased flood stage and damage. Conversely, a reduction in sediment supply can also cause adverse impacts to river systems by reducing the supply of incoming sediment thus promoting channel degradation and head-cutting. A radical change in sediment yield as a result of some disturbance, such as a recent fire or long-term land use changes, would suggest that stream instability conditions either already exist or might readily develop.

Assessment of watershed sediment yield first requires understanding the sediment sources in the watershed and the types of erosion that are most prevalent. The physical processes causing erosion can be classified as sheet erosion, rilling, gullying, and stream channel erosion. Other types of erosional processes are classified under the category of mass movement and include

soil creep, mudflows, and landslides. Data from publications and maps produced by the NRCS and the USGS (1982) can be used along with field observations to evaluate the area of interest.

USGS, 1982, "Measurement and Computation of Streamflow".

<http://pubs.usgs.gov/wsp/wsp2175/>

Actual quantification of sediment yield is at best an imprecise science. The most useful information is typically obtained not from analysis of absolute magnitude of sediment yield, but rather from the relative changes in yield as a result of a given disturbance. One useful approach to quantifying sediment yield, based on regression equations, is the Revised Universal Soil Loss Equation 2 (RUSLE2) (NRCS, 2014). The RUSLE2 equation is an empirical formula for predicting annual soil loss due to sheet and rill erosion and is perhaps the most widely recognized method for predicting soil erosion. The United States Department of Agriculture (USDA), Natural Resource Conservation Service (NRCS) provides detailed descriptions of this equation and its terms (NRCS, 2014).

NRCS, 2014, "Revised Universal Soil Loss Equation 2 (RUSLE2)".

<http://www.ars.usda.gov/Research/docs.htm?docid=6028>

## **Step 5 Incipient Motion Analysis**

An evaluation of relative channel stability can be made by evaluating incipient motion parameters. The definition of incipient motion is based on the critical or threshold conditions where hydrodynamic forces acting on one particle of sediment have reached a value that, if increased even slightly, will move the particle. Under critical conditions, or at the point of incipient motion, the hydrodynamic forces acting on the particle are just balanced by the resisting forces of the particle.

Evaluation of the incipient motion size for various discharge conditions provides insight on channel stability and the magnitude of the flood that might potentially disrupt channel stability. The results of such analysis are generally more useful for analysis of gravel or cobble-bed systems. When applied to a sand bed channel, incipient motion results usually indicate that all particles in the bed material are capable of being moved for even very small discharges. Shields diagram (not shown here) may be used to evaluate the particle size at incipient motion for a given discharge. For most river flow conditions, **Equation 606-1** is appropriate for the evaluation of incipient motion and was derived from Shields diagram.

## **Step 6 Evaluate Armoring Potential**

The armoring process begins as the non-moving coarser particles segregate from the finer material during transport. The coarser particles remain in the bed, where they accumulate in a sublayer. Fine bed material is leached up through this coarse sublayer to augment the material in transport. As sediment movement continues and degradation progresses, an increasing number of non-moving particles accumulate in the sublayer. Eventually, enough coarse particles accumulate to shield or "armor" the entire bed surface.

An armor layer, sufficient to protect the bed against moderate discharges, can be disrupted during high flow but may be restored as the flow diminishes. Therefore, as in any hydraulic design, the analysis must be based on a certain design event. If the armor layer is stable for that

design event, it is reasonable to conclude that no degradation will occur under design conditions. However, flows exceeding the design event may disrupt the armor layer, resulting in degradation.

Potential for development of an armor layer can be assessed using incipient motion analysis and a representative bed material composition. In this case, the representative bed material composition is that which is typical of the depth of anticipated degradation. For given hydraulic conditions, the incipient motion particle size can be computed as shown above in Step 5. If no sediment of the computed size or larger is present in significant quantities in the bed, armoring will not occur.

The  $D_{90}$  or  $D_{95}$  size of the representative bed material is frequently found to be the size "paving the channel" when degradation has ceased. Within practical limits of planning and design, the  $D_{95}$  size is considered to be about the maximum size for pavement formation (Gessler, 1971). Therefore, armoring is probable when the computed incipient motion size is equal to or smaller than the  $D_{95}$  size in the bed material.

By observing the percentage of the bed material equal to or larger than the armor particle size ( $P_c$ ), the depth of scour ( $y_s$ ) necessary to establish an armor layer can be calculated by:

$$y_s = y_a \left( \frac{1}{P_c} - 1 \right) \quad 605-10$$

(FHWA, April 2012, HEC-20, Eq. 6.16, p. 6.28)

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12004.pdf>

where:

$y_s$	=	depth of scour necessary to establish an armor layer, ft
$y_a$	=	thickness of armoring layer, ft
$P_c$	=	decimal fraction of material coarser than the armoring particle size

The thickness of the armoring layer ranges from one to three times the armor particle size ( $D_c$ ) depending on the value of  $D_c$ . Field observations suggest that a relatively stable armoring condition requires a minimum of two layers of armoring particles.

## Step 7 Evaluate Rating Curve Shifts

When stream gage data is available, such as that collected by the USGS, an analysis of the stage-discharge rating curve over time can provide insight on stream stability. For example, a rating curve that was stable for many years, but suddenly shifts might indicate a change in watershed conditions causing increased channel erosion or sedimentation, or some other change related to channel stability. Similarly, a rating curve that shifts continually would be a good indicator that channel instability exists. However, it is important to note that not all rating curve shifts are the result of channel instability. Other factors promoting a shift in a rating curve include changes in channel vegetation, ice conditions, and beaver activity.

The most common cause of rating curve shifts in natural channel control sections is generally scour and fill (USGS, 1982). A positive shift in the rating curve results from scour and depth, and hence, the discharge is increased for a given stage. Conversely, a negative shift results from fill and the depth and the discharge will be less for a given stage.



Shifts may also be the result of changes in channel width. Channel width may increase due to bank-cutting or decrease due to undercutting of steep streambanks. In meandering streams, changes in channel width can occur as point bars are created or destroyed.

Analysis of rating curve shifts is typically available from the agency responsible for the stream gage. If such information is not available, field inspection combined with the methods described by USGS (1982) can be utilized to analyze observed rating curve shifts. If the shifts can be traced to scour, fill, or channel width changes, such information will be a reliable indicator of potential channel instability.

Gaging stations at which continuous sediment data are collected may also provide clues to the existence of gradation problems. Any changes in the long-term sediment load may indicate lateral movement of the channel, gradation changes, or a change in sediment supply from the watershed.

## **Step 8      Evaluate Scour Conditions**

**Section 607** provides an overview of scour at bridge crossings and Hydrologic Engineering Circular No. 18 (HEC-18), "Evaluating Scour at Bridges," (FHWA, April 2012) provides detailed computational procedures. These problems are attributed to the effects of obstructions in the flow (local scour) and contraction of the flow or channel deepening at the outside of a bend. Calculation of the three components of scour, local scour, contraction scour, and aggradation/degradation quantifies the potential instability at a bridge crossing. Scour-susceptible bridges are those that have potentially large depths of any one of the scour components, and/or the cumulative depth of more than one scour component is large. These bridges should be carefully monitored and/or countermeasures should be installed.

The engineer is referred to HEC-20 (FHWA, April 2012), Chapter 6 for detail on the Level 2 Analyses.

### **605.4.3    Level 3: Mathematical and Physical Model Studies**

Detailed evaluation and assessment of stream stability can be accomplished using either mathematical or physical model studies. A mathematical model is simply a quantitative expression of the relevant physical processes involved in stream channel stability. Various types of mathematical models are available for evaluation of sediment transport depending on the application (watershed or channel analysis) and the level of analysis required. These models can provide detailed information on erosion and sedimentation throughout a study reach, and allow for evaluation of a variety of "what-if" scenarios. HDS-7, "Hydraulic Design of Safe Bridges" (FHWA, April 2012) provides a survey of 1- and 2-dimensional mathematical models available for alluvial river analyses. HEC-18, "Evaluating Scour at Bridges, Fifth Edition" (FHWA, April 2012) summarizes the capabilities of 1- dimensional and 2- dimensional mathematical models for unsteady flow tidal hydraulic analyses.

FHWA, April 2012, "HDS-7, Hydraulic Design of Safe Bridges".

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12018.pdf>

FHWA, April 2012, "HEC-18, Evaluating Scour at Bridges, Fifth Edition".

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12003.pdf>



Similarly, physical model studies completed in a hydraulics laboratory can provide detailed information on flow conditions and, to some extent, sediment transport conditions at a bridge crossing. The hydraulic laws and principles involved in scaling physical models are well-defined and understood allowing for accurate extrapolation of model results to prototype conditions. Physical model studies can sometimes provide better information on complex flow conditions than mathematical models due to the complexity of the process and the limitations of 2-dimensional and 3-dimensional mathematical models. Often the use of both physical and mathematical models can provide complementary information (see FHWA, HDS-6, December 2001).

FHWA, December 2001, "HDS-6, River Engineering for Highway Encroachments, Highways in the River Environment".

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/nhi01004.pdf>

The need for detailed information and accuracy from using either mathematical or physical models must be balanced by the time and resources available. As the analysis becomes more complicated, accounting for more factors, the level of effort necessary becomes proportionally larger. The decision to proceed with a Level 3 type analysis has historically been made only for high-risk locations, extraordinarily complex problems, or for forensic analysis where losses and liability costs are high. However, the importance of stream stability to the safety and integrity of all bridges suggests that Level 3 type analyses should be considered more routinely. The widespread use of personal computers and the continued development of more sophisticated software have greatly facilitated completion of Level 3 type investigations and have reduced the level of effort and cost required.

## **605.5 References**

FHWA, April 1988, "HEC-15 Design of Roadside Channels with Flexible Linings".

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hec/hec15si.pdf>

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## 606 Sediment Transport

### General Discussion

Sediment transport is a complex process, and as a result, a large body of engineering research and many technical publications have been developed to assist in the prediction of the physical processes involved. This section includes discussion, equations, and further references regarding the following types of sediment analyses:

- Aggradation and degradation
- Incipient motion
- Sediment continuity
- Equilibrium slope
- Sediment transport

Aggradation or degradation of a watercourse is the end result of the physical processes related to the last four topics listed above. Sediment transport analyses are best performed on a watercourse that has been delineated into reaches. A reach is a specified length along the watercourse, whereby each reach is delineated based on similar characteristics such as slope, cross section, roughness, vegetation, bed and bank soils, and bed armoring, if present. Reach lengths should be somewhat equal, particularly for manual sediment continuity analyses.

Sediment particles will begin movement after the hydrodynamic forces exceed the threshold of movement condition (incipient motion). Sediment continuity in and out of a stream reach will determine if a reach will aggrade, degrade or be near equilibrium (have an equilibrium slope). If sediment inflow into a reach exceeds the sediment transport capacity of the reach, then the sediment load in excess of the transport capacity will deposit (aggrade). If sediment inflow into a reach is less than the transport capacity of the reach, then the reach will erode or degrade until the sediment transport capacity is achieved. If the sediment inflow to a reach equals the transport capacity of the reach, then aggradation nor degradation will occur, so the reach may be in or near equilibrium. The channel slope would therefore be at an equilibrium slope.

Sediment transport capacity may be computed by various equations. Equation selection is dependent on the range of parameters for which the equation was developed and should correlate with the range of parameters present in the watercourse. Sediment transport capacity may be computed by manual methods, spreadsheets, or by computer programs such as HEC-RAS. Sediment transport analyses should be completed for the entire hydrograph. The hydrograph may be discretized (horizontal hydrograph segments assumed to represent an average discharge for a given time segment). Note that for each segment discharge, an average velocity and top width may be estimated from the hydraulic analysis results. The total sediment transport capacity is the summation of the transport capacity of the hydrograph segments. Sediment transport analysis for the entire watercourse and hydrograph may be simulated with a HEC-RAS quasi-steady flow model.

### 606.1 Aggrading and Degrading Channels

At any given time, a perennial stream or an arroyo channel reach may be in any one of three states: aggrading, degrading, or stable. Changes in bed level elevations may be obtained from comparison to previous surveys, photographs, and as-built drawings.

Perennial streams and river channels are generally in equilibrium or stable over a long time. Whereas evidence shows, ephemeral channels or arroyos in New Mexico may experience several episodes of down cutting and filling over time spans as short as one hundred years.

The following are some general indicators to help determine if a perennial stream or arroyo channel reach is aggrading, degrading, or is in equilibrium (stable). These are general indicators of an extremely complex process. The engineer is advised to review the referenced literature for an in-depth discussion of the mechanisms at work during each of these unique and complex processes (FHWA's HEC-18, April 2012; HEC-20, April 2012; and Highways in the River Environment, February 1990).

Aggrading Channel Reaches May Exhibit the Following Characteristics:

- Braided streams
- Sloping banks or low vertical banks down to the water surface or low flow bank
- Uniform sandy/silty bed material present in the channel bottom
- High width-to-depth ratio (generally > 10)
- Fan-shaped alluvial deposition which is an indication of an aggrading reach of channel
- Channel-widening over time, indicates an aggrading or vertically stable channel
- Channel slopes less than the average valley slope are usually aggrading reaches of channel
- Aggrading channels are dominated by early seral-stage herbaceous plant species such as Russian thistle (tumbleweed), kochia (sununer cyprus), and mustard species

Degrading Channel Reaches May Exhibit the Following Characteristics:

- An incised channel is probably a degrading or degraded channel
- Steep, near vertical banks, with evidence of recent bank failure, or sloughing into the channel
- Head-cutting is an obvious indication of channel degradation. The head-cut progresses in the upstream direction
- Channels getting deeper and narrower over time indicate degrading channels
- Channel slopes which are greater than the average valley slope is usually degrading reaches of channel
- Slowly degrading channels or channels just starting to degrade are characterized by a dominance of woody plant species, cacti, and native bunch grasses such as apache plume, big sage, chamisa, and cottonwood
- Rapidly degrading channel reaches are generally void of vegetation

## **606.2 Incipient Motion Analysis**

The FHWA document “Stream Stability at Highway Structures,” (HEC-20) states that an evaluation of relative channel stability can be made by evaluating incipient motion parameters. The definition of incipient motion is based on the critical or threshold conditions where hydrodynamic forces acting on one particle of sediment have reached a value that, if increased even slightly, will move the particle. Under critical conditions, or at the point of incipient motion, the hydrodynamic forces acting on the particle are just balanced by the resisting forces of the particle.

Shields diagram (Henderson F.M. 1966, Fig. 10-3, p. 413) may be used to evaluate the particle size ( $D_c$ ) at incipient motion for a given discharge. For most river flow conditions, the following equation, derived from Shields diagram, is appropriate for the evaluation of incipient motion:

$$D_c = \frac{\tau_o}{K_s (\gamma_s - \gamma)} \quad 606-1$$

(FHWA, April 2012, HEC-20, Eq. 6.13, p. 6.26)

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12004.pdf>

where:

$D_c$	=	diameter of sediment particle at incipient motion, ft
$\gamma_s$	=	specific weight of sediment particle, lb/ft <sup>3</sup>
$\gamma$	=	specific weight of water, lb/ft <sup>3</sup>
$\tau_o$	=	boundary shear stress, lb/ft <sup>2</sup>
$K_s$	=	Shields parameter (see below)

As originally proposed, the Shields parameter is 0.06 for flow conditions in the turbulent range. The value of 0.047 for sand sizes was suggested by Meyer-Peter and Mueller (1948) and further supported by Gessler (1971). Recent research has indicated that this coefficient is not constant (values range from 0.03 to 0.10). Equations have been derived as a function of surface and subsurface particle size. However, as a first estimate, 0.047 should provide reasonable results in most situations. Lower values (0.03) are commonly used for gravel and cobble sizes.

For gradually varied flow, the boundary shear stress ( $\tau_o$ ) in **Equation 606-1** is given by **Equation 605-1** ( $\tau = \gamma R S$ ). When significant bed forms are present in the flow, **Equation 605-1** may overestimate the shear stress acting on individual sediment particles. In this case, the boundary shear stress ( $\tau_o$ ) is given by the equation:

$$\tau_o = \frac{\rho V^2}{[5.75 \log (12.27 \frac{y_o}{k_s})]^2} \quad 606-2$$

(AMAFCA, November 1994, Sediment and Erosion Design Guide, Eq. 3.26, p. 3-26)

[http://www.bernco.gov/uploads/FileLinks/cbb8af72471e4746ba9e92e9a67318bb/Sediment\\_and\\_Erosion\\_Design\\_Guide\\_AMAFCA\\_.pdf](http://www.bernco.gov/uploads/FileLinks/cbb8af72471e4746ba9e92e9a67318bb/Sediment_and_Erosion_Design_Guide_AMAFCA_.pdf)

where:

$\tau_o$	=	boundary shear stress, lb/ft <sup>2</sup>
$V$	=	average velocity in the channel, ft/s
$k_s$	=	characteristic grain size (usually taken as 3.5 $D_{84}$ of the bed material), ft (to convert $D_{84}$ from mm to ft, divide by 304.8)
$y_o$	=	flow depth, ft
$\rho$	=	fluid density, 1.94 slug/ft <sup>3</sup>

Evaluation of the incipient motion size for various discharge conditions provides insight on channel stability and what flood might potentially disrupt channel stability. The results of such an analysis are generally more useful for analysis of gravel or cobble-bed systems. When applied

to a sand-bed channel, incipient motion results usually indicate that all particles in the bed material are capable of being moved for even very small discharges.

### 606.3 Sediment Continuity

Whether a channel or channel reach is aggrading or degrading can be determined by evaluating the sediment inflow to the reach and comparing it to the sediment outflow from the reach. Expressed qualitatively, where sediment inflow exceeds the sediment outflow, aggradation occurs. Conversely, degradation occurs where sediment outflow exceeds the inflow. This concept can be expressed mathematically as follows:

$$\Delta SV = SV_{IN} - SV_{OUT}$$

**606-3**

(FHWA, April 2012, HEC-20, Eq. 6.26, p. 6.35)

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12004.pdf>

where:

$\Delta SV$	=	change in sediment volume in the reach (volume of sediment stored or eroded), ft <sup>3</sup>
$SV_{IN}$	=	sediment volume transported into the reach, ft <sup>3</sup>
$SV_{OUT}$	=	sediment volume transported out of the reach, ft <sup>3</sup>

A reach is a segment of stream length developed to assist in sediment continuity and sediment transport analyses. A reach should be delineated to have relative similar characteristics such as cross section shape, profile slope and bed form, and roughness coefficients. Reach lengths should be relatively equal to avoid erroneous results when computing sediment volumes with **Equation 606-3**.

### 606.4 Equilibrium Slope Analysis

For clear-water releases of flow from dams or detention ponds, the channel immediately downstream would be expected to degrade until the reduction in slope results in a boundary shear stress too low to entrain the bed material. In a sand bed channel, the channel slope would be extremely mild to reach incipient motion conditions. Or in other words, sand is transported easily after reaching incipient motion conditions. For a gravel bed channel, channel degradation would also occur. In addition to the reduction in slope, the formation of a “pavement” could arrest degradation. Depending on the bed and bank materials, the degrading channel can narrow as it deepens, or the banks can become unstable and the channel can widen. Channel widening temporarily replenishes sediment supply.

For the case of no sediment supply from upstream, combining the incipient motion relation **Equation 606-1** and the Manning Equation (**Equation 502-1**), results in an equation to estimate the equilibrium slope ( $S_{eq}$ ) where bed material movement ceases, as follows:

$$S_{eq} = \left[ K_s D_c \left( \frac{\gamma_s - \gamma}{\gamma} \right) \right]^{(10/7)} \left( \frac{K_u}{q n} \right)^{(6/7)} \quad \text{606-4}$$

(FHWA, April 2012, HEC-20, Eq. 6.17, p. 6.29)

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12004.pdf>

where:

$S_{eq}$	=	channel slope at which particles $D_c$ will no longer move, ft/ft
$q$	=	channel discharge per unit width, cfs/ft
$K_s$	=	Shields parameter
$K_u$	=	1.486 coefficient for English units
$\gamma_s$	=	specific weight of sediment particle, lb/ft <sup>3</sup>
$\gamma$	=	specific weight of water, lb/ft <sup>3</sup>
$n$	=	Manning's roughness coefficient
$D_c$	=	critical bed material size, ft

This relationship assumes that the channel width remains constant for future conditions. The critical size, ( $D_c$ ) used in this equation, should be  $D_{90}$  because the bed will coarsen as degradation occurs.

Another approach to determining an equilibrium slope under conditions of no upstream sediment supply is presented by the USBR using the Meyer-Peter Muller Equation for the beginning of transport (USBR, 1984). Adjustment of the hydraulic depth due to the reduction in channel slope is included in the USBR Equation:

$$S_{eq} = K_u \frac{(D_{50})^{10/7} n^{9/7}}{(D_{90})^{5/14} q^{9/7}} \quad \text{606-5}$$

(FHWA, April 2012, HEC-20, Eq. 6.18, p. 6.29)

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12004.pdf>

where:

$K_u$	=	60.1 for English units
all other terms defined previously		

The degradation computed from the reduction in slope could result in channel narrowing or bank failure and channel widening. Also, the appropriate discharge for use in the equation is difficult to select. A range of discharges are responsible for forming the channel. Given long periods of time, extreme discharges would ultimately be responsible for forming the channel under these conditions. An initial estimate of the clear-water condition is to use the bankfull discharge recognizing that as the channel degrades the dimensions will adjust.

A more typical situation involves a reduction in sediment supply. In this case, the equilibrium slope can be predicted using sediment transport relationships. A reduction in sediment supply or an increase in discharge can cause a reduction in channel slope and degradation. The new equilibrium slope will produce hydraulic conditions where the channel sediment transport capacity matches the upstream sediment supply. This procedure can be performed using

sediment transport equations directly or through simplified relationships. A detailed discussion of available sediment transport equations is presented in HDS-6 (FHWA, 2001).

The following equation is often useful to develop a sediment transport capacity relationship for a river reach in the form of:

$$q_s = a V^b Y^c \quad 606-6$$

(FHWA, April 2012, HEC-20, Eq. 6.19, p. 6.30)

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12004.pdf>

where:

$q_s$	=	sediment transport capacity per unit width, cfs/ft
$V$	=	channel average velocity, ft/s
$Y$	=	channel average depth, ft
$a, b, c$	=	coefficient and exponents

The coefficient and exponents ( $a, b, c$ ) can be determined from fitting **Equation 606-6** to observed data or to a sediment transport equation appropriate to the stream conditions. If the coefficient and exponents are fit to Yang's sediment transport equation for sand (FHWA, 2001) and Yang (1996), reasonable results (generally within 25 percent) are produced by the following equations. In English units the coefficients are:

$$a = 0.025 n^{(2.39 - 0.8 \log(D_{50}))} (D_{50} - 0.07)^{-1.4} \quad 606-7$$

$$b = 4.93 - 0.74 \log(D_{50}) \quad 606-8$$

$$c = -0.46 + 0.65 \log(D_{50}) \quad 606-9$$

(FHWA, April 2012, HEC-20, Eqs. 6.20 – 6.22, p. 6.30)

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12004.pdf>

where:

$D_{50}$	=	mean sediment size, mm (for both SI and English applications)
$n$	=	Manning's roughness coefficient

The range of data used to develop **Equations 606-7** through **606-9** are shown in **Table 606-1**.



**Table 606-1 Range of Parameters**

Source: FHWA, April 2012, HEC-20, Table 6.2, p. 6.30.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12004.pdf>

Parameter	Value Range
D <sub>50</sub> , mm	0.1 - 2.0
Velocity, ft/s	2.0 – 8.0
Depth, ft	2.0 – 25
Slope, ft/ft	0.00005 - 0.002
Manning roughness coefficient	0.015 - 0.045
Froude Number	0.07 - 0.70
Unit Discharge, cfs/ft	1.0 – 200

For specific values of a, b, and c, the equilibrium slope ( $S_{eq}$ ) can then be computed from:

$$S_{eq} = \left( \frac{a}{q_s} \right)^{\frac{10}{3(c-b)}} q^{\frac{2(2b+3c)}{3(c-b)}} \left( \frac{n}{K_u} \right)^2 \quad \text{606-10}$$

(FHWA, April 2012, HEC-20, Eq. 6.23, p. 6.31)

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12004.pdf>

where:

$S_{eq}$  = equilibrium slope for the channel to match the upstream sediment supply, ft/ft

$q_s$  = upstream sediment supply per unit width, cfs/ft

$q$  = unit discharge, cfs/ft

$K_u$  = 1.486 for English units

all other terms defined previously

In the case of a reduction in sediment supply to a reach that was previously in equilibrium and with all other characteristics remaining constant (discharge, roughness and channel width), the equilibrium slope ( $S_{eq}$ ) can be related to the existing channel slope ( $S_{ex}$ ) by simplifying **Equation 606-10** to produce:

$$S_{eq} = S_{ex} \left( \frac{Q_{s(\text{future})}}{Q_{s(\text{existing})}} \right)^{\frac{10}{3(b-c)}} \quad \text{606-11}$$

(FHWA, April 2012, HEC-20, Eq. 6.24, p. 6.31)

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12004.pdf>

where:

$S_{eq}$  = equilibrium slope for the channel to match the upstream sediment supply, ft/ft  
 $S_{ex}$  = existing channel slope, ft/ft  
 $Q_s$  = sediment supply (existing and future), ft<sup>3</sup>/s  
all other variable were previously defined

The sediment supply ( $Q_s$ ), for existing conditions can be measured or computed. The sediment supply for future conditions must be computed using an applicable sediment transport relationship (FHWA, 2001, HDS-6). **Equations 606-10** and **606-11** also assume that the channel width and bed material size remain constant as the channel degrades. The appropriate discharge for use in these equations is the effective discharge, which is defined as the discharge responsible for the greatest amount of sediment transport and; therefore, is considered to be responsible for channel formation. If the sediment rating curve is combined with a flow duration curve, the flow that transports the greatest quantity of sediment is the effective discharge.

Because **Equations 606-10** and **606-11** use sediment transport capacity and sediment supply where each is determined from the same sediment transport relationship, the selection of the discharge does not greatly affect the equilibrium slope prediction. The bankfull discharge can be used as a reasonable estimate when additional information is unavailable. An example equilibrium slope problem is solved in HEC-20 (FHWA, April 2012, Section 6.5).

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12004.pdf>

### Base Level Control

The equilibrium slope calculations provide an estimate for the slope adjustment inferred by the Lane relationship (Lane, E.W., 1955), but do not yield a prediction of the extent or amount of degradation or the amount of time required to reach equilibrium. In a sediment deficient reach, degradation occurs first at the upstream end of the reach and progresses downstream. The downstream extent of degradation is limited by some vertical control to the channel base level (**Figure 606-1**).

The base level control could be a geologic outcrop of erosion resistant material or extremely coarse material. In a tributary channel, the confluence with a much larger river could act as a downstream control. Lakes, reservoirs, or oceans can also act as controls. Grade control structures and culverts can also limit the extent of degradation downstream. If none of these controls exist, then degradation will continue until the channel reaches the equilibrium slope along the entire profile or until armoring takes place. As tributaries contribute sediment to the downstream channel, the effects of the reduced upstream sediment supply are diminished. The amount of ultimate degradation ( $Y_s$ ) at a location upstream of the base level control can be estimated from the equilibrium slope computation as:

$$Y_s = L (S_{ex} - S_{eq})$$

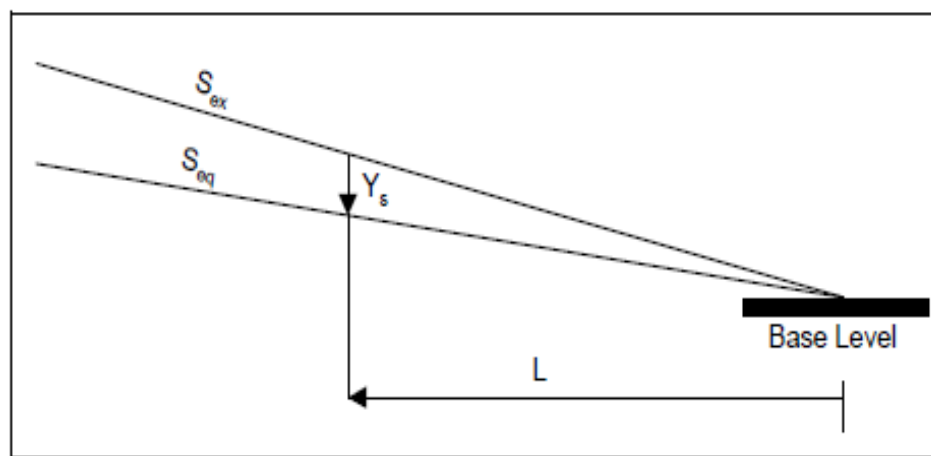
**606-12**

(FHWA, April 2012, HEC-20, Eq. 6.25, p. 6.32)

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12004.pdf>

where:

$Y_s$	=	ultimate degradation amount, ft
$L$	=	distance upstream of base level control, ft
$S_{ex}$	=	existing channel slope, ft/ft
$S_{eq}$	=	equilibrium channel slope, ft/ft



Source: FHWA, April 2012, HEC-20, Figure 6.16, p. 6.32.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12004.pdf>

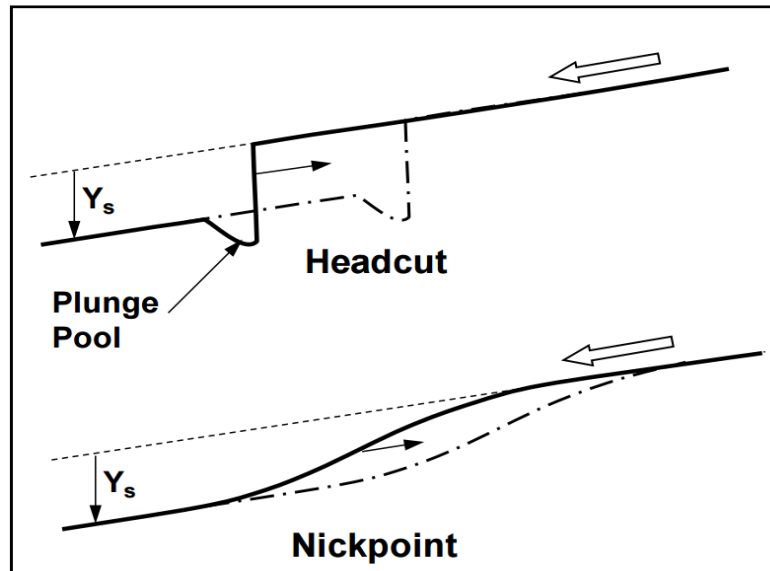
**Figure 606-1 Base Level Control and Degradation Due to Changes in Slope**

Another consideration for base level control occurs when a control is removed or lowered on a primary channel and channel degradation progresses upstream. When a primary channel degrades, the base level control is also lowered for each of its tributaries and degradation can progress up these channels.

**Figure 606-2** illustrates two types of degradation migrating upstream. Headcuts that form in cohesive sediment often form vertical or near-vertical drops with plunge pools at the base of the drop. Headcuts can also be overhanging when weak layers are overlain by a more erosion resistant layer. **Figure 606-3** shows a headcut that will migrate upstream and through the bridge crossing during future runoff events. The features of a headcut that can threaten a bridge include the long-term degradation that persists after the headcut has migrated upstream of the bridge, the plunge pool when the headcut is under the bridge, and channel widening that occurs when bed lowering destabilizes the channel banks.

Nickpoints form in noncohesive sediments in which the over-steepened reach translates upstream. For both headcuts and nickpoints, the cause of the degradation is a lowering of the downstream base level control. Headcuts and nickpoints are best identified through channel reconnaissance (refer to Section 5.2 of HEC-20). It is reasonable to assume that the amount of

degradation will be consistent over the entire stream reach unless there is a longitudinal change in bed and bank materials. It is recommended that detailed monitoring of downstream headcuts be performed to assess threats to existing structures because there are no predictive equations for plunge pool depths, long-term degradation, channel widening, or rate of upstream migration for headcuts or nickpoints.



Source: FHWA, April 2012, HEC-20, Figure 6.17, p. 6.33.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12004.pdf>

**Figure 606-2 Headcuts and Nickpoints**



Source: FHWA, April 2012, HEC-20, Figure 6.18, p. 6.33.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12004.pdf>

**Figure 606-3 Headcut Downstream of Bridge**

### 606.5 Sediment Transport with HEC-RAS

The HEC-RAS program has the capability to simulate sediment transport. Significant effort and time are required to prepare the input data required to develop sediment transport models. Experience and sound engineering judgment are required to evaluate the model results and develop conclusions as to the validity of the results.

The HEC-RAS reference manual provides a description of the transport equations that are available and describes the sediment size ranges for which each equation is most suited. The user must be careful to select equations that are applicable to the actual sediment size ranges and hydraulic conditions of the watercourse. Future versions of HEC-RAS may have additional sediment transport equations besides the currently available equations listed here.

- Ackers and White
- England and Hansen
- Copeland's form of Laursen
- Meyer-Peter and Muller (MPM)
- MPM - Toffaleti
- Toffaleti
- Yang (sand and gravel equations)
- Wilcok and Crowe

A brief overview of the main steps required to develop a HEC-RAS sediment transport model is presented here. Refer to the HEC-RAS Users Manual (current program download) for further explanation and specific model input requirements.

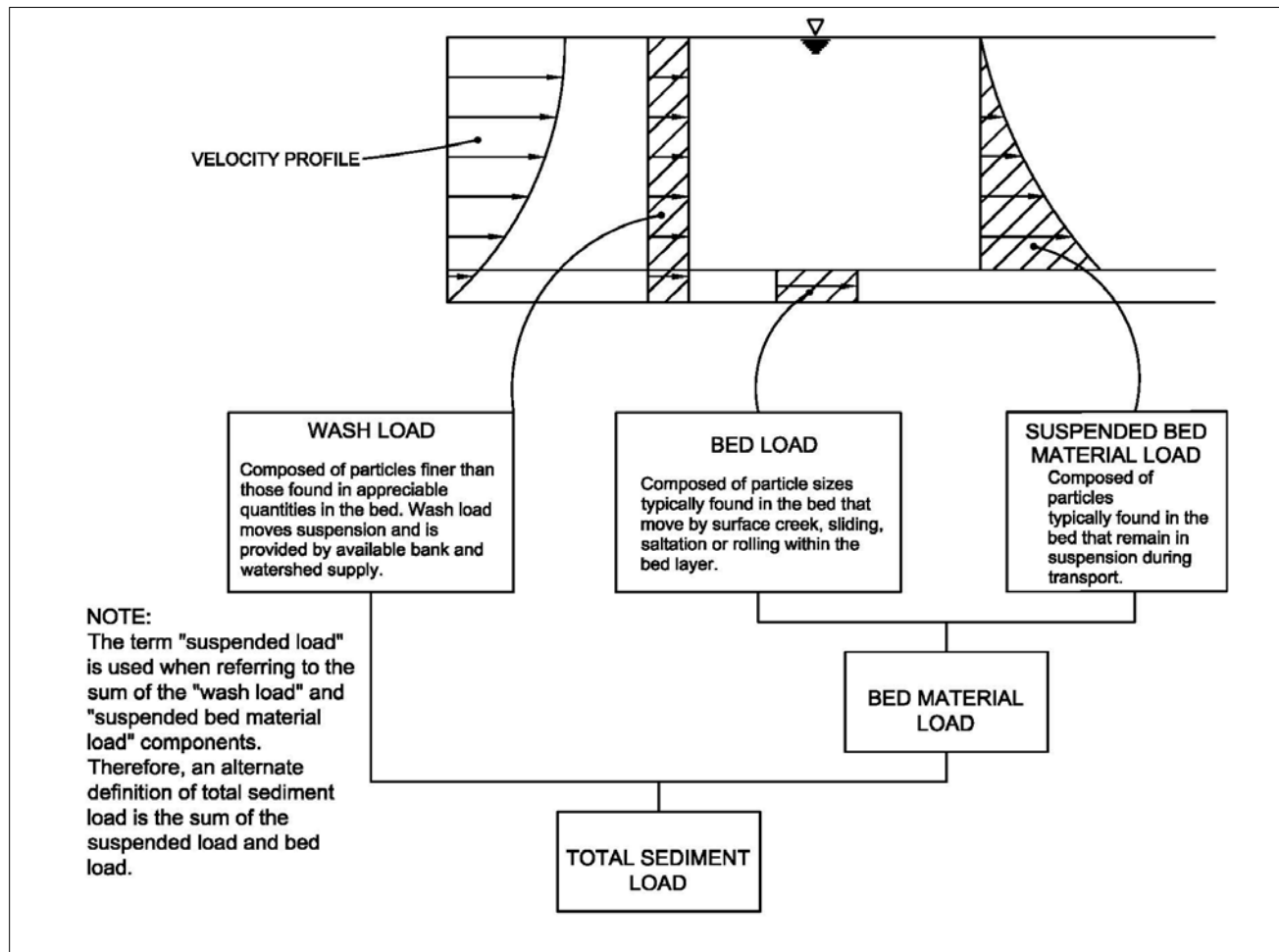
A HEC-RAS Steady Flow model must be developed and finalized prior to the sediment transport model preparation. Then a Quasi-Steady flow model is built that will simulate sediment transport. That model contains the following data:

- A discretized hydrograph (a stepped hydrograph) developed from the runoff hydrograph (this is the quasi-steady flow)
- Sediment gradation data
- Assignment of vertical movable bed depth limits and horizontal erosion limits (this is a subjective effort with assumptions based on site and project information, physical data, constraints such as a bed armor layer, stable banks, vegetated bed and banks, rock banks, floodplain, etc.)
- Selection of an appropriate sediment transport equation

### 606.6 Selected Sediment Transport Equations

**Figure 606-4** is a definition sketch for the different sediment load components, which will be helpful for understanding the following discussion.

Literally, dozens of equations and methods have been developed over the years to quantitatively determine sediment transport rates. Unfortunately, there is no one equation or method that has been proven to apply to all situations in New Mexico.



Source: AMAFCA, November 1994, Sediment and Erosion Design Guide, Figure 3.6, p. 3-22.

[http://www.bernc.gov/uploads/FileLinks/cbb8af72471e4746ba9e92e9a67318bb/Sediment\\_and\\_Erosion\\_Design\\_Guide\\_AMAFCA\\_.pdf](http://www.bernc.gov/uploads/FileLinks/cbb8af72471e4746ba9e92e9a67318bb/Sediment_and_Erosion_Design_Guide_AMAFCA_.pdf)

**Figure 606-4 Definition of Sediment Load Components**

Two equations are presented in the following sections that may provide reasonable sediment transport capacity results if applied properly and within the range of parameters for which the equations were developed.

The two methods to quantitatively determine sediment transport capacity rates are:

- Zeller – Fullerton Equation
- MPM – Woo Equation

These equations have been selected for their relative ease of use and applicability to the types of rivers and arroyos found in New Mexico. A brief discussion on each method follows. The engineer is encouraged to study the applicable referenced material to make an informed decision as to which of the two methods should be used for a given situation or determine if another sediment transport equation may be more applicable.

### 606.6.1 Zeller-Fullerton Equation

Development of this equation is based on data from alluvial channels in and around Tucson and Pima County in Arizona. This region is geologically and topographically similar to many areas of New Mexico. The bed load transport was computed for the data using the Meyer-Peter and Muller (MPM) 1948 equation, whereas the suspended load transport was computed using the Modified Einstein Equation. Zeller and Fullerton then developed a single relationship for the bed material sediment transport, using the results of the computed bed load and suspended load, by regression analysis. The equation is:

$$q_s = 0.0064 \frac{n^{1.77} V^{4.32} G^{0.45}}{Y^{0.3} D_{50}^{0.61}} \quad 606-13$$

(AMAFCA, November 1994, Sediment and Erosion Design Guide, Eq. 3.40, p. 3-34)

[http://www.bernco.gov/uploads/FileLinks/cbb8af72471e4746ba9e92e9a67318bb/Sediment and Erosion Design Guide AMAFCA .pdf](http://www.bernco.gov/uploads/FileLinks/cbb8af72471e4746ba9e92e9a67318bb/Sediment_and_Erosion_Design_Guide_AMAFCA_.pdf)

where:

$q_s$	=	bed material load transport rate per unit channel width, cfs/ft
$n$	=	Manning's roughness coefficient
$V$	=	average velocity, ft/s
$G$	=	gradation coefficient per <b>Equation 606-14</b>
$Y$	=	hydraulic depth, ft ( $y = \text{area (sq ft)} / \text{top width (ft)}$ )
$D_{50}$	=	grain size diameter for which 50% is finer (median grain size), mm

Note that  $D_{50}$  is in millimeters and all other parameters are in the English system of units.

The gradation coefficient (G) in the equation above is determined from the following equation:

$$G = \frac{1}{2} \left( \frac{D_{84}}{D_{50}} + \frac{D_{50}}{D_{16}} \right) \quad 606-14$$

(AMAFCA, November 1994, Sediment and Erosion Design Guide, Eq. 3.14, p. 3-11)

[http://www.bernco.gov/uploads/FileLinks/cbb8af72471e4746ba9e92e9a67318bb/Sediment and Erosion Design Guide AMAFCA .pdf](http://www.bernco.gov/uploads/FileLinks/cbb8af72471e4746ba9e92e9a67318bb/Sediment_and_Erosion_Design_Guide_AMAFCA_.pdf)

where:

the subscripts refer to the percent of sediment material which is finer, mm

When using **Equation 606-13** the values of the parameters must be within or near the ranges given in **Table 606-2**. For values significantly outside the given ranges, another equation should be selected to determine sediment transport rates.

The total transport rate ( $Q_s$ ), is obtained by multiplying the unit transport rate ( $q_s$ ) by the width ( $W$ ) of the channel or channel reach being analyzed.

$$Q_s = q_s W \quad \text{606-15}$$

(AMAFCA, November 1994, Sediment and Erosion Design Guide, Eq. 3.44, p. 3-37)

[http://www.bernco.gov/uploads/FileLinks/cbb8af72471e4746ba9e92e9a67318bb/Sediment and Erosion Design Guide AMAFCA .pdf](http://www.bernco.gov/uploads/FileLinks/cbb8af72471e4746ba9e92e9a67318bb/Sediment_and_Erosion_Design_Guide_AMAFCA_.pdf)

where:

$Q_s$	=	the total bed material transport rate for the channel or channel reach, cfs
$q_s$	=	bed material load transport rate per unit channel width, cfs/ft
$W$	=	the total channel flow width, ft

**Table 606-2 Range of Parameter Values for Zeller-Fullerton Equation**

Source: AMAFCA, November 1994, Sediment and Erosion Design Guide, Table 3.5, p. 3-35.

[http://www.bernco.gov/uploads/FileLinks/cbb8af72471e4746ba9e92e9a67318bb/Sediment and Erosion Design Guide AMAFCA .pdf](http://www.bernco.gov/uploads/FileLinks/cbb8af72471e4746ba9e92e9a67318bb/Sediment_and_Erosion_Design_Guide_AMAFCA_.pdf)

Parameter	Range
Depth	1 – 20 ft
Velocity	3 – 30 ft/s
Manning's "n"	0.018 – 0.035
Bed slope	0.001 – 0.040 ft/ft
Unit discharge	10 – 200 cfs/ft
Particle size	$0.5 \text{ mm} \leq D_{50} \leq 10 \text{ mm}$
Gradation coefficient	$2 \leq G \leq 5$

Appendix 8 contains **Example Problem 8-1** with the Zeller-Fullerton Equation.

## 606.6.2 MPM-Woo Equation

This equation is presented in the Southern Sandoval County (SSCAFCA) Sediment and Erosion Guide, November 2008. It was developed for conditions where steep slopes may produce a flow of water and sediment with a high concentration of suspended sediment. Development of



the equation (Mussetter, 2000) is based on the Meyer-Peter and Muller bed load equation with a correction to account for a very high suspended sediment concentration based on Woo's research (1985).

The equation is:

$$q_s = a V^b Y^c \left(1 - \frac{C_f}{10^6}\right)^d \quad \text{606-16}$$

(SSCAFCA, November 2008, Sediment and Erosion Design Guide, Eq. C.3, p. C.4)

[http://sscafca.org/development/documents/sediment\\_design\\_guide/Sediment%20Design%20Guide%2012-30-08.pdf](http://sscafca.org/development/documents/sediment_design_guide/Sediment%20Design%20Guide%2012-30-08.pdf)

where:

$q_s$	=	bed material load transport rate per unit channel width, cfs/ft
$V$	=	average velocity, ft/s
$Y$	=	hydraulic depth, ft
$C_f$	=	fine sediment (silt and clay) concentration by weight, ppm
$a, b, c, d$	=	coefficients and exponents

Values of  $a$ ,  $b$ ,  $c$ , and  $d$  are obtained from **Figure 606-5**. **Table 606-3** gives the acceptable range of parameter values for use of the MPM-Woo Equation.

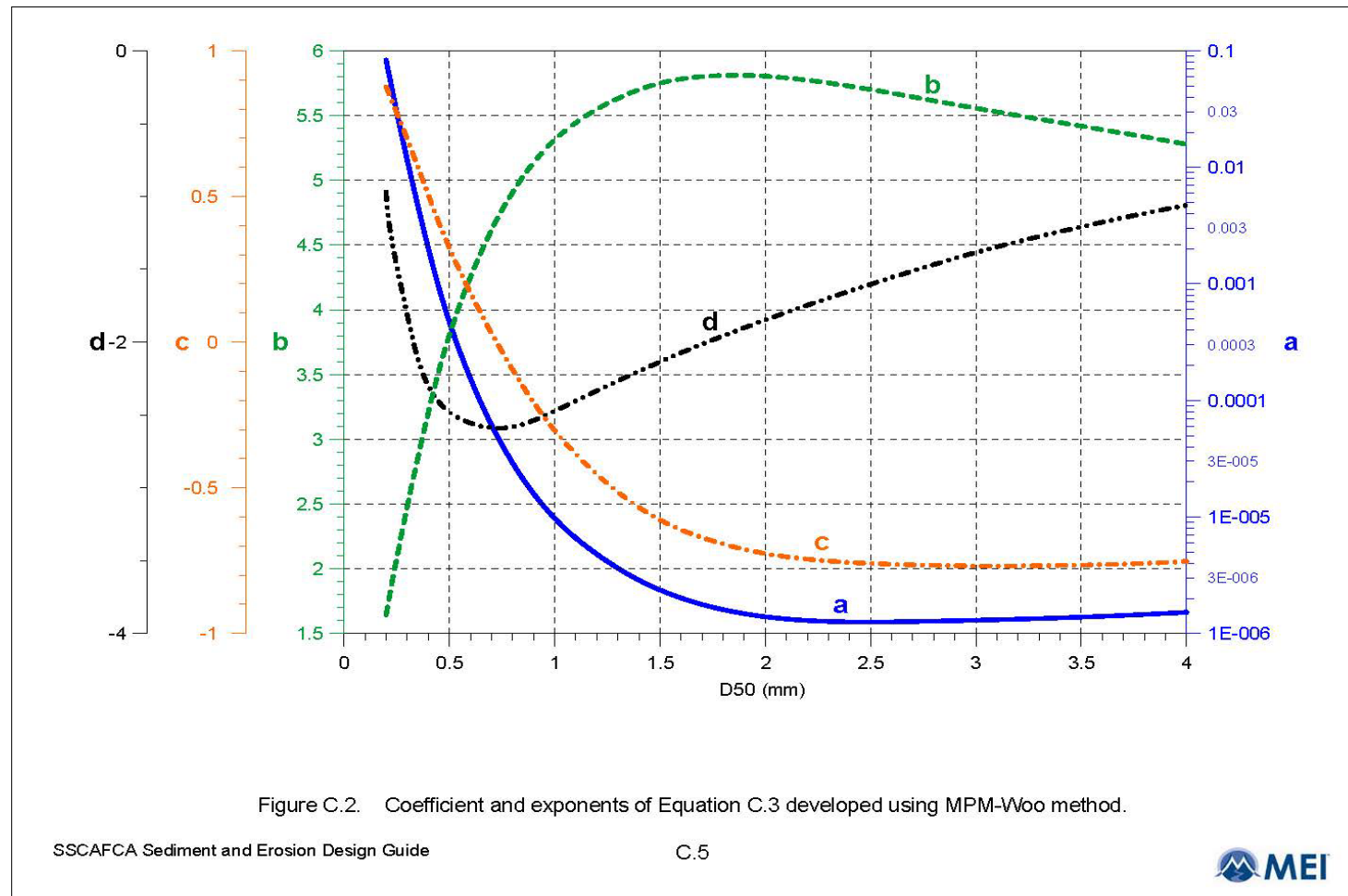
The total transport rate ( $Q_s$ ), is obtained by multiplying the unit transport rate ( $q_s$ ) by the width, ( $W$ ) of the channel or channel reach being analyzed, (**Equation 606-15**).

**Table 606-3 Range of Parameter Values for MPM-Woo Equation**

Source: SSCAFCA, November 2008, Sediment and Erosion Design Guide, Eq. C.3, p. C.4.

[http://sscafca.org/development/documents/sediment\\_design\\_guide/Sediment%20Design%20Guide%2012-30-08.pdf](http://sscafca.org/development/documents/sediment_design_guide/Sediment%20Design%20Guide%2012-30-08.pdf)

Parameter	Range
Unit discharge (water), $q$	1 – 80 cfs/ft
Average velocity, $V$	1.9 – 20.8 ft/s
Hydraulic depth, $Y$	0.3 – 7.2 ft
Channel Slope, $S_o$	0.005 – 0.04 ft/ft
Fine sediment concentration (by weight), $C_f$	0 – 60,000 ppm
Median bed material size ( $D_{50}$ )	0.2 – 4.00 mm



Source: SSCAFCA, November 2008, Sediment and Erosion Design Guide, Figure C.2, p. C.5.

[http://sscafca.org/development/documents/sediment\\_design\\_guide/Sediment%20Design%20Guide%2012-30-08.pdf](http://sscafca.org/development/documents/sediment_design_guide/Sediment%20Design%20Guide%2012-30-08.pdf)

**Figure 606-5 Coefficient and Exponents for MPM-Woo Equation**

Additional restrictions on the use of the MPM-Woo Equation follow.

The computed bed material concentration ( $C_s$ ) (in ppm) must not exceed the maximum bed material concentration ( $C_{sMAX}$ ) (in ppm) given by:

$$C_{sMAX} = 510,000 - 65,000 D_{50} \quad \mathbf{606-17}$$

(SSCAFCA, November 2008, Sediment and Erosion Design Guide, Eq. C.4, p. C.6)

[http://sscafca.org/development/documents/sediment\\_design\\_guide/Sediment%20Design%20Guide%2012-30-08.pdf](http://sscafca.org/development/documents/sediment_design_guide/Sediment%20Design%20Guide%2012-30-08.pdf)

where:

$D_{50}$  = median bed material size, mm  
 $C_{smax}$  = bed material concentration by weight, ppm, computed from the relation:

$$C_s = \frac{2.65 \times 10^6 Q_s}{(Q + 2.65 Q_s)} \quad \mathbf{606-18}$$

(SSCAFCA, November 2008, Sediment and Erosion Design Guide, Eq. C.5, p. C.7)

[http://sscafca.org/development/documents/sediment\\_design\\_guide/Sediment%20Design%20Guide%2012-30-08.pdf](http://sscafca.org/development/documents/sediment_design_guide/Sediment%20Design%20Guide%2012-30-08.pdf)

where:

$C_s$  = bed material concentration by weight, ppm  
 $Q_s$  = the bed material transport rate for the channel or channel reach, cfs  
 $Q$  = the flow rate of water only, cfs

The total bed material load ( $Q_s$ ) is computed as follows:

$$Q_s = q_s W \quad \mathbf{606-19}$$

where:

$Q_s$  = the total bed material transport rate for the channel or channel reach, cfs  
 $q_s$  = bed material load transport rate per unit channel width, cfs/ft  
 $W$  = the total channel flow width, ft

Finally, the fine sediment concentration ( $C_f$ ), should be determined by using the Revised Universal Soil Loss Equation 2 (RUSLE2) (NRCS, 2014). The RUSLE2 equation is an empirical formula for predicting annual soil loss due to sheet and rill erosion. It is perhaps the most widely recognized method for predicting soil erosion. The United States Department of Agriculture (USDA), Natural Resource Conservation Service (NRCS) provides detailed descriptions of this equation and its terms (NRCS, 2014).

Alternatively, the fine sediment load ( $Q_{SF}$ ) may be computed as follows:

$$Q_{SF} = \frac{C_f Q}{2.65 \times 10^6} \quad \text{606-20}$$

(SSCAFCA, November 2008, Sediment and Erosion Design Guide, Eq. C.7, p. C.7)

[http://sscafca.org/development/documents/sediment\\_design\\_guide/Sediment%20Design%20Guide%2012-30-08.pdf](http://sscafca.org/development/documents/sediment_design_guide/Sediment%20Design%20Guide%2012-30-08.pdf)

where:

$Q_{SF}$	=	fine sediment load, cfs
$C_f$	=	fine sediment concentration by weight, ppm
$Q$	=	the flow rate of water only, cfs

Now the total sediment load can be computed from

$$Q_{S_{total}} = Q_S + Q_{SF} \quad \text{606-21}$$

(SSCAFCA, November 2008, Sediment and Erosion Design Guide, Eq. C.8, p. C.7)

[http://sscafca.org/development/documents/sediment\\_design\\_guide/Sediment%20Design%20Guide%2012-30-08.pdf](http://sscafca.org/development/documents/sediment_design_guide/Sediment%20Design%20Guide%2012-30-08.pdf)

where:

$Q_{S_{total}}$	=	the total sediment transport rate for the channel or channel reach, cfs
$Q_S$	=	the bed material transport rate for the channel or channel reach, cfs
$Q_{SF}$	=	fine sediment load, cfs

## Summary

Regardless of the equation selected to determine sediment transport rates, the computed values should be checked against known or measured transport rates if possible to validate the computed results. If no other information or data exists, verification by another sediment transport method is suggested to provide verification of the results although the results may be different. Therefore, engineering judgment is required to select the final results.

Engineering judgment and field verifications are also necessary steps when evaluating sediment transport rates. If computations indicate an aggrading reach and field investigation indicates a degrading reach, then the input parameters must be checked, and/or the system dynamics further investigated by the engineer.

All methods of computing sediment transport rates are approximate. Sediment transport is a complex subject which is not yet completely understood as is evident by the variability of computed results for the same data. The engineer is encouraged to obtain a complete understanding of the derivation and use of any equation or computational method used for computing sediment transport in addition to understanding the limitations of such equations or methods.

## 606.7 References

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## 607 Bridge Scour and Aggradation

Reasonable and prudent hydraulic analysis of a bridge design requires an assessment of the proposed bridge's vulnerability to undermining due to potential scour. Because of the extreme hazard and economic hardships posed by a rapid bridge collapse, special considerations must be given to selecting appropriate flood magnitudes for use in the analysis.

The hydraulic engineer must endeavor to always be aware of and use the most current scour computation methodologies. Users of this manual should consult HEC-18 (FHWA, April 2012) for a more thorough discussion on scour and scour prediction methodology. A companion FHWA document to HEC-18 is HEC-20, "Stream Stability at Highway Structures" (FHWA, April 2012).

The inherent complexities of stream stability, further complicated by highway stream crossings, requires a multilevel solution procedure. The evaluation and design of a highway stream crossing or encroachment should begin with a qualitative assessment of stream stability (**Section 605**). This involves the application of geomorphic concepts to identify potential problems and alternative solutions. This qualitative analysis should be followed with basic hydrologic, hydraulic, and sediment transport engineering concepts. Such analyses could include evaluation of flood history, channel hydraulic conditions (up to and including water surface profile analysis), and basic sediment transport analyses such as evaluation of watershed sediment yield, incipient motion analysis, and scour calculations.

These analyses can be adequate for many locations if the relationships between different factors affecting stability are adequately explained. If not, more complex quantitative analyses based on detailed mathematical modeling and/or physical hydraulic models should be considered. This multi-level approach is presented in HEC-20 (April 2012).

Where freeboard is limited, problems associated with increased flood hazards to upstream property or to the traveling public due to more frequent roadway overtopping may occur. Where aggradation is expected, it may be necessary to evaluate these impacts.

### **607.1 Scour Types**

Present technology dictates that bridge scour be evaluated as interrelated components:

- Plan form change (lateral channel movement),
- Long-term profile changes (aggradation/degradation),
- Contraction scour/deposition, and
- Local scour (piers, abutments, and embankments)

Note that total scour is the summation of the long-term scour, contraction scour and local scour.

#### **Plan Form Changes**

Plan form changes are morphological changes such as meander migration or bank widening. The lateral movement of meanders can threaten bridge approaches as well as increase scour by changing flow patterns approaching a bridge opening. Bank widening can cause significant changes in the flow distribution and thus the bridge's flow contraction ratio.

#### **Long-Term Profile Changes**

Long-term profile changes can result from stream bed profile changes that occur due to general aggradation and/or degradation. Aggradation is the deposition of bed load due to a decrease in the energy gradient. Degradation is the scouring of bed material due to increased stream sediment transport capacity which results from an increase in the energy gradient.

Forms of degradation and aggradation should be considered as imposing a permanent future change for the stream bed elevation at a bridge site whenever they can be identified.

Factors that affect long-term bed elevation changes are: dams and reservoirs (upstream or downstream of the bridge), changes in watershed land use (urbanization, deforestation, etc.), channelization, cutoffs, meander bends (natural or man-made), changes in the downstream channel base level (control), gravel mining from the stream bed, diversion of water into or out of the stream, natural lowering of the total system, movement of a bend, bridge location with respect to stream plan form, and stream movement in relation to the crossing.

An assessment of long-term stream bed elevation changes should be made using the principles of river mechanics. Such an assessment requires the consideration of all influences upon the stream system, including runoff from the watershed to a stream (hydrology), the sediment delivery to the channel (erosion), the sediment transport capacity of a stream (hydraulics), and the response of a stream to these factors (geomorphology and river mechanics). Many of the largest impacts are from human activities. This assessment requires a study of the history of the river and human activities as well as a study of present water use, land use, storm runoff, and stream control activities. It is important that all agencies involved with the watercourse should be contacted to determine possible future changes.

To organize such an assessment, the three-level fluvial system approach can be used, which is presented previously in **Section 605.4**.

1. A qualitative determination based on general geomorphic and river mechanics relationships;

2. An engineering geomorphic analysis using established qualitative and quantitative relationships to estimate the probable behavior of the stream system to various scenarios of future conditions; and
3. Physical process computer modeling using mathematical models to make predictions of quantitative changes in stream bed elevation due to changes in the stream and watershed.

Refer to HEC-20 (FHWA, April 2012) and "Highways in the River Environment" (FHWA, February 1990) for detailed information on this three-level approach.

FHWA, February 1990, "Highways in the River Environment".

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hire1990.pdf>

FHWA, April 2012, "HEC-20, Stream Stability at Highway Structures, Fourth Edition".

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12004.pdf>

### **Contraction Scour**

Channel contraction scour results from a constriction of the channel which may, in part, be caused by bridge piers in the waterway, but is more often caused by abutments placed within the natural channel floodplain. Deposition results from an expansion of the channel or the bridge site being located immediately downstream of a steeper reach of stream. Highways, bridges, and natural channel contractions are the most common causes of contraction scour.

### **Local Scour**

Abutments or piers located within the flood flow area increase the potential for scour to occur immediately adjacent to the piers and abutments. The basic mechanism causing local scour at piers or abutments is the formation of vortices at their base. The vortices result from the pileup of water on the upstream surface of the obstruction and subsequent acceleration of the flow around the nose of the pier or embankment. The action of the vortex removes bed material from around the base of the obstruction. The transport rate of sediment away from the base region is greater than the transport rate into the region, and, consequently, a scour hole develops. As the depth of scour increases, the strength of the vortex is reduced, thereby reducing the transport rate from the base region. Eventually, for live-bed local scour, equilibrium is reestablished and scouring ceases. Live-bed scour occurs when there is bed material being transported into the channel reach being analyzed.

## **607.2 Armoring**

Armoring occurs where a stream or river is unable to move the coarser material comprising either the bed or, if some bed scour occurs, its underlying material. Scour may occur initially, but later it becomes arrested by armoring before the full scour potential is reached during a subsequent flood magnitude. When armoring does occur, the coarser bed material will tend to remain in place or quickly redeposit to form a layer of riprap-like armor on the stream bed or in the scour holes, and thus limit further scour. This armoring effect may decrease scour hole depths, which were predicted based on formulae developed for sand or other fine material channels for a particular flood magnitude. When a larger flood occurs, than was used to define



the probable scour hole depths, scour will probably penetrate deeper until armoring occurs again.

Armoring may also cause bank widening. Bank widening encourages rivers or streams to seek a more unstable, braided regime. Such instabilities may pose serious problems for bridges as they encourage further instabilities, making it difficult to assess plan form changes. Bank widening also spreads the approach flow distribution which in turn results in a more severe bridge opening contraction.

Caution is needed in determining the scour resistance of bed materials and the underlying strata. With sandy material, the passage of a single flood may result in the predicted scour depths. Conversely, in scour-resistant material the maximum predicted depth of scour may not be realized during the passage of a particular flood; however, some scour-resistant material may be lost. Commonly, this material is replaced with more easily scoured material. Thus, at some later date another flood may reach the predicted scour depth. Serious scour has been observed to occur in materials commonly perceived to be scour-resistant such as consolidated soils and glacial till, as well as so-called bed rock streams and streams with gravel and boulder beds.

### **607.3 Scour Analysis Methods**

Prior to applying the various scour estimating methods for contraction and local scour, it is necessary to: (1) obtain the fixed-bed channel hydraulics, (2) estimate the long-term impact of degradation or aggradation on the bed profile, (3) if appropriate, adjust the fixed-bed hydraulics to reflect these changes, and (4) compute the bridge hydraulics.

The seven steps recommended for estimating scour at bridges are:

- Step 1 Determine scour analysis variables
- Step 2 Analyze long-term bed elevation change
- Step 3 Evaluate the scour analysis method
- Step 4 Compute the magnitude of contraction scour
- Step 5 Compute the magnitude of local scour at abutments
- Step 6 Compute the magnitude of local scour at piers
- Step 7 Plot the total scour depths

The procedures for each of the steps, including recommended scour equations, are discussed in detail in HEC-18 (FHWA, April 2012). The basic equations associated with steps 4, 5, and 6 follows. Refer to HEC-18 for more information on the applicability and derivation of these equations.

In the following sections, various formulas are presented to determine the average scour depth " $y_s$ ". A positive value indicates the depth of the scour hole below the initial channel invert or ground surface before scour. A negative value indicates that there will not be any scour for the condition analyzed and a scour depth of zero should be assumed. A negative value should be interpreted as an indication of aggradation.

#### **607.3.1 Contraction Scour Equations**

HEC-18 presents contraction scour equations which are based on the principle of conservation of sediment transport. It simply means that the fully developed scour in a bridge cross section

reaches equilibrium when sediment transported into the contracted section equals sediment transported out of the section in the case of live-bed scour. In other words, the effective shear stress in the contracted section has been decreased because the scouring has increased the flow area so that the effective shear stress is equal to the critical shear stress for the sediment at the bottom of the contracted cross section.

There are two forms of contraction scour depending on the ability of the uncontracted approach flow to transport bed material into the contraction location. Live-bed scour occurs when there is bed material being transported into the channel reach being analyzed. Clear-water scour is the case when there is no bed material being transported in the flow upstream of the cross section.

### 607.3.2 Live-Bed Contraction Scour

A modified version of Laursen's 1960 equation for live-bed scour at a long contraction is recommended to predict the depth of scour in a contracted section (Laursen, 1960). The original equation can be found in Appendix C of HEC-18 (FHWA, April 2012). The modification is to eliminate the ratio of Manning's "n" (see HEC-18 for more information). The equation assumes that bed material is being transported from the upstream section.

$$\frac{y_2}{y_1} = \left( \frac{Q_2}{Q_1} \right)^{6/7} \left( \frac{W_1}{W_2} \right)^{K_1} \quad 607-1$$

(FHWA, April 2012, HEC-18, Eq. 6.2, p. 6.10)

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12003.pdf>

The average scour depth is  $y_s = y_2 - y_o$ .

where:

$y_1$	=	average depth in the upstream channel, ft
$y_2$	=	average depth in the contracted section, ft
$y_o$	=	existing depth in the contracted section before scour, ft (see note 5)
$Q_1$	=	flow in the upstream channel transporting sediment, ft <sup>3</sup> /s
$Q_2$	=	flow in the contracted channel, ft <sup>3</sup> /s
$W_1$	=	bottom width of the upstream main channel that is transporting bed material, ft
$W_2$	=	bottom width of the main channel in contracted section less pier width(s), ft
$K_1$	=	exponent determined below

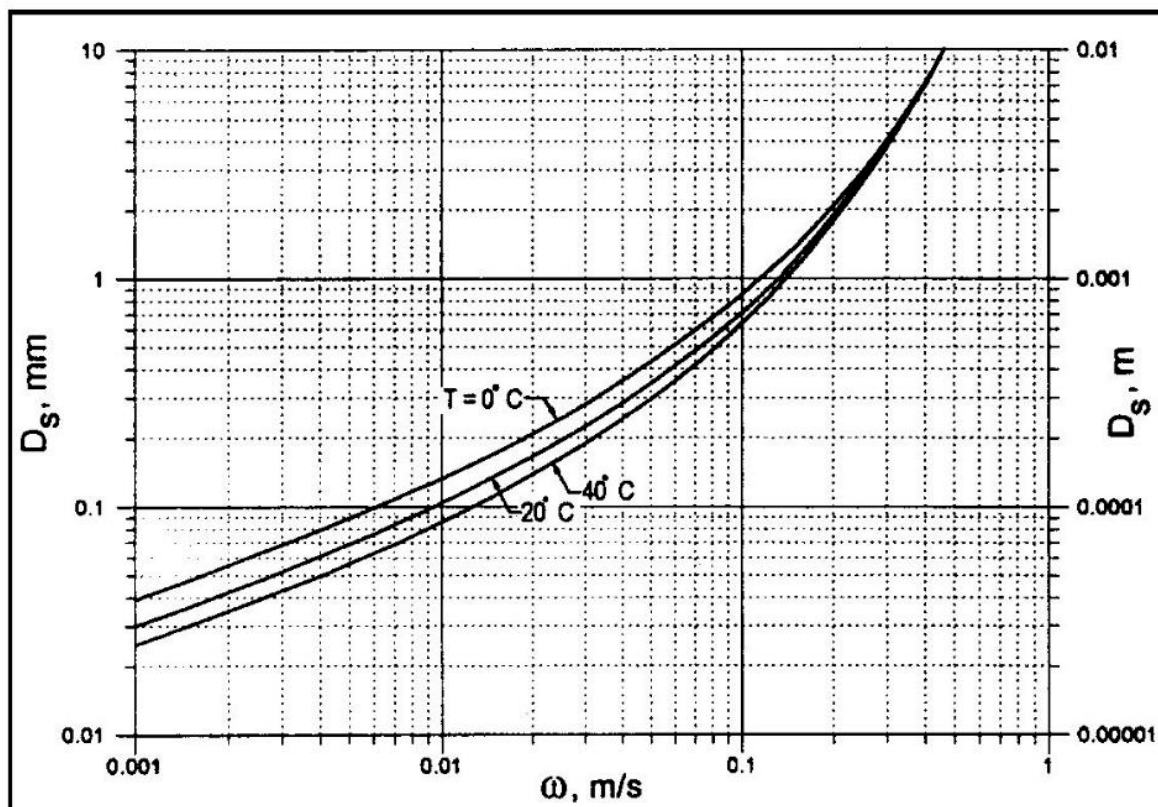
$V_* / \omega$	$K_1$	Mode of Bed Material Transport
< 0.5	0.59	Mostly contact bed material discharge
0.5 to 2.0	0.64	Some suspended bed material discharge
> 2.0	0.69	Mostly suspended bed material discharge

where:

$V_*$	=	$(\tau_o / \Delta)^{1/2} = (g y_1 S_1)^{1/2}$	, shear velocity in the upstream section, ft/s
$\omega$	=	fall velocity of bed material based on the $D_{50}$ (see <b>Figure 607-1</b> ), m/s For fall velocity in English units (ft/s) multiply $\omega$ in m/s by 3.28	
$g$	=	gravitational acceleration, 32.2 ft/s <sup>2</sup>	
$S_1$	=	slope of energy grade line of main channel, ft/ft	
$\tau_o$	=	shear stress on the bed, lb/ft <sup>2</sup>	
$\Delta$	=	density of water, 1.94 slugs/ft <sup>3</sup>	

Notes: related to **Equation 607-1** and variables:

1.  $Q_2$  may be the total flow going through the bridge opening for contraction scour but is not in all cases. Refer to HEC-18 for more information on contraction scour cases.
2.  $Q_1$  is the flow in the main channel upstream of the bridge, not including overbank flows.
3.  $W_1$  and  $W_2$  are not always easily defined. In some cases, it is acceptable to use the top width of the main channel to define these widths. Whether top width or bottom width is used, it is important to be consistent so that  $W_1$  and  $W_2$  refer to either bottom widths or top widths.
4. Laursen's Equation will overestimate the depth of scour at the bridge if the bridge is located at the upstream end of a natural contraction or if the contraction is the result of the bridge abutments and piers. At this time, however, it is the best equation available.
5. In sand channel streams where the contraction scour hole is filled in on the falling limb of the hydrograph, the  $y_o$  depth may be approximated by  $y_1$ . Sketches or surveys through the bridge can help in determining the existing bed elevation.
6. Scour depths with live-bed contraction scour may be limited by coarse sediments in the bed material armoring the bed. Where coarse sediments are present, it is recommended that scour depths be calculated for live-bed scour conditions using the clear-water scour equation (given in the next section) in addition to the live-bed equation and that the smaller calculated scour depth be used.



Source: FHWA, April 2012, HEC-18, Figure 6.8, p. 6.11.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12003.pdf>

**Figure 607-1 Fall Velocity of Sand-sized Particle with Specific Gravity 2.65 in Metric Units**

### 607.3.3 Clear-Water Contraction Scour

Clear-water contraction scour occurs in a long contraction when (1) there is no bed material transport from the upstream reach into the downstream reach or (2) the material being transported in the upstream reach is transported through the downstream reach mostly in suspension and at less than capacity of the flow. With clear-water contraction scour the area of the contracted section increases until the velocity of the flow ( $V$ ) or the shear stress ( $\tau_o$ ) on the bed is equal to the critical velocity ( $V_c$ ) or the critical shear stress ( $\tau_{c0}$ ) of a certain particle size ( $D$ ) in the bed material. Normally, the width ( $W$ ) of the contracted section is constrained and the depth ( $y$ ) increases until the limiting conditions are reached.

The recommended clear-water contraction scour equation is based on a development suggested by Laursen (1963) (presented in Appendix C of HEC-18). The equation is:

$$y_2 = \left[ \frac{0.0077 Q^2}{D_m^{2/3} W^2} \right]^{3/7}$$

**607-2**

(FHWA, April 2012, HEC-18, Eq. 6.4, p. 6.12)

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12003.pdf>

$y_s = y_2 - y_o$  = average scour depth

**607-3**

(FHWA, April 2012, HEC-18, Eq. 6.5, p. 6.12)

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12003.pdf>

where:

$y_s$	=	average scour depth, ft
$y_2$	=	average equilibrium depth in the contracted section after contraction scour, ft
$y_o$	=	average existing depth in the contracted section, ft
$Q$	=	discharge through the bridge or on the set-back overbank area at the bridge associated with the width $W$ , ft <sup>3</sup> /s
$D_m$	=	diameter of the smallest non-transportable particle in the bed material (1.25 $D_{50}$ ) in the contracted section, ft
$D_{50}$	=	median diameter of bed material, ft
$W$	=	bottom width of the contracted section less pier widths, ft

A reasonable lower limit of  $D_{50}$  equal to 0.2 mm can be applied to this equation. Using a size smaller than 0.2 mm will over-estimate clear-water contraction scour.

Because  $D_{50}$  is not the largest particle in the bed material, the scoured section can be slightly armored. Therefore, the  $D_m$  is assumed to be 1.25  $D_{50}$ . For stratified bed material the depth of scour can be determined by using the clear-water scour equation sequentially with successive  $D_m$  of the bed material layers.

### 607.3.4 Scour at Abutments

The HEC-18, April 2012 document lists several sources for abutment scour equations. However, the document only presents detail for the following abutment scour equations:

- The Froehlich Equation
- The HIRE Equation
- The NCHRP 24-20 Approach/Method.

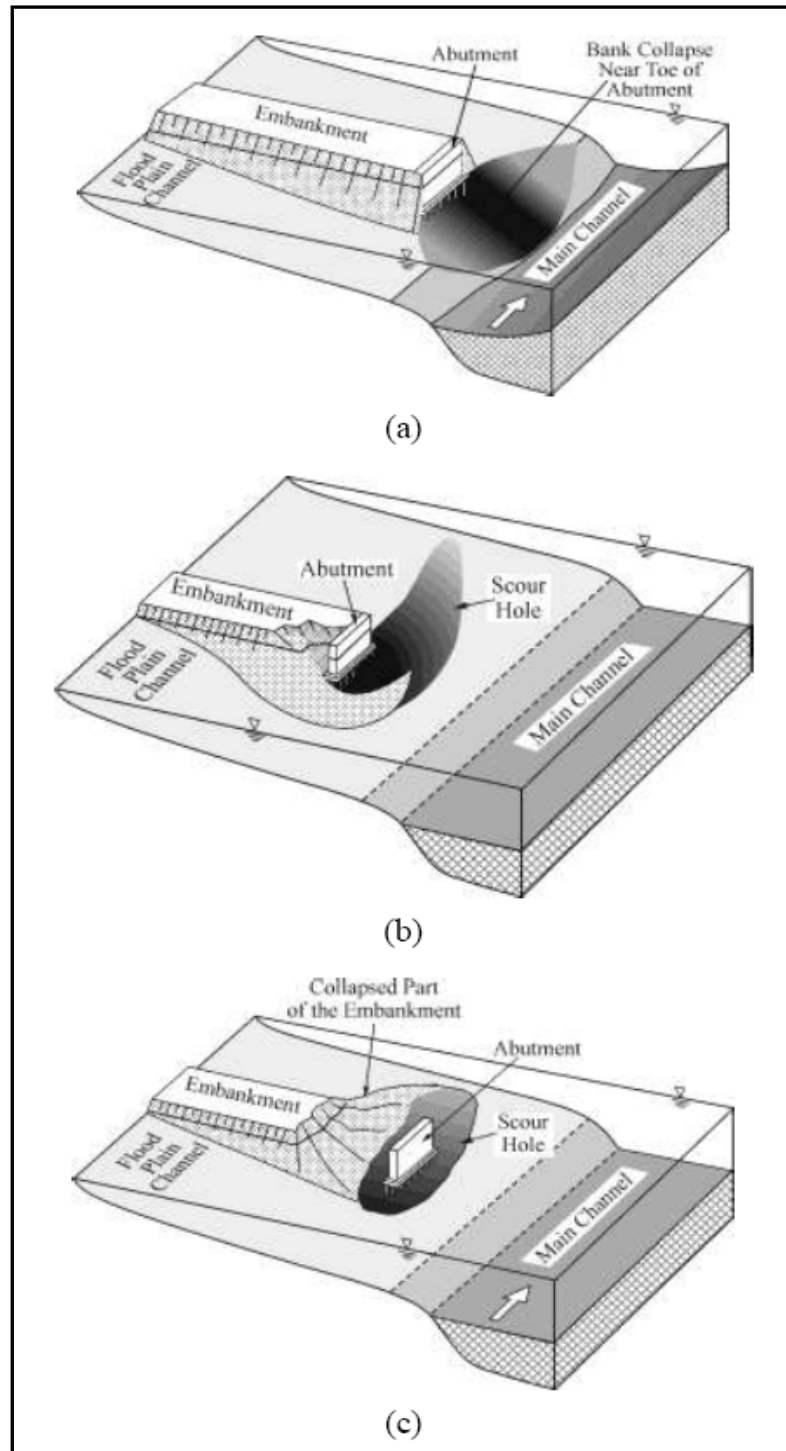
The FHWA has informed the NMDOT Drainage Design Bureau that the NCHRP 24-20 Approach is preferred over the other equations. Therefore, only the NCHRP 24-20 Approach is presented here.

FHWA, April 2012, HEC-18, Eq. 6.5, p. 6.12

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12003.pdf>

#### NCHRP 24-20 Abutment Scour Approach

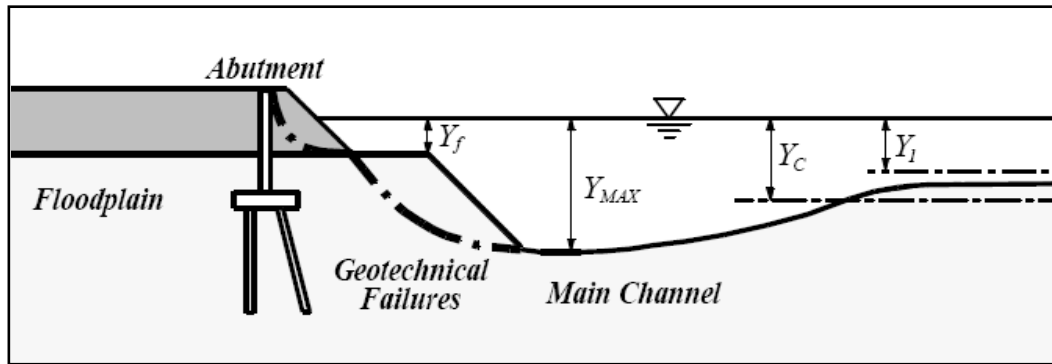
NCHRP (2010b) developed abutment scour equations considering a range of abutment types, abutment locations, flow conditions, and sediment transport conditions. These equations use contraction scour as the starting calculation for abutment scour and apply a factor to account for large-scale turbulence that develops in the vicinity of the abutment. One important distinction regarding the contraction scour calculation is that the abutment creates a non-uniform flow distribution in the contracted section. The flow is more concentrated in the vicinity of the abutment and the contraction scour component is greater than for average conditions in the constricted opening. The three scour conditions illustrated in **Figure 607-2** are (a) scour occurring when the abutment is in or close to the main channel, (b) scour occurring when the abutment is set back from the main channel, and (c) scour occurring when the embankment breaches and the abutment foundation acts as a pier. As illustrated in **Figure 607-3**, the NCHRP study also concluded that there is a limiting depth of abutment scour when the geotechnical stability of the embankment or channel bank is reached.



(FHWA, April 2012, HEC-18, Figure 8.7, p. 8.9)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12003.pdf>

**Figure 607-2 Abutment Scour Conditions**



(FHWA, April 2012, HEC-18, Figure 8.8, p. 8.10)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12003.pdf>

**Figure 607-3 Conceptual Geotechnical Failures Resulting from Abutment Scour**

The abutment scour computed from the NCHRP approach is total scour at the abutment; it is not added to contraction scour because it already includes contraction scour. The advantages of using the NCHRP abutment scour equations include (1) not using the effective embankment length,  $L'$ , which is difficult to determine in many situations; (2) the equations are more physically representative of the abutment scour process; and (3) the equations predict total scour at the abutment rather than the abutment scour component that is then added to contraction scour. The scour equations for conditions (a) and (b) respectively are:

$$y_{\max} = \alpha_A y_c \text{ or } y_{\max} = \alpha_B y_c \quad 607-4$$

$$y_s = y_{\max} - y_0 \quad 607-5$$

(FHWA, April 2012, HEC-18, Eq. 8.3 & 8.4, p. 8.8)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12003.pdf>

where:

- $y_{\max}$  = maximum flow depth resulting from abutment scour, ft
- $y_c$  = flow depth including live-bed or clear-water contraction scour, ft
- $\alpha_A$  = amplification factor for live-bed conditions
- $\alpha_B$  = amplification factor for clear-water conditions
- $y_s$  = abutment scour depth, ft
- $y_0$  = flow depth prior to scour, ft



Based on the NCHRP (2010b) study, if the projected length of the embankment ( $L$ ) is 75 percent or greater than the width of the floodplain ( $B_f$ ), scour condition (a) in **Figure 607-2** occurs and the contraction scour calculation is performed using a live-bed scour calculation. The contraction scour equation is a simplified version of the live-bed contraction scour equation. The equation combines the discharge and width ratios due to the similarity of the exponents because other uncertainties are more significant. By combining the discharge and width, the live-bed contraction scour equation simplifies to the ratio of two-unit discharges. Unit discharge ( $q$ ) can be estimated either by discharge divided by width or by the product of velocity and depth. The contraction scour equation is:

$$y_c = y_1 \left( \frac{q_{2c}}{q_1} \right)^{6/7} \quad 607-6$$

(FHWA, April 2012, HEC-18, Eq. 8.5, p. 8.10)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12003.pdf>

where:

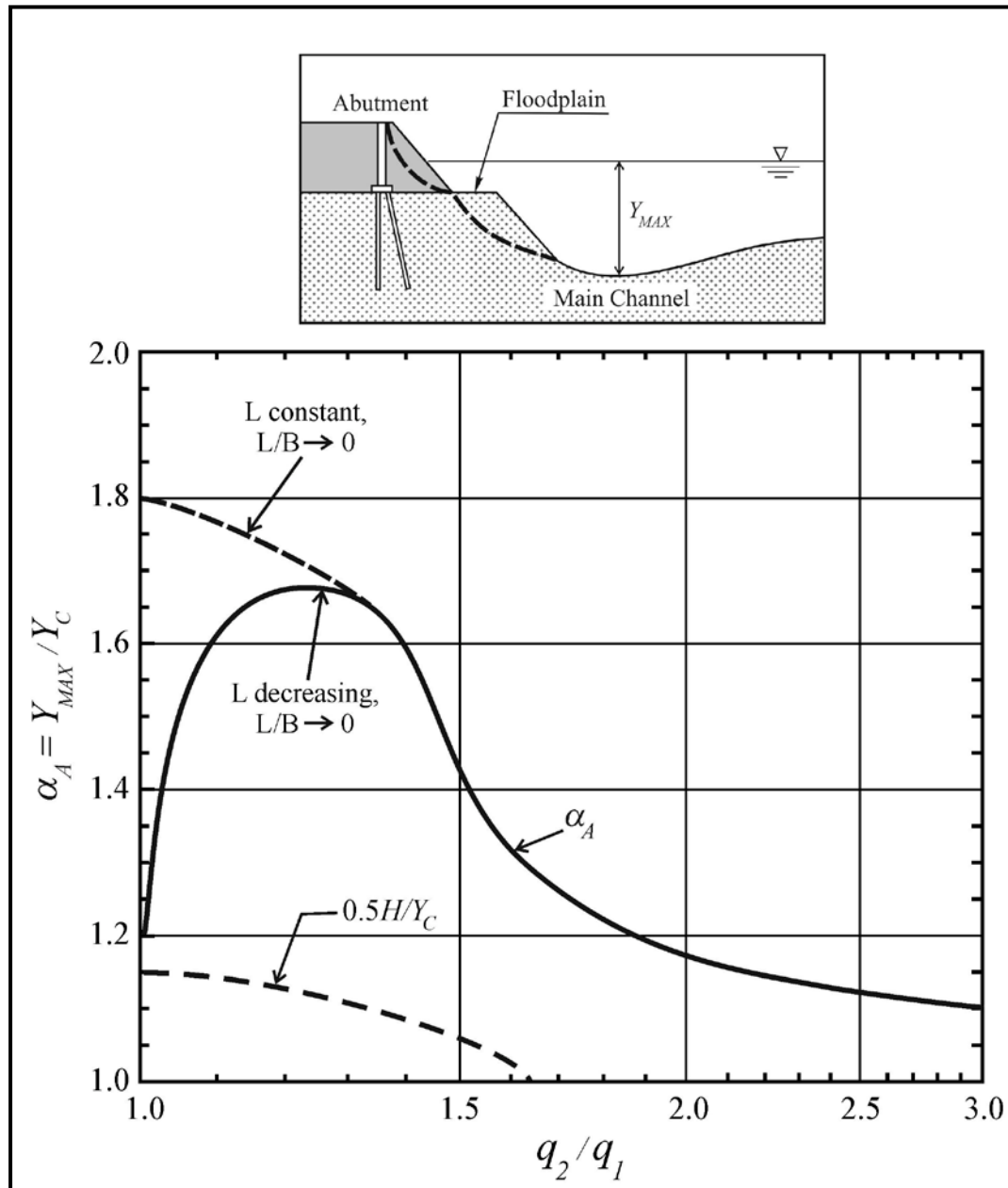
$y_c$	=	flow depth including live-bed contraction scour, ft
$y_1$	=	upstream flow depth, ft
$q_1$	=	upstream unit discharge, ft <sup>2</sup> /s
$q_{2c}$	=	unit discharge in the constricted opening accounting for non-uniform flow distribution, ft <sup>2</sup> /s

The value of  $q_{2c}$  can be estimated as the total discharge in the bridge opening divided by the width of the bridge opening. The value of  $y_c$  is then used in **Equation 607-4** to compute the total flow depth at the abutment. The value of  $\alpha_A$  is selected from one of the following figures:

**Figure 607-4** for spill-through abutments, and

**Figure 607-5** for wingwall abutments.

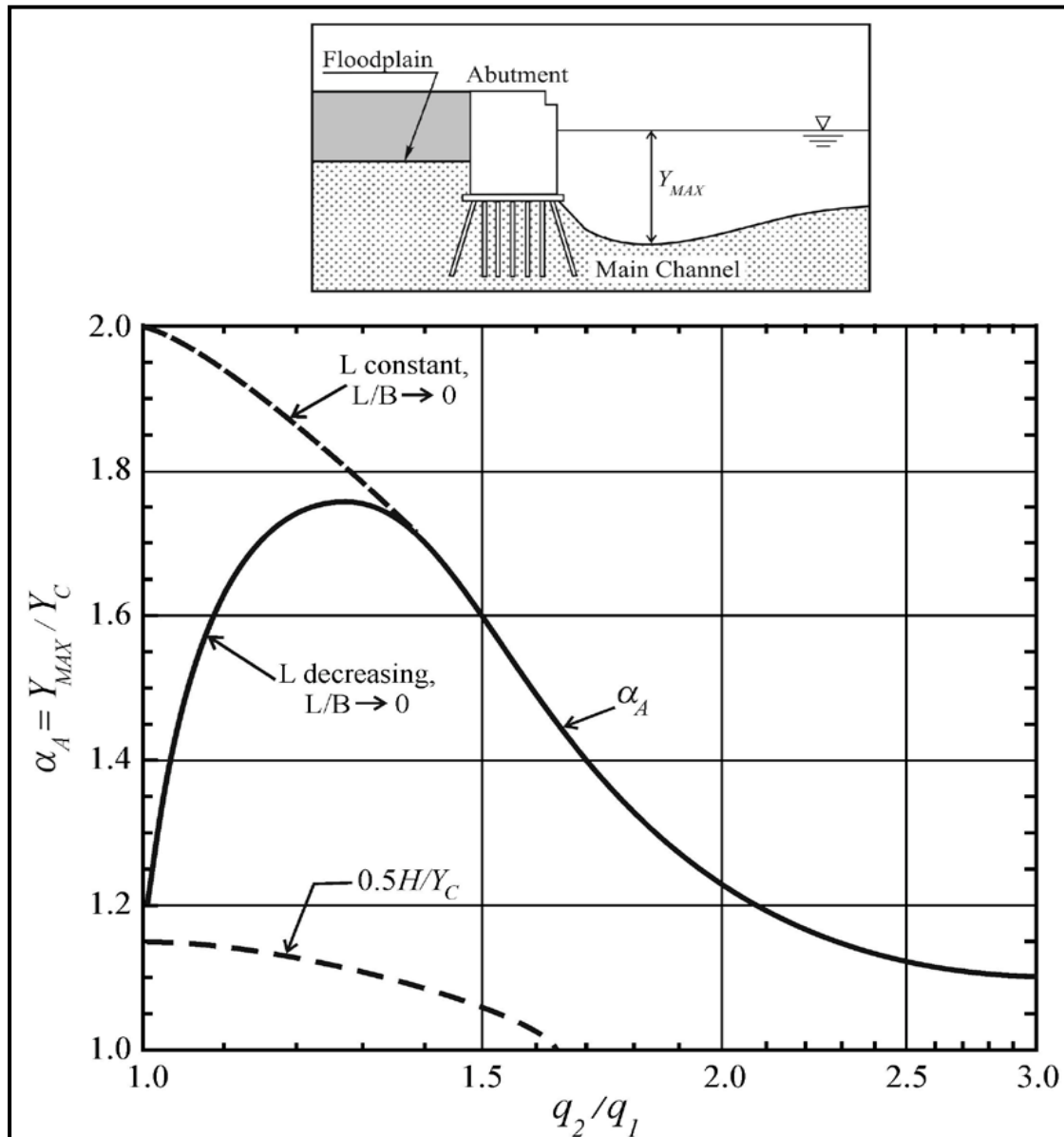
The solid curves should be used for design. The dashed curves represent theoretical conditions that have yet to be proven experimentally. For low values of  $q_2/q_1$ , contraction scour is small, but the amplification factor is large because flow separation and turbulence dominate the abutment scour process. For large values of  $q_2/q_1$ , contraction scour dominates the abutment scour process and the amplification factor is small.



(FHWA, April 2012, HEC-18, Figure 8.9, p. 8.11)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12003.pdf>

**Figure 607-4 Scour Amplification Factor for Spill-Through Abutments and Live-Bed Conditions**



(FHWA, April 2012, HEC-18, Figure 8.10, p. 8.12)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12003.pdf>

**Figure 607-5 Scour Amplification Factor for Wingwall Abutments and Live-Bed Conditions**

If the projected length ( $L$ ) of the embankment is less than 75 percent of the width of the floodplain ( $B_f$ ), scour condition (b) in **Figure 607-2** occurs and the contraction scour calculation is performed using a clear-water scour calculation. The clear-water contraction scour equation also uses unit discharge ( $q$ ), which can be estimated either by discharge divided by width or by the product of velocity and depth. Two clear-water contraction scour equations may be applied. The first equation is the standard equation based on grain size:

$$y_c = \left( \frac{q_{2f}}{K_u D_{50}^{1/3}} \right)^{6/7}$$

**607-7**

(FHWA, April 2012, HEC-18, Eq. 8.6, p. 8.12)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12003.pdf>

where:

$y_c$	=	flow depth including clear-water contraction scour, ft
$q_{2f}$	=	unit discharge in the constricted opening accounting for non-uniform flow distribution, ft <sup>2</sup> /s
$K_u$	=	11.17 English units
$D_{50}$	=	particle size with 50 percent finer, ft

Note that a lower limit of particle size of 0.2 mm (0.00657 ft) is reasonable because cohesive properties limit the critical velocity and shear stress for cohesive soils. If the critical shear stress is known for a floodplain soil, then an alternative clear-water scour equation can be used:

$$y_c = \left( \frac{\gamma}{\tau_c} \right)^{3/7} \left( \frac{n q_{2f}}{K_u} \right)^{6/7}$$

**607-8**

(FHWA, April 2012, HEC-18, Eq. 8.7, p. 8.13)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12003.pdf>

where:

$n$	=	Manning's "n" of the floodplain material under the bridge
$\tau_c$	=	critical shear stress for the floodplain material, lb/ft <sup>2</sup> , (Pa)
$\gamma$	=	unit weight of water, lb/ft <sup>3</sup>
$K_u$	=	1.486 English units
all other variables are previously defined		

The value of  $q_{2f}$  should be estimated including local concentration of flow at the bridge abutment. The value of  $q_f$  is the floodplain flow upstream of the bridge. The value of  $y_c$  is then used in **Equation 607-4** to compute the total flow depth at the abutment. The value of  $\alpha_B$  is selected from the following figures:

**Figure 607-6** for spill-through abutments, and

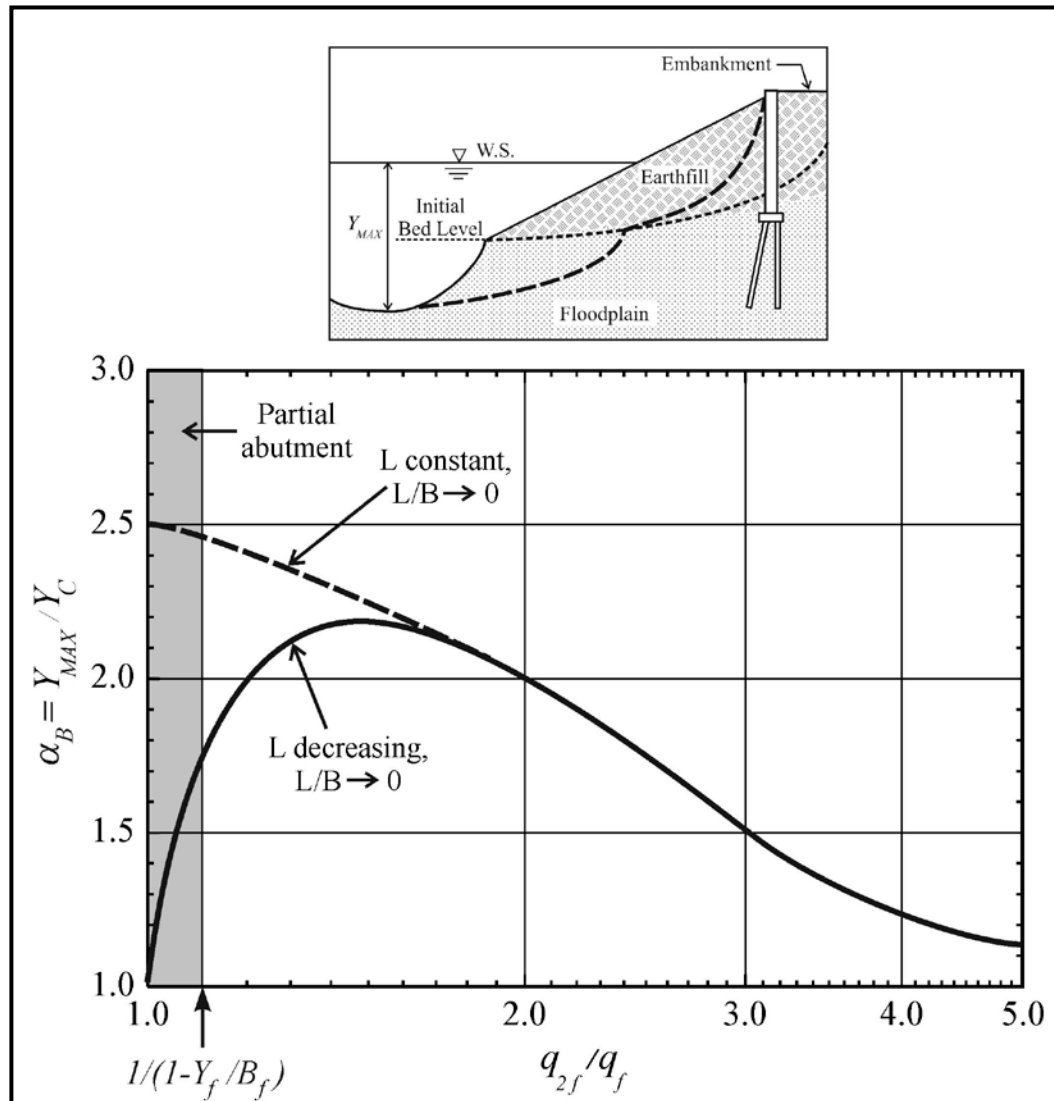
**Figure 607-7** for wingwall abutments.

The solid curves should be used for design. The dashed curves represent theoretical conditions that have yet to be proven experimentally. For low values of  $q_2/q_1$ , contraction scour is small, but the amplification factor is large because flow separation and turbulence dominate the abutment

scour process. For large values of  $q_2/q_1$ , contraction scour dominates the abutment scour process and the amplification factor is small.

For scour estimates determined for either condition (a) or (b) the geotechnical stability of the channel bank or embankment should be considered. If the channel bank or embankment is likely to fail, then the limiting scour depth is the geotechnically stable depth and erosion will progress laterally. This may cause the embankment to breach and another scour estimate can be performed treating the abutment foundation as pier.

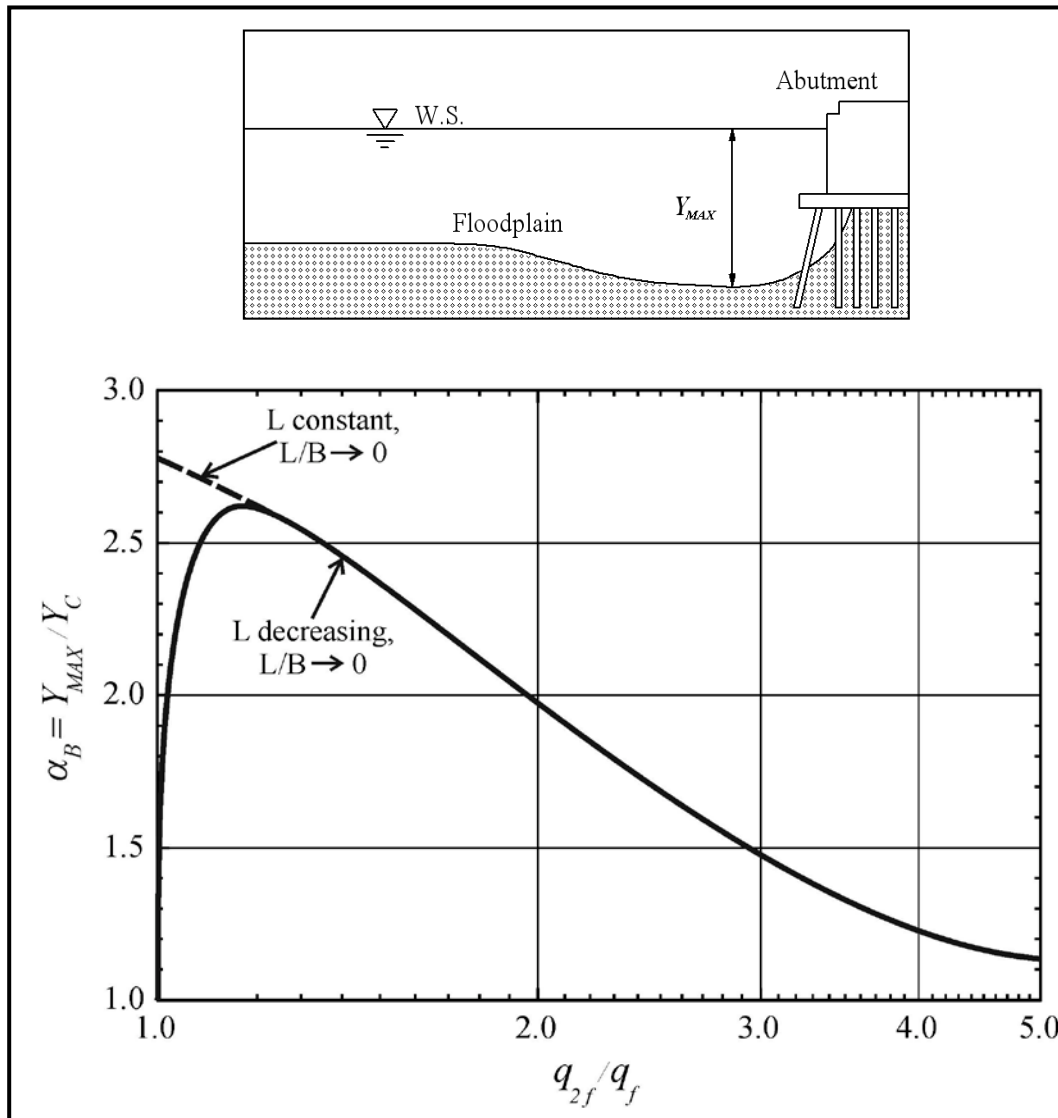
There are many uncertainties in determining the variables for these abutment scour equations. Determining the grain size or critical shear stress of the floodplain soils is one source of uncertainty. Determining the value of the unit discharge near the abutment is another source of uncertainty. Two-dimensional models provide much better estimates of the unit discharge throughout the bridge opening than one-dimensional models. This is illustrated in **Figure 607-8**. Unit discharge can be calculated at any point in the two-dimensional flow field by multiplying velocity and depth. Although two-dimensional modeling is strongly recommended for bridge hydraulic design, HEC-23 (FHWA, 2009) includes a method for estimating the velocity at an abutment. This method is used to size abutment riprap but can also be used to determine the unit discharge at an abutment.



(FHWA, April 2012, HEC-18, Figure 8.11, p. 8.14)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12003.pdf>

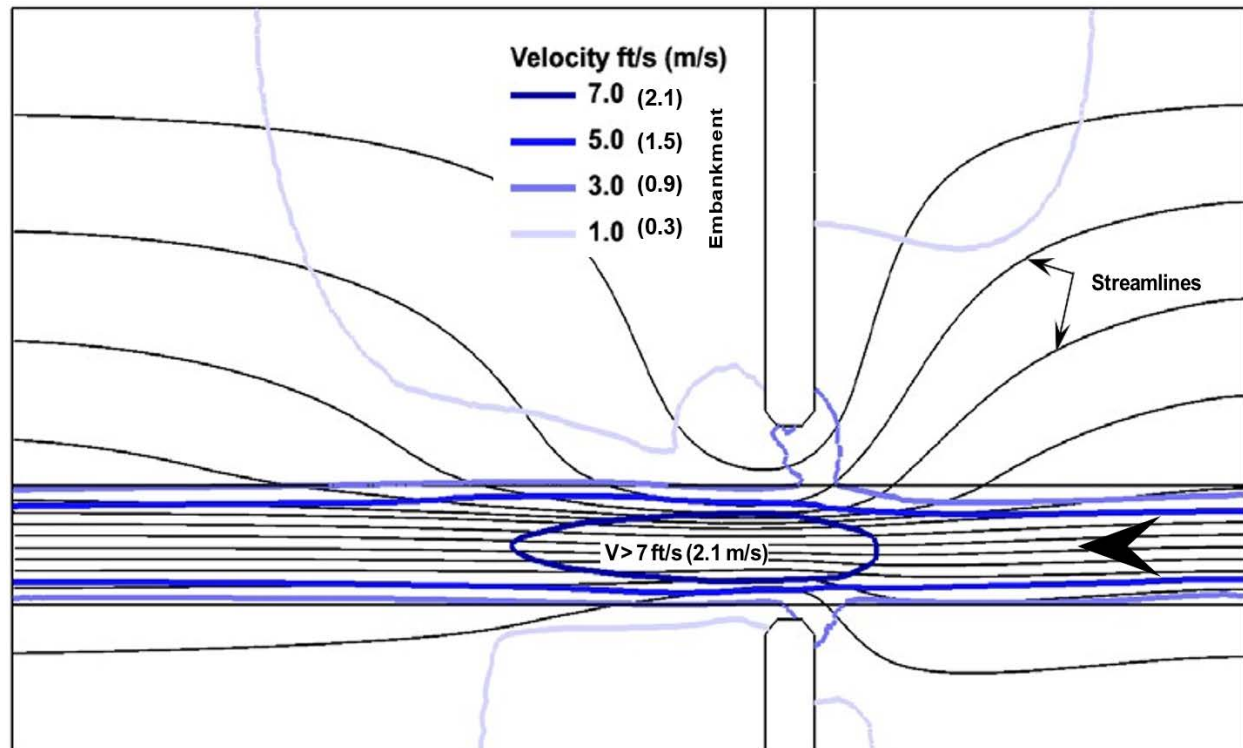
**Figure 607-6 Scour Amplification Factor for Spill-Through Abutments and Clear-water Conditions**



(FHWA, April 2012, HEC-18, Figure 8.12, p. 8.15)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12003.pdf>

**Figure 607-7 Scour Amplification Factor for Wingwall Abutments and Clear-water Conditions**



(FHWA, April 2012, HEC-18, Figure 8.13, p. 8.16)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12003.pdf>

### Figure 607-8 Velocity and Streamlines at a Bridge Constriction

The recommended procedure for selecting the velocity and unit discharge for abutment scour calculation is to use two-dimensional modeling. If one-dimensional modeling is used velocity and unit discharge are estimated as follows:

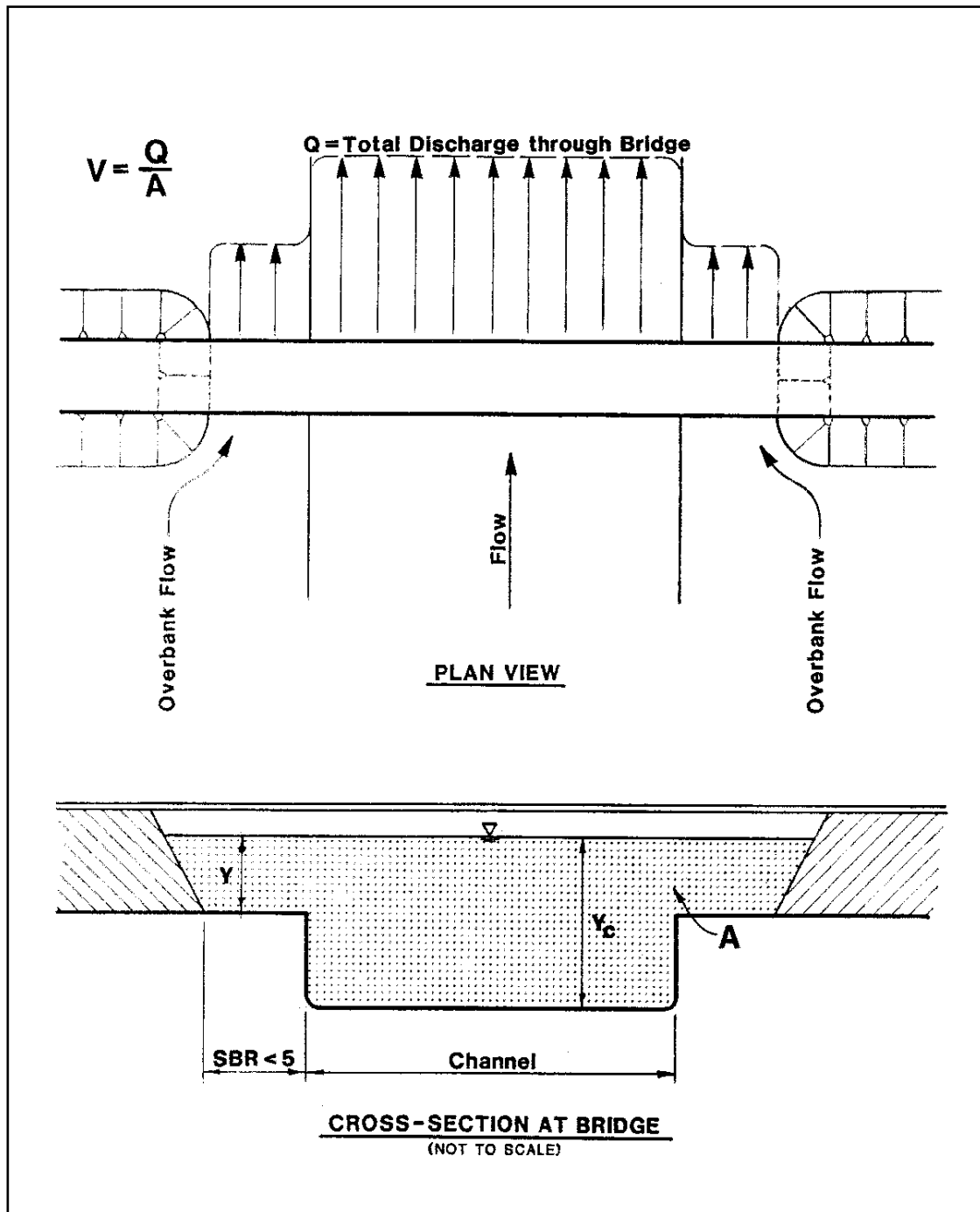
1. Determine the set-back ratio (SBR) of each abutment. SBR is the ratio of the set-back length to channel flow depth. The set-back length is the distance from the near edge of the main channel to the toe of abutment.

$$\text{SBR} = \text{Set-back length} / \text{average channel flow depth}$$

- a. If SBR is less than 5 for both abutments (**Figure 607-9**), compute a velocity,  $Q/A$ , based on the entire contracted area through the bridge opening. This includes the total upstream flow, exclusive of that which overtops the roadway. Unit discharge in the channel is the computed velocity times the channel flow depth. Unit discharge at the abutment is the computed velocity times the floodplain flow depth.
- b. If SBR is greater than 5 for an abutment (**Figure 607-10**), compute a velocity,  $Q/A$ , for the respective overbank flow only. Assume that the entire upstream overbank flow stays in the overbank section through the bridge opening. Unit discharge at the abutment is the computed velocity times the floodplain flow depth.



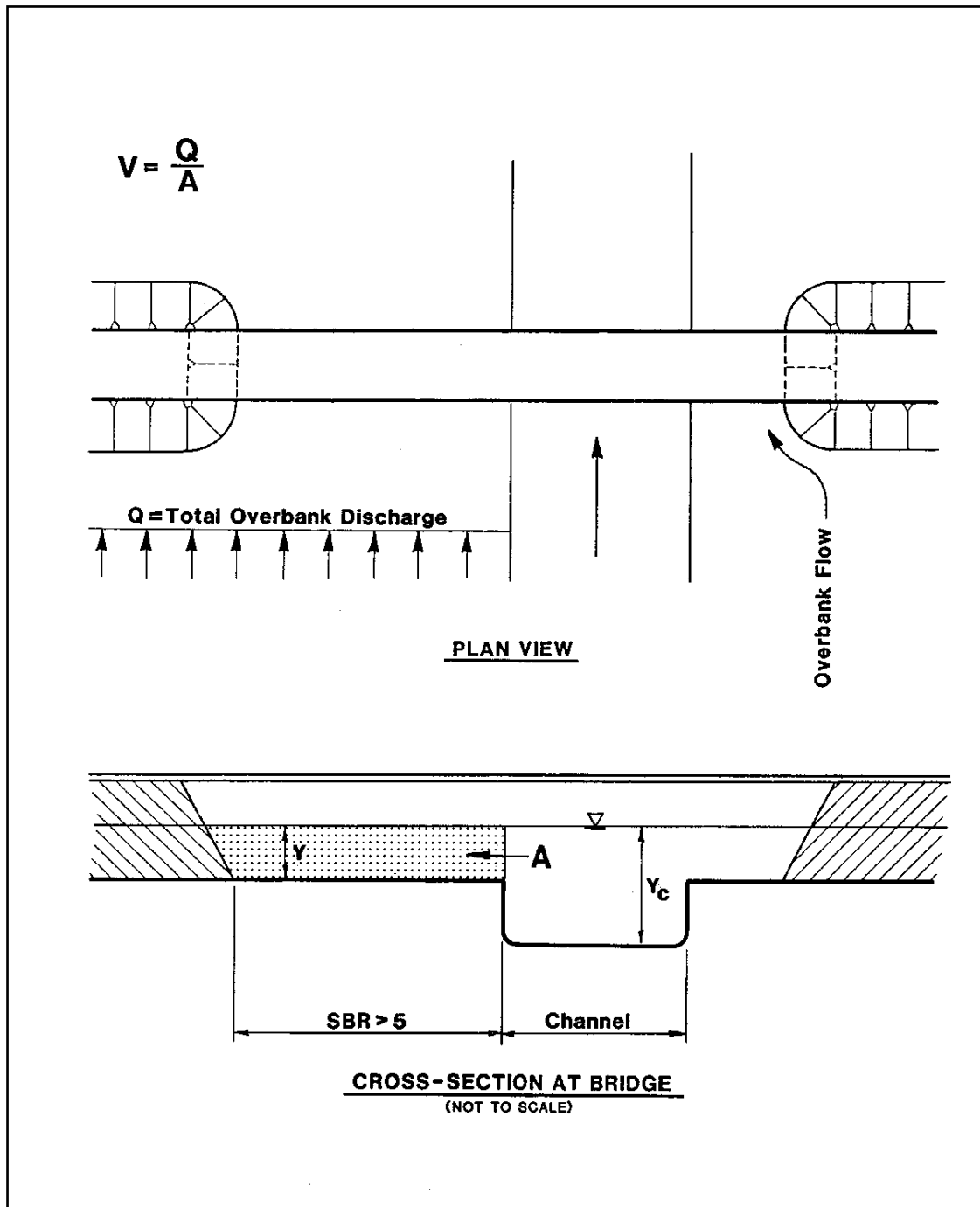
- c. If SBR for an abutment is less than 5 and SBR for the other abutment at the same site is more than 5 (**Figure 607-11**), a velocity determined from Step 1a for the abutment with SBR less than 5 may be unrealistically low. This would, of course, depend upon the opposite overbank discharge as well as how far the other abutment is set back. For this case, the velocity for the abutment with SBR less than 5 should be based on the flow area bounded by the abutment and the opposite channel bank. The appropriate discharge is the upstream channel flow and the upstream floodplain flow associated with that abutment. Unit discharge in the channel is the computed velocity times the channel flow depth and unit discharge at the abutment is the computed velocity times the floodplain flow depth.



(FHWA, April 2012, HEC-18, Figure 8.14, p. 8.17)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12003.pdf>

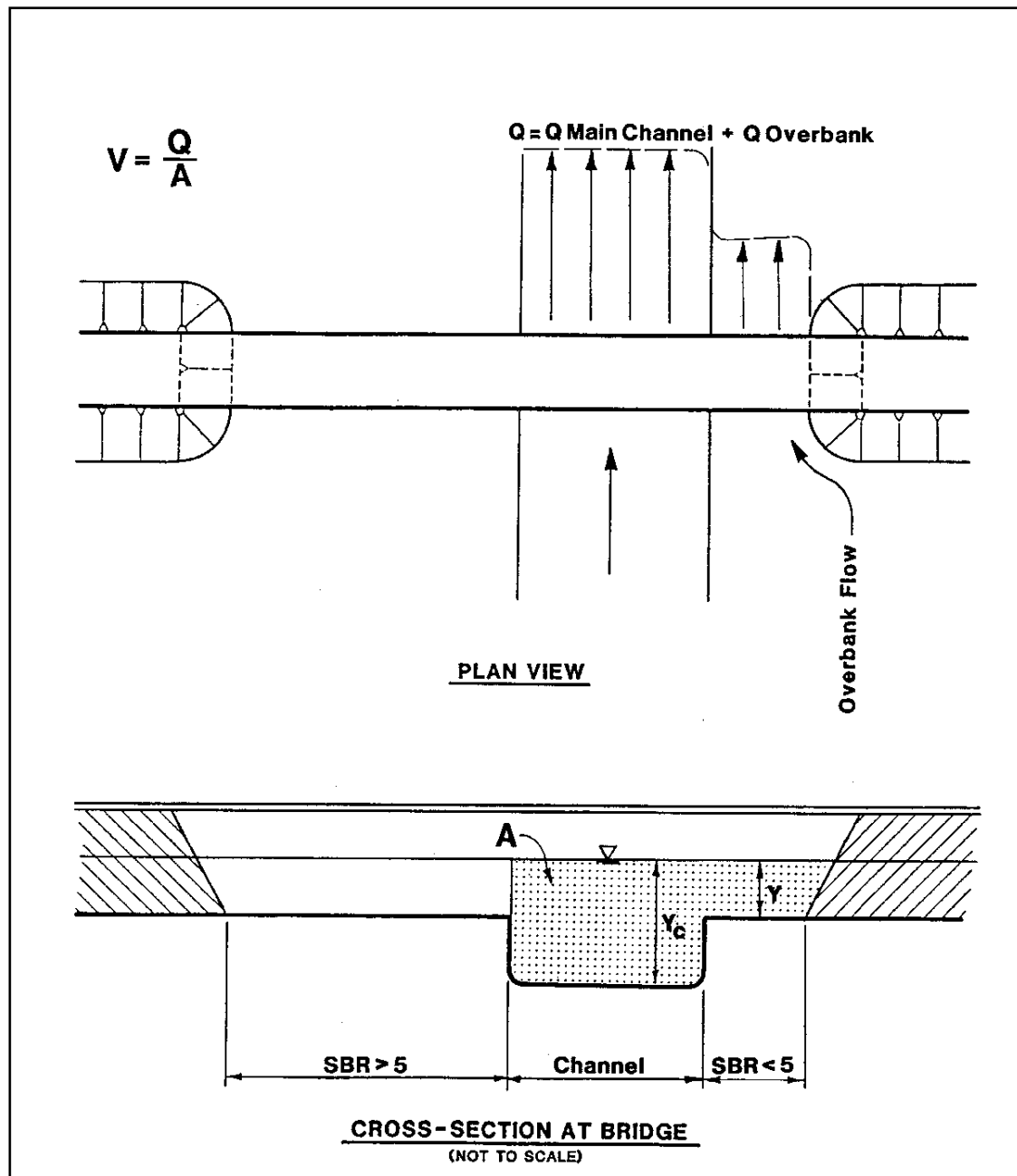
Figure 607-9 Velocity for  $SBR < 5$ .



(FHWA, April 2012, HEC-18, Figure 8.15, p. 8.18)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12003.pdf>

Figure 607-10 Velocity for SBR>5.



(FHWA, April 2012, HEC-18, Figure 8.16, p. 8.19)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12003.pdf>

**Figure 607-11 Velocity for SBR>5 and SBR<5.**

1. Recent research results published by the Transportation Research Board as NCHRP Report 587, "Countermeasures to Protect Bridge Abutments from Scour," endorse the use of the SBR approach for sizing riprap at spill-through abutments (NCHRP 2007). NCHRP Report 568, "Riprap Design Criteria, Recommended Specifications, and Quality Control," recommends an additional criterion for selecting the velocity

when applying the SBR Method (NCHRP 2006). Based on the results of 2-dimensional computer modeling of a typical abutment configuration, NCHRP Report 568 concludes:

- a. Whenever the SBR is less than 5, the average velocity in the bridge opening provides a good estimate for the velocity at the abutment.
- b. When the SBR is greater than 5, the recommended adjustment is to compare the velocity from the SBR Method to the maximum velocity in the channel within the bridge opening and select the lower velocity.
- c. The SBR Method is well suited for estimating velocity at an abutment if the estimated velocity does not exceed the maximum velocity in the channel.

### 607.3.5 Pier Scour

The Colorado State University (CSU) equation for pier scour (FHWA, February 1990) is:

$$\frac{y_s}{y_1} = 2.0 K_1 K_2 K_3 \left( \frac{a}{y_1} \right)^{0.65} Fr_1^{0.43} \quad 607-9$$

(FHWA, April 2012, HEC-18, Eq. 7.1, p. 7.3)

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12003.pdf>

where:

$y_s$	=	scour depth, ft
$y_1$	=	flow depth just upstream of the pier, ft
$K_1$	=	correction factor for pier nose shape from <b>Figure 607-12</b> and <b>Table 607-1</b>
$K_2$	=	correction factor for angle of attack of flow from Table 607-2 or <b>Equation 607-11</b>
$K_3$	=	correction factor for bed condition from <b>Table 607-3</b>
$a$	=	pier width, ft
$Fr_1$	=	Froude number directly upstream of the pier = $V_1 / (g y_1)^{0.5}$
$V_1$	=	mean velocity of flow directly upstream of the pier, ft/s
$g$	=	gravitational acceleration, 32.2 ft/s <sup>2</sup>
$L$	=	length of pier, ft

As a guideline, the maximum scour depth for round nose piers aligned with the flow is:

$$y_s \leq 2.4 \text{ times the pier width (a) for } Fr \leq 0.8$$

$$y_s \leq 3.0 \text{ times the pier width (a) for } Fr > 0.8$$

In terms of  $y_s/a$ , **Equation 607-9** is:

$$\frac{y_s}{a} = 2.0 K_1 K_2 K_3 \left( \frac{y_1}{a} \right)^{0.35} Fr_1^{0.43} \quad \text{607-10}$$

(FHWA, April 2012, HEC-18, Eq. 7.3, p. 7.3)

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12003.pdf>

where:

all other variables are previously defined

The correction factor,  $K_2$ , for angle of attack of the flow, is calculated using the equation:

$$K_2 = \left( \cos \theta + \frac{L}{a} \sin \theta \right)^{0.65} \quad \text{607-11}$$

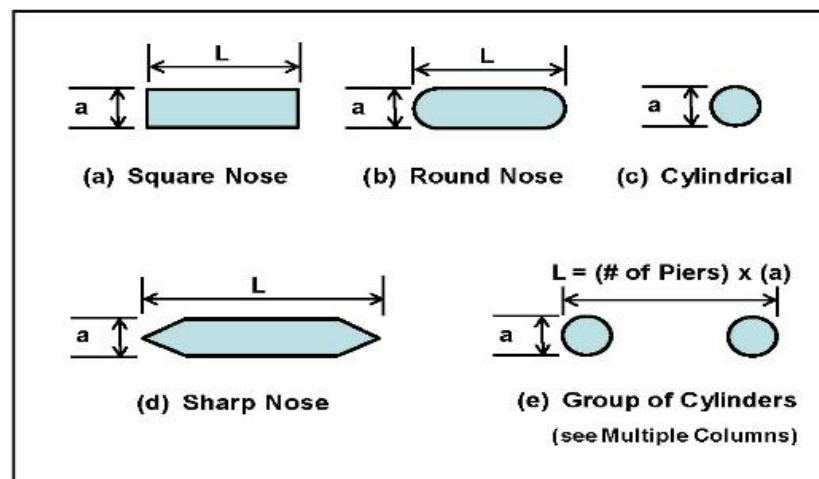
(FHWA, April 2012, HEC-18, Eq. 7.4, p. 7.4)

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12003.pdf>

where:

$K_2$	=	correction factor for angle of attack of flow
$L$	=	length of pier, ft
$a$	=	pier width, ft
$\theta$	=	angle, degree

If  $L/a$  is larger than 12, use  $L/a = 12$  as a maximum in **Equation 607-11** and **Table 607-2**. **Table 607-2** illustrates the magnitude of the effect of the angle of attack on local pier scour.



Source: FHWA, April 2012, HEC-18, Figure 7.3, p. 7.4.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12003.pdf>

**Figure 607-12 Common Pier Shapes**

**Table 607-1 Correction Factor,  $K_1$ , for Pier Nose Shape**

Source: FHWA, April 2012, HEC-18, Table 7.1, p. 7.4.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12003.pdf>

Shape of Pier Nose	$K_1$
(a) Square nose	1.1
(b) Round nose	1.0
(c) Circular cylinder	1.0
(d) Sharp nose	0.9
(e) Group of cylinders	1.0

**Table 607-2 Correction Factor,  $K_2$ , Angle  $\theta$  Degree for Angle of Attack of the Flow**

Source: FHWA, April 2012, HEC-18, Table 7.2, p. 7.4.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12003.pdf>

Angle $\theta$ Degrees	$L / a = 4$	$L / a = 8$	$L / a = 12$
0	1.0	1.0	1.0
15	1.5	2.0	2.5
30	2.0	2.75	3.5
45	2.3	3.3	4.3
90	2.5	3.9	5.0
Angle = skew angle of flow			
L = length of pier parallel to flow, ft			

**Table 607-3 Increase in Equilibrium Pier Scour Depths,  $K_3$ , for Bed Condition**

Source: FHWA, April 2012, HEC-18, Table 7.3, p. 7.5.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12003.pdf>

Bed Condition	Dune Height (ft)	$K_3$
Clear-Water Scour	N/A	1.1
Plane Bed and Antidune Flow	N/A	1.1
Small Dunes	$10 > H \geq 2$	1.1
Medium Dunes	$30 > H \geq 10$	1.2 to 1.1
Large Dunes	$H \geq 30$	1.3

## Notes:

1. The correction factor  $K_1$  for pier nose shape should be determined using **Table 607-1** for angles of attack up to 5 degrees. For greater angles,  $K_2$  dominates and  $K_1$  should be considered as 1.0. If  $L/a$  is larger than 12, use the values for  $L/a = 12$  as a maximum in and **Equation 607-11**.
2. The value of the correction factor  $K_2$  should be applied only when the field conditions are such that the entire length of the pier is subjected to the angle of attack of the flow. Use of this factor directly from the table will result in a significant over-prediction of scour if: (1) a portion of the pier is shielded from the direct impingement of the flow by an abutment or another pier, or (2) an abutment or another pier redirects the flow in a direction parallel to the pier. For such cases, judgment must be exercised to reduce the value of the  $K_2$  factor by selecting the effective length of the pier actually subjected to the angle of attack of the flow. **Equation 607-11** should be used for evaluation and design and is intended to illustrate the importance of angle of attack in pier scour computations and to establish a cutoff point for  $K_2$  (i.e. a maximum value of 5.0).
3. The correction factor  $K_3$  (**Table 607-3**) results from the fact that for plane-bed conditions, which is typical of most bridge sites for the flood frequencies employed in scour design, the maximum scour may be ten percent greater than computed with **Equation 607-9**. In the unusual situation where a dune-bed configuration with large dunes exists at a site during flood flow, the maximum pier scour may be thirty percent greater than the predicted equation values. This may occur on very large rivers, such as the Mississippi. For smaller streams that have a dune-bed configuration at flood flow, the dunes will be smaller, and the maximum scour may be only ten to twenty percent larger than equilibrium scour. For antidune-bed configuration the maximum scour depth may be ten percent greater than the computed equilibrium pier scour depth.
4. Piers set close to abutments (for example at the toe of a spill through abutment) must be carefully evaluated for the angle of attack and velocity of the flow coming around the abutment.
5. Engineering judgement and observations of debris on existing piers should be used when determining the appropriate pier width to use in analysis. For bridges with large trees upstream, a larger width than true pier width will be appropriate. Several simulation runs can be prepared, assuming different pier debris widths, to determine the sensitivity of the analysis.



### 607.3.6 Scour for Complex Pier Foundations

The pier scour computations can involve several items, such as:

- Superposition of scour component methods
- Determination of pier stem scour depth component
- Determination of the pile cap (footing) scour depth component
- Plotting pile cap (footing) in flow above the bed
- Bottom of pile cap (footing) located on or below the bed
- Other considerations such as skew, etc.

Please refer to HEC-18 (FHWA, April 2012) for complete discussions of variations of the scour equations such as for pressure scour, and many equations for pier scour computations.

Appendix 8 contains **Example Problem 8-2** that presents bridge scour analyses and scour countermeasure recommendations for a real bridge.

### 607.4 References

FHWA, February 1990, "Highways in the River Environment".

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hire1990.pdf>

FHWA, April 2012, "HEC-18, Evaluating Scour at Bridges, Fifth Edition".

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12003.pdf>

FHWA, April 2012, "HEC-20, Stream Stability at Highway Structures, Fourth Edition".

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12004.pdf>

Laursen, E.M., 1960, "Scour at Bridge Crossings", Journal Hydraulic Division, American Society of Civil Engineers, Vo. 86, No. HY2.

Laursen, E.M., 1963, "An Analysis of Relief Bridge Scour", Journal Hydraulic Division, American Society of Civil Engineers, Vo. 89, No. HY3.

## **608 Selection of Countermeasures for Stream Instability**

### **608.1 Introduction**

A countermeasure is defined as a feature incorporated into a highway stream-crossing system to monitor, control, inhibit, change, delay, or minimize stream and bridge stability problems or it is the action plan for monitoring structures during and/or after flood events. This includes river stabilizing works over a reach of the river upstream and downstream of the crossing.

Countermeasures may be installed at the time of highway construction or retrofitted to resolve stability problems at existing crossings. Retrofitting is good economics and good engineering practice in many locations because the magnitude, location, and nature of potential stability problems are not always discernible at the design or construction stage and may take several years to develop. Also, a countermeasure may not be a separate structure, but may be an integral part of the highway. For example, relief bridges on floodplains are countermeasures which alleviate general scour from flow contraction at the bridge over the stream channel. Some features that are integral to the highway design serve as countermeasures to minimize stream stability problems. Abutments and piers oriented with the flow direction achieve the most efficient utilization of the available waterway to convey flow and reduce local scour and scour due to contraction.

Countermeasures which are not integral to the highway may serve one function at one location and a different function at another. For example, bank revetment may be installed to control bank erosion from meander migration, or it may be used to stabilize stream banks in the contracted area at a bridge. Other countermeasures are useful for only one function. This category of countermeasures includes spurs constructed in the stream channel to control meander migration.

### **608.2 Considerations for the Selection of Countermeasures**

The selection of an appropriate countermeasure for a specific bank erosion problem is dependent on factors such as the erosion mechanism, stream characteristics, construction and maintenance requirements, potential for vandalism, and costs. Perhaps a more important consideration is the effectiveness of the countermeasure selected in performing the required function.

Protection of an existing bank may be accomplished with revetments, spurs, retardance structures, longitudinal dikes, or bulkheads. Spurs, longitudinal dikes, and area retardance structures can be used to establish a new flow path and channel alignment, or to constrict flow in a channel. Bulkheads may be used for any of the functions, but because of their high cost, are appropriate for use only where space is available. Channel relocation may be used separately or in conjunction with other countermeasures to change the flow path and flow orientation.

#### **Erosion Mechanism**

Bank erosion mechanisms include surface erosion and/or mass wasting. Surface erosion is the removal of soil particles by the velocity and turbulence of flowing water. Mass wasting is caused by slides, rotational slip, piping, and block failure. In general, slides, rotational slip, and block

failure result from the bank being undercut by the flow. Also, seepage force of the pore water in the bank is another factor that can cause surface erosion or mass wasting. The type of mechanism is determined by the magnitude of the erosive forces of the water, type of bed and bank material, vegetation, and gradation stability of the stream. These mechanisms are described in “Highways in the River Environment” (FHWA, February 1990).

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hire1990.pdf>

### **Stream Characteristics**

Stream characteristics that influence the selection of countermeasures include: channel width, bank height, channel configuration and material, vegetative cover, channel bed sediment transport condition, bend radii, channel velocities and flow depth, ice and debris, and floodplains.

Channel Width - Channel width can limit the use of spur-type countermeasures. On smaller streams (< 250 ft wide), flow constriction resulting from the use of spurs may cause erosion of the opposite bank. However, spurs can be used on small channels where the purpose is to shift the location of the channel.

Bank Height - Low banks (<10 ft) may be protected by any of the countermeasures including bulkheads. Medium height banks (from 10 to 20 ft) may be protected by revetment, retardance structures, spurs, and longitudinal dikes. High banks (> 20 ft) generally require revetments used alone or in conjunction with other measures.

Channel Configuration - Spurs and jack fields have been successfully used as a countermeasure to control the location of the channel in meandering and braided streams. Also, bulkheads, revetments, and riprap have been used to control bank erosion resulting from stream migration. On anabranching streams, revetments, riprap, and spurs have been used to control bank erosion and channel shifting. Also, channels that do not carry large flows can be closed off.

Channel Material - Spurs, revetments, riprap, jack fields, or check dams can be used in any type of channel material if they are designed correctly. However, jack fields should only be placed on streams that carry appreciable debris and sediment in order for the jacks to cause deposition and eventually be covered up.

Bank Vegetation - Vegetation, such as willows, can enhance the performance of structural countermeasures and may, in some cases, reduce the level of structural protection needed. Meander migration and other bank erosion mechanisms are accelerated on many streams in reaches where vegetation has been cleared.

Sediment Transport - Sediment transport conditions can be described as regime, threshold, or rigid. Regime channel beds are those which are in motion under most flow conditions, generally in sand or silt-size non-cohesive materials. Threshold channel beds have no bed material transport at normal flows and become mobile at higher flows. They may be cut through cohesive or noncohesive materials. An armor layer of coarse-grained material can develop on the channel bed. Rigid channel beds are cut through rock or boulders and rarely become mobile. In general, permeable structures will cause deposition of bed material in transport and are better suited for use in regime and some threshold channels than in rigid channel conditions. Impermeable structures are more effective than permeable structures in channels with little or

no bed load, but impermeable structures can also be effective in mobile bed conditions. Revetments can be effectively used with mobile or immobile channel beds.

Bend Radii - Bend radii affect the design of countermeasures. The cost-per-foot of bank protection provided by a specific countermeasure may differ considerably on short-radius as compared to longer radius bends.

Channel Velocities and Flow Depth - Channel hydraulics affect countermeasure selection because structural stability and induced scour must be considered. Some of the permeable flow retardance measures may not be structurally stable. Also, countermeasures which utilize piles may be susceptible to scour failure in high velocity environments.

Ice and Debris - Ice and debris can damage or destroy countermeasures; therefore, this damage should always be considered during the selection process. On the other hand, the performance of some permeable spurs and area retardance structures is enhanced by debris, where debris accumulation causes increased sediment deposition.

Floodplains - In selecting countermeasures for stream stability and scour, the amount of flow on the floodplain is an important factor. For example, if there is appreciable overbank flow, then guide banks to protect abutments should be considered. Also, spurs perpendicular to the approach embankment may be needed to control erosion.

### **Construction and Maintenance Requirements**

Standard requirements regarding construction or maintenance such as the availability of materials, construction equipment requirements, site accessibility, time of construction, contractor familiarity with construction methods, and a program of regular maintenance, inspection, and repair are applicable to the selection of appropriate countermeasures. Additional considerations for countermeasures that are located in stream channels include considerations of constructing and maintaining a structure which may be partially under water at all times, the extent of bank disturbance that may be necessary, and the desirability of preserving stream bank vegetative cover to the extent practicable.

### **Vandalism**

Vandalism is always a maintenance concern since effective countermeasures can be rendered ineffective by vandals. Documented vandalism includes dismantling of devices, burning, and cutting or chopping with knives, wire cutters, or axes. Countermeasure selection or material selection for construction may be affected by concern for vandalism. For example, rock-filled baskets (gabions) may not be appropriate in some urban environments.

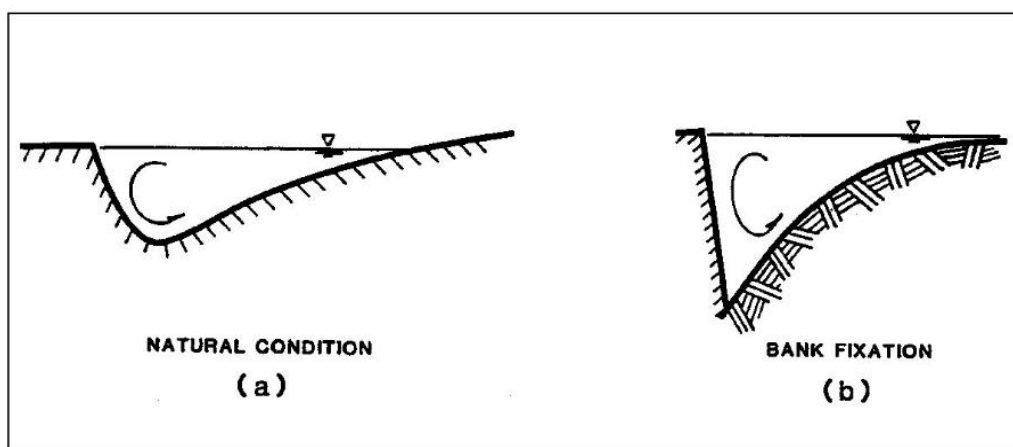
### **Costs**

Cost comparisons should be included in the study of alternative countermeasures with an understanding that the measures were installed under widely varying stream conditions, that the conservatism (or lack thereof) of the engineer is not accounted for, that the relative effectiveness of the measures cannot be quantitatively evaluated, and that some measures included in the cost data may not have been fully tested by floods.

### 608.3 Countermeasures for Meander Migration

The best countermeasure against meander migration is to locate the bridge crossing on a relatively straight reach of the stream between bends. At many such locations, countermeasures may not be required for several years because of the time required for the bend to move to a location where it becomes threatening to the highway facility. However, bend migration rates on streams may be at such a rate that countermeasures will be required after a few years or a few flood events. Therefore, these countermeasures should be installed during initial construction.

Stabilizing channel banks at a highway stream crossing can cause a change in the channel cross section and an increase in stream sinuosity upstream of the stabilized banks. **Figure 608-1 (a)** illustrates a natural channel section in a bend with the deeper section at the outside of the bend and a gentle slope toward the inside bank resulting from deposition in the remainder of the section. **Figure 608-1 (b)** illustrates the scour which results from stabilizing the outside bank of the channel and the steeper slope of the point bar on the inside of the bend. This effect must be considered in the design of the countermeasure and the bridge. It should also be recognized that the thalweg (lowest watercourse invert) location and flow direction can change as sinuosity upstream increases.



Source: FHWA, September 2009, HEC-23, Figure 3.2, p. 3.10.

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/09111/09111.pdf>

**Figure 608-1 Comparison of Channel Bend Cross Sections (a) for Natural Conditions and (b) for a Stabilized Bend**

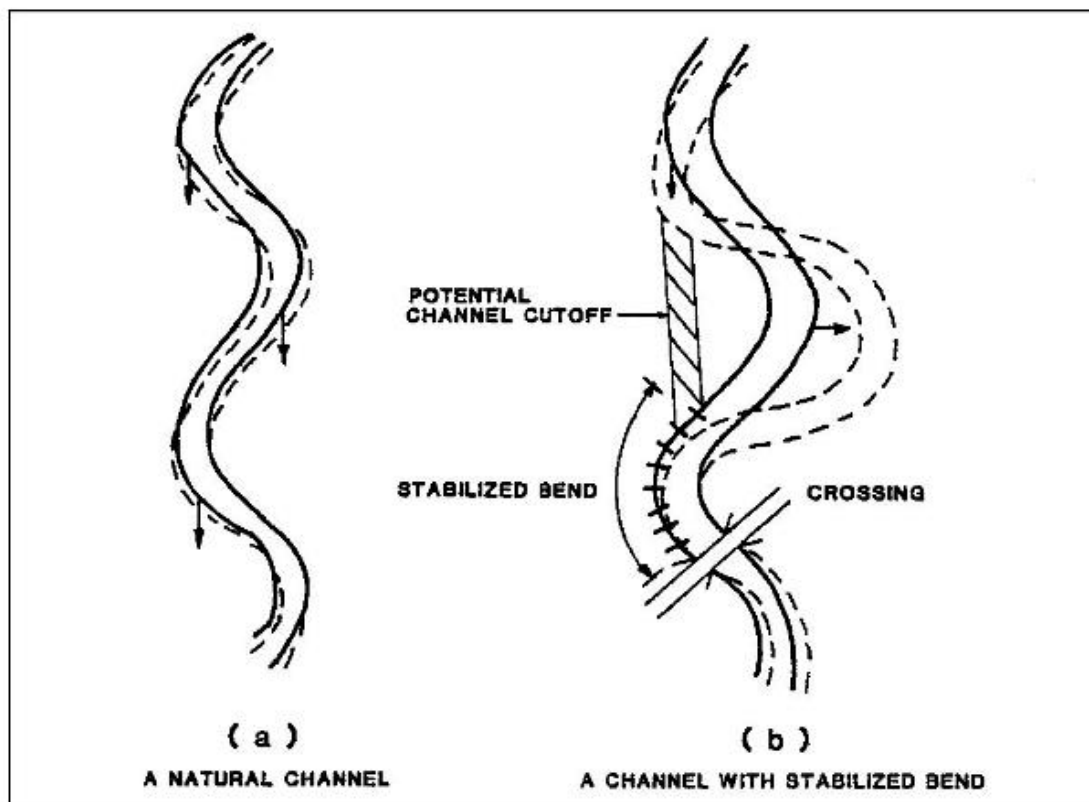
**Figure 608-2 (a)** illustrates meander migration in a natural stream and **Figure 608-2 (b)** the effects of bend stabilization on upstream sinuosity. As sinuosity increases, meander amplitude may increase, meander radii may become smaller, deposition may occur because of reduced slopes, and the channel width-depth ratio may increase as a result of bank erosion and deposition, as at the bridge location shown in **Figure 608-2 (b)**. Ultimately, cutoffs can occur. These changes can also result due to changing hydraulics downstream of the stabilized bend.

Countermeasures for meander migration include:

- Protect an existing bank line,

- Establish a new flow line or alignment, and
- Control and constrict channel flow

Countermeasures identified for bank stabilization and bend control are bank revetments, spurs, retardance structures, longitudinal dikes, vane dikes, bulkheads, and channel relocations. Also, a carefully planned cutoff may be an effective way to counter problems created by meander migration. These measures may be used individually or in combination to combat meander migration at a site. Some of these countermeasures are also applicable for bank erosion from causes other than bend migration. Refer to HEC-23 for more information on countermeasures (FHWA, September 2009).



Source: FHWA, September 2009, HEC-23, Figure 3.3, p. 3.11.

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/09111/09111.pdf>

**Figure 608-2 Meander Migration in (a) a Natural Channel, and (b) a Channel with Stabilized Bend**

#### **608.4 Countermeasures for Scour at Bridges**

Scour is the result of the erosive action of running water excavating and carrying-away material from the bed and banks of streams. Different materials scour at different rates. Loose granular soils are rapidly eroded under water action while cohesive or cemented soils are more scour-resistant. However, ultimate scour in cohesive or cemented soils can be as deep as scour in sand bed streams. Scour will reach its maximum depth in sand and gravel bed materials in

hours; cohesive bed materials in days; glacial tills, poorly cemented sand stones and shales in months; hard, dense and cemented sandstone or shales and limestones in years; and dense granites in centuries. Massive rock formations with few discontinuities can be highly resistant to scour and erosion during the lifetime of a typical bridge.

Engineers and inspectors need to carefully study site-specific subsurface information in evaluating scour potential at bridges, giving particular attention to foundations on rock.

Total Scour - Total scour at a highway crossing is comprised of three components. These components are:

Aggradation and Degradation - These represent long-term stream bed elevation changes due to natural or man-induced causes within the reach of the river on which the bridge is located.

Contraction Scour - This type of scour involves the removal of material from the bed and banks across all or most of the width of a channel. This scour can result from a contraction of the flow by the approach embankments to the bridge encroaching onto the floodplain and/or into the main channel, a change in downstream control of the water surface elevation, or the location of the bridge in relation to a bend. In each case, the scour is caused by an increase in transport of the bed material in the bridge cross section.

Local Scour - This scour occurs around piers, abutments, spurs, and embankments and is caused by the acceleration of the flow and the development of vortices induced by these obstructions to the flow.

In addition to the types of scour mentioned above, lateral movement of the stream may also erode the approach roadway to the bridge or change the total scour by changing the angle of the flow in the waterway at the bridge crossing. Factors that affect lateral movement and the stability of a bridge are the geomorphology of the stream, location of the crossing in the stream, flood characteristics, and the characteristics of the bed and bank materials.

#### Fail Safe Bridge Design

The fail-safe design alternative is a single span structure which spans stream flow at the flood stage. This includes a superstructure that will not be submerged or partially submerged at the flood stage. Obviously, this may only be practicable for small, relatively stable, incised streams.

Please note that a single span bridge may not be a viable alternative at some alluvial stream crossing sites. However, where the alternative is available and practicable, consideration should be given to making the crossing or bridge fail-safe.

An alternative, which is necessary for all bridges over alluvial streams, is to locate the bridge foundation, pile tips, or drilled shafts, at an elevation at which sufficient support will be maintained after scour occurs. Where pile bearing capacity is based on skin friction, driving the piles to refusal may be inadequate. Pile tip elevation should be applied as a second criterion, and tip elevation should be based on the bearing capacity after scour.

As a practical matter, fail-safe bridge designs are usually not feasible, but strategies against scour are available. Countermeasures are appurtenances to the highway stream-crossing system. Design alternatives and countermeasures are both discussed in detail in HEC-23 (FHWA, September 2009).

### **608.5 Countermeasures for Channel Braiding and Anabranching**

Channel braiding occurs in streams with an overload of sediment, causing deposition and aggradation. As aggradation occurs, the slope of the channel increases, velocities increase, and multiple, interlaced channels develop. The overall channel system becomes wider and multiple channels are formed as bars of sediment are deposited in the main channel. Braiding can also occur where banks are easily eroded and there is a large range in discharge. The channel becomes wider at high flows, and low flows form multiple interlaced channels. In an anabranching stream, flow is divided by islands rather than bars, and the anabranches are more permanent than braided channels and generally convey more flow.

Meandering streams may change to a braided stream if slope is increased by channel straightening or if the dominant discharge is increased. Lane's (1955) relation may be used to determine if there can be a shift from a meandering channel to a braided one. If, after a change in discharge or slope, the stream still plots in the meandering zone, then it will remain a meandering stream. However, if it moves closer to or into the braided zone, then the stream may become braided.

Braided channels change alignment rapidly and are wide and shallow even at flood flow. They present problems at bridge sites because of the high cost of bridging the complete channel system, unpredictable channel locations and flow directions, difficulties with eroding channel banks, and difficulty in maintaining bridge openings unobstructed by bars and islands.

Countermeasures used on braided and anabranching streams are usually intended to confine the multiple channels to one channel. This tends to increase sediment transport capacity in the principal channel and encourage deposition in secondary channels. These measures usually consist of dikes constructed from the limits of the multiple channels to the channel over which the bridge is constructed. Guide banks directly upstream and downstream of bridges used in combination with revetment on highway fill slopes, riprap on highway fill slopes, and spurs arranged in the stream channels to constrict flow to one channel, have also been used successfully.

Since anabranches are permanent channels that may convey substantial flow, diversion and confinement of an anabranching stream is likely to be more difficult than for a braided stream. The engineer may be faced with a choice of designing either more than one bridge, a long bridge, or diverting anabranches into a single channel to allow a single span bridge.

### **608.6 Countermeasures for Degradation and Aggradation**

Degradation and aggradation problems are common in alluvial streams. Degradation in streams can cause the loss of bridge piers in stream channels and can contribute to the loss of piers and abutments located on caving banks. Aggradation causes the loss of the waterway opening and thus flow conveyance in bridges. Where channels become wide because of aggrading stream beds, overbank piers and abutments can be undermined. Aggradation in severe cases may cause streams to abandon their original channels and establish new flow paths which may sever highways from the existing bridge.



### **608.6.1 Countermeasures to Control Degradation**

The type of countermeasures used to control degradation depend on the structure in question, the watercourse location, the erosion or anticipated erosion and physical site limitations such as right-of-way constraints.

Due to the enormous volume of countermeasure design information available for specific locations and situations, please refer to HEC-23 (FHWA, September 2009, Volumes 1 and 2, hotlinks are provided below) for specific countermeasure design guidance for:

- **Stream Instability**
  - Bendway Weirs/Stream Barbs
  - Spurs
  - Check Dams/Drop Structures
- **Streambank and Roadway Embankment Protection**
  - Riprap Revetment
  - Riprap Design for Embankment Overtopping
  - Wire Enclosed Riprap Mattresses
  - Soil Cement
  - Articulating Concrete Block Systems
  - Grout-Filled Mattresses
  - Gabion Mattresses
- **Countermeasures for Bridge Pier Protection**
  - Articulating Concrete Block Systems at Bridge Piers
  - Grout Filled Mattresses at Bridge Piers
  - Gabion Mattresses at Bridge Piers
  - Partially Grouted Riprap at Bridge Piers
- **Countermeasures for Abutment Protection**
  - Grout/Cement Filled Bags
  - Rock Riprap at Bridge Abutments
  - Guide Banks
- **Filter Design**
  - Granular Filters
  - Geotextile Filters

FHWA, September 2009, "HEC-23, Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance, Third Edition, Volume 1".

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/09111/09111.pdf>

FHWA, September 2009, "HEC-23, Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance, Third Edition, Volume 2".

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/09111/09112.pdf>

### **608.6.2 Countermeasures to Control Aggradation**

Currently, measures used in the attempt to alleviate aggradation problems at highways include channelization, debris basins, bridge modification, and/or continued maintenance, or a combination of these. Channelization may include excavating and clearing channels, constructing small dams to form debris basins, constructing cutoffs to increase the local slope, constructing flow control structures to reduce and control the local channel width, and constructing relief channels to improve flow capacity at the crossing. Except for debris basins and relief channels, these measures are intended to increase the sediment transport capacity of the channel, thus reducing or eliminating problems with aggradation. Cutoffs must be designed after considerable study as they can cause erosion upstream and deposition downstream. These studies would involve the use of sediment transport relations or the use of sediment transport models. The most common bridge modifications are increasing the bridge length by adding spans and increasing the effective flow area beneath the structure by raising the bridge deck.

A program of regular maintenance has been successfully used to control problems at bridges on aggrading streams. In such a program, a monitoring system is usually set up to survey the affected crossing at regular intervals. When some pre-established deposition depth is reached, the bridge opening is dredged or cleared of the deposited material. In some cases, this requires opening a clearing after every major flood. This solution requires surveillance and dedication to the continued maintenance of an adequate waterway under the bridge. Otherwise, it is only a temporary solution. A debris basin or a deeper channel upstream of the bridge may be easier to maintain.

Over the short term, maintenance programs prove to be cost-effective when compared with the high cost of channelization, bridge alterations, or relocations. However, when costs over the entire life of the structure are considered, maintenance programs may cost more than some of the initially more expensive measures. Also, the reliability of maintenance programs is generally low because maintenance programs are often abandoned for budgetary or priority reasons. A program of regular maintenance could prove to be the most cost-efficient solution if analysis of the transport characteristics and sediment supply in a stream system reveals that the aggradation problem is only temporary (perhaps the excess sediment supply is coming from a construction site) or will have only minor effects over a relatively long period of time.

An alternative similar to a maintenance program, which could be used on streams with persistent aggradation problems, such as those on alluvial fans, is the use of controlled sand and gravel mining from a debris basin constructed upstream of the bridge site. Use of this alternative would require careful analysis to ensure that the gravel mining does not upset the balance of sediment and water discharges downstream of the debris basin. Excessive mining could produce a degradation profile downstream, potentially impacting the bridge or other structures.

The following is a condensed list of guidelines regarding aggradation countermeasures:

- Extensive channelization projects have generally proven unsuccessful in alleviating general aggradation problems although some successful cases have been documented.
- A sufficient increase in the sediment carrying capacity of the channel is usually not achieved to significantly reduce or eliminate the problem. Channelization should be considered only if analysis shows that the desired results will be achieved.

- Alteration or replacement of a bridge is often required to accommodate maximum aggradation depths.
- Maintenance programs have proved unreliable, but they may provide the most cost-effective solution where aggradation is from a temporary source or on small channels where the problem is limited in magnitude.
- At aggrading sites on wide, shallow streams, spurs or dikes with flexible revetment have been successful in several cases in confining the flow to narrower, deeper sections.
- A debris basin and controlled sand and gravel mining might be the best solution at alluvial fans and other crossings with severe problems.

### **608.7 Countermeasures for Outlet Protection**

There are two types of erosion that can occur: local scour and channel degradation. Local scour is the erosion of soil near an obstruction in the flow path. The obstruction causes local velocities to increase and scour occurs until the scour hole cross section becomes large enough so that flow velocities around the obstruction equal the flow velocity in the remainder of the channel. Channel degradation is erosion of the channel bed and banks. It is dependent on the geometry of the channel, particularly slope, flow regime, and downstream activities that may generate a headcut to the culvert.

Local scour at a culvert outlet can also occur because of the higher velocities produced in a culvert. The outlets of culverts and lined channels are susceptible to erosion if not properly protected. To prevent scour at stormwater outlets, an energy dissipator is needed to absorb the initial impact of the flow and reduce the flow velocity to a level that minimizes erosion of the receiving channel or area. **Table 608-1** presents energy dissipator types and limitations. Additional information for each of the energy dissipators listed may be found in HEC-14 (FHWA, July 2006).

**Table 608-1 Energy Dissipators and Limitations**

Source: FHWA, July 2006, HEC-14, Table 1.1, p. 1-2.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/06086/hec14.pdf>

Dissipator Type	Froude Number <sup>7</sup>	Allowable Debris <sup>1</sup>			Tailwater (TW)
		Silt/Sand	Boulders	Floating	
Flow transitions	N/A	H	H	H	Desirable
Scour hole	N/A	H	H	H	Desirable
Hydraulic jump	>1	H	H	H	Required
Tumbling flow <sup>2</sup>	>1	M	L	L	Not needed
Increased resistance <sup>3</sup>	N/A	M	L	L	Not needed
USBR Type IX baffled apron	<1	M	L	L	Not needed
Broken-back culvert	>1	M	L	L	Desirable
Outlet weir	2 to 7	M	L	M	Not needed
Outlet drop/weir	3.5 to 6	M	L	M	Not needed
USBR Type III Stilling Basin	4.5 to 17	M	L	M	Required
USBR Type IV Stilling Basin	2.5 to 4.5	M	L	M	Required
SAF stilling basin	1.7 to 17	M	L	M	Required
CSU rigid boundary basin	< 3	M	L	M	Not needed
Contra Costa basin	<3	H	M	M	< 0.5 D
Hook basin	1.8 to 3	H	M	M	Not needed
USBR Type VI impact basin <sup>4</sup>	N/A	M	L	L	Desirable
Riprap basin	<3	H	H	H	Not needed
Riprap apron <sup>8</sup>	N/A	H	H	H	Not needed
Straight drop structure <sup>5</sup>	<1	H	L	M	Required
Box inlet drop structure <sup>6</sup>	<1	H	L	M	Required
USACE stilling well	N/A	M	L	N	Desirable

<sup>1</sup>Debris notes: N = none, L = low, M = moderate, H = heavy<sup>2</sup>Bed slope must be in the range  $4\% < S_o < 25\%$ <sup>3</sup>Check headwater for outlet control<sup>4</sup>Discharge,  $Q < 400 \text{ ft}^3/\text{s}$  and Velocity,  $V < 50 \text{ ft/s}$ <sup>5</sup>Drop < 15 feet<sup>6</sup>Drop < 12 feet<sup>7</sup>At release point from culvert or channel<sup>8</sup>Culvert rise less than or equal to 60 inches

N/A = not applicable

### 608.7.1 Aprons for Culvert Outlet Protection

The most commonly used method for outlet protection is a lined apron. These aprons are generally designed with riprap, grouted riprap, gabions or concrete. They may be constructed at a zero grade for a distance or at minimal grade.

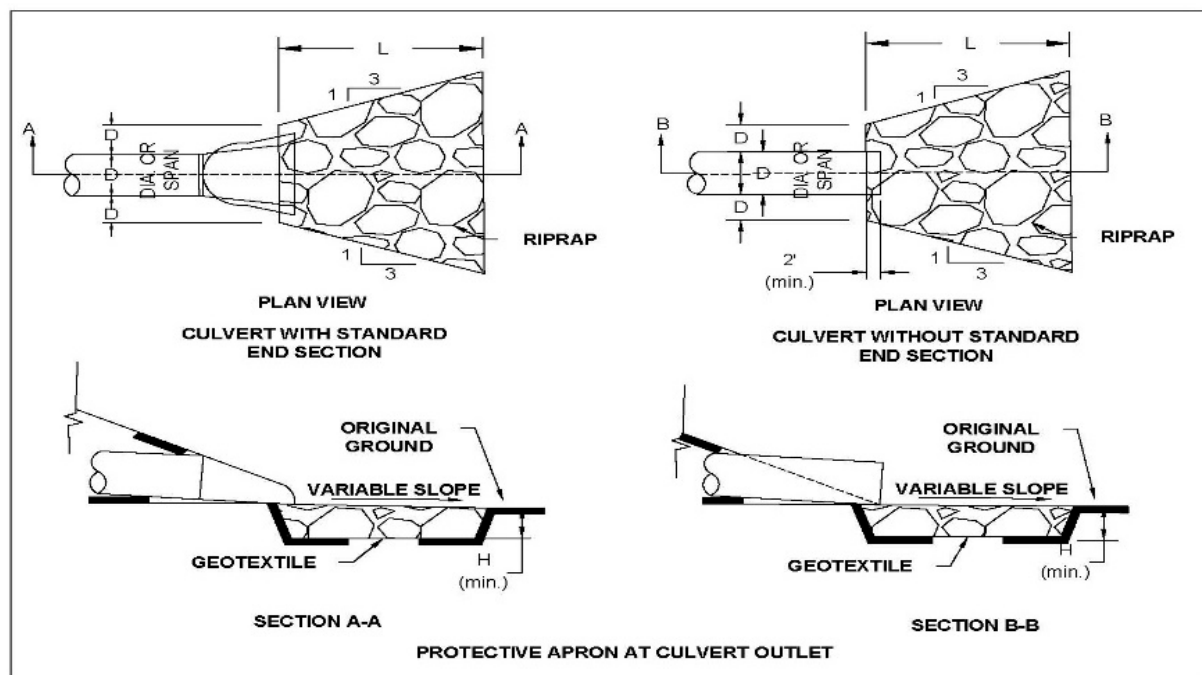
#### Riprap Aprons

Riprap aprons are the most common outlet protection designed for culverts 60 inches or smaller due to ease of construction and cost benefits over other methods. Wire-enclosed riprap aprons or gabions may also be considered if designed in the same manner as the riprap apron presented here. The following procedure is from HEC-14 (FHWA, July 2006).

FHWA, July 2006, "HEC-14, Hydraulic Design of Energy Dissipators for Culverts and Channels, Third Edition".

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/06086/hec14.pdf>

An example schematic of an apron taken from the Federal Lands Division of the Federal Highway Administration is shown in **Figure 608-3**.



Source: FHWA, July 2006, HEC-14, Figure 10.4, p. 10-16.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/06086/hec14.pdf>

**Figure 608-3 Placed Riprap at Culverts**

Aprons are constructed of riprap or grouted riprap at a zero grade for a distance that is often related to the outlet pipe diameter. These aprons dissipate energy through increased roughness for a short distance. However, they do serve to spread the flow helping to transition to the natural drainage way or to sheet flow where no natural drainage way exists. However, if they are too short, or otherwise ineffective, they simply move the location of potential erosion downstream. The key design elements of the riprap apron are the riprap size as well as the length, width, and depth of the apron.

Several relationships have been proposed for riprap sizing for culvert aprons, which are discussed in greater detail in Appendix D of HEC-14 (FHWA, July 2006). The independent variables in these relationships include one or more of the following variables: outlet velocity, rock specific gravity, pipe dimension (e.g. diameter), outlet Froude number, and tailwater. The following equation (Fletcher and Grace, 1972) is recommended for circular culverts:

$$D_{50} = 0.2 D \left( \frac{Q}{\sqrt{g} D^{2.5}} \right)^{4/3} \left( \frac{D}{TW} \right) \quad \text{608-1}$$

(FHWA, July 2006, HEC-14, Eq. 10.4, p. 10-17)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/06086/hec14.pdf>

where:

$D_{50}$	=	riprap size, ft
$Q$	=	design discharge, ft <sup>3</sup> /s
$D$	=	culvert diameter (circular), ft
$TW$	=	tailwater depth, ft
$g$	=	acceleration due to gravity, 32.2 ft/s <sup>2</sup>

Tailwater depth for **Equation 608-1** should be limited to between 0.4  $D$  and 1.0  $D$ . If tailwater is unknown, use 0.4  $D$ .

Whenever the flow is supercritical in the culvert, the culvert diameter is adjusted as follows:

$$D' = \frac{D + y_n}{2} \quad \text{608-2}$$

(FHWA, July 2006, HEC-14, Eq. 10.5, p. 10-17)

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/06086/hec14.pdf>

where:

$D'$	=	adjusted culvert rise, ft
$y_n$	=	normal (supercritical) depth in the culvert, ft

**Equation 608-1** assumes that the rock specific gravity is 2.65. If the actual specific gravity differs significantly from this value, the  $D_{50}$  should be adjusted inversely to specific gravity. The engineer should calculate  $D_{50}$  using **Equation 608-1** and compare with available riprap classes. A project or design standard can be developed such as the example from the FHWA shown in **Table 608-2** (first two columns). The class of riprap to be specified is that which has a  $D_{50}$  greater than or equal to the required size. For projects with several riprap aprons, it is often cost effective to use fewer riprap classes to simplify acquiring and installing the riprap at multiple

locations. In such a case, the designer must evaluate the tradeoffs between over sizing riprap at some locations in order to reduce the number of classes required on a project.

The apron dimensions must also be specified. **Table 608-2** provides guidance on the apron length and depth. Apron length is given as a function of the culvert rise (D) and the riprap size. Apron depth ranges from  $3.5D_{50}$  for the smallest riprap to a limit of  $2.0D_{50}$  for the larger riprap sizes. The final dimension, width, may be determined using the 1:3 flare shown in **Figure 608-3** and should conform to the dimensions of the downstream channel. A filter blanket should also be provided as described in HEC-11 (FHWA, March 1989).

FHWA, March 1989, "HEC-11, Design of Riprap Revetment".

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hec/hec11sl.pdf>

**Table 608-2 Example Riprap Classes and Apron Dimensions**

Source: FHWA, July 2006, HEC-14, Table 10.1, p. 10-18.

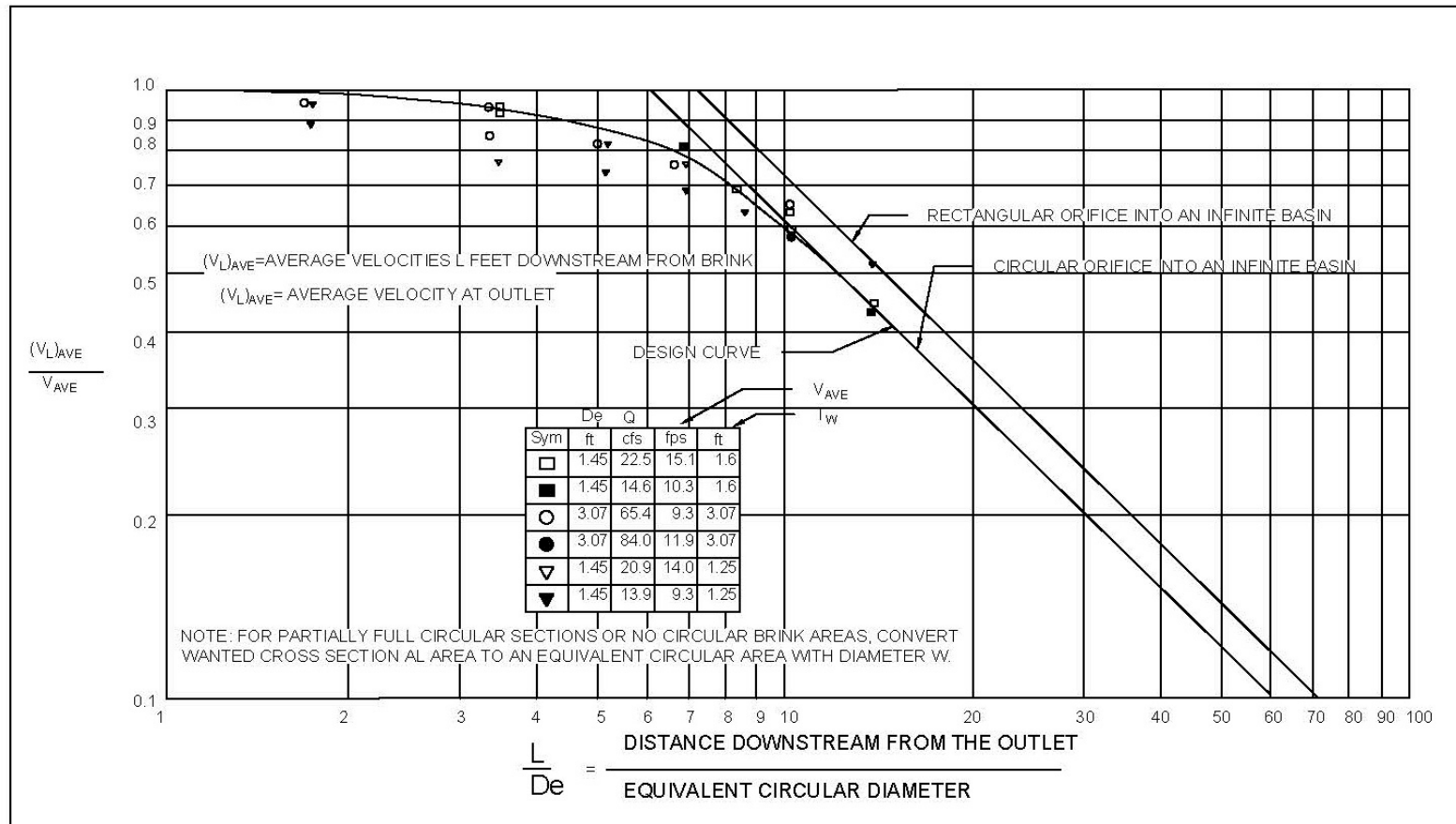
<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/06086/hec14.pdf>

Class	$D_{50}$ (mm)	$D_{50}$ (in)	Apron Length <sup>1</sup>	Apron Depth
1	125	5	4D	$3.5D_{50}$
2	150	6	4D	$3.3D_{50}$
3	250	10	5D	$2.4D_{50}$
4	350	14	6D	$2.2D_{50}$
5	500	20	7D	$2.0D_{50}$
6	550	22	8D	$2.0D_{50}$

<sup>1</sup>D is the culvert rise

For tailwater conditions above the acceptable range for **Equation 608-1** ( $TW > 1.0 D$ ), **Figure 608-4** should be used to determine the velocity downstream of the culvert. The guidance in Section 10.3 of HEC-14 (FHWA, July 2006) may be used for sizing the riprap. The apron length is determined based on the allowable velocity and the location at which it occurs based on **Figure 608-4**.

Over their service life, riprap aprons experience a wide variety of flow and tailwater conditions. In addition, the relations summarized in **Table 608-2** do not fully account for the many variables in culvert design. To ensure continued satisfactory operation, maintenance personnel should inspect them after major flood events. If repeated severe damage occurs, the riprap location may be a candidate for extending the apron or for another type of energy dissipator.



Source: FHWA, July 2006, HEC-14, Figure 10.3, p. 10-4.

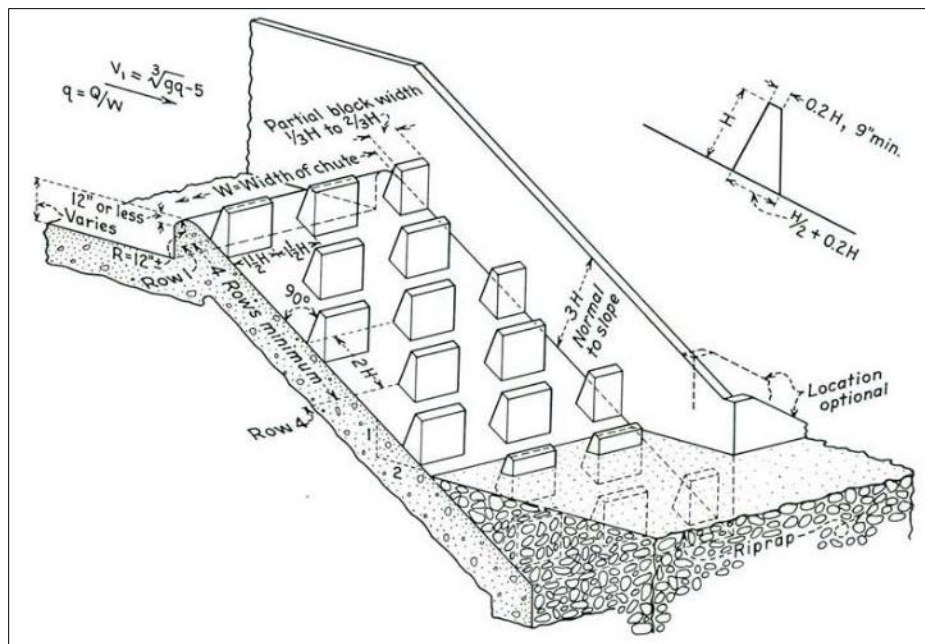
<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/06086/hec14.pdf>

**Figure 608-4 Distribution of Centerline Velocity for Flow from Submerged Outlets**



### 608.7.2 Basins for Culvert Outlet Protection

Basins to dissipate energy may be required for culverts greater than 60 inches in diameter or 60 inches in height. A basin has a depression and will require more material than an apron. A basin may be designed with concrete, soil cement, gabions, wire-enclosed riprap, or riprap. Large vertical drops are more likely to require baffle chute type spillways and the USBR (1978) has several typical designs available depending on various criteria and site conditions. **Figure 608-5** presents the USBR (1978) Type IX baffled apron and basin. Less expensive riprap basins may be more applicable in many situations.



Source: FHWA, July 2006, HEC-14, Figure 7.9, p. 7-33. Peterka, 1978  
<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/06086/hec14.pdf>

**Figure 608-5 USBR Type IX Baffled Apron**

#### Riprap Basins

For culverts greater than 60 inches in diameter or height, and when discharges and velocities are large, riprap basins may be required. The following procedure is from HEC-14 (FHWA, July 2006).

This section includes discussions of both riprap aprons and riprap basin energy dissipators. Both can be used at the outlet of a culvert or chute (channel) by themselves or at the exit of a stilling basin or other energy dissipator to protect against erosion downstream. Section 10.1 of HEC-14 (FHWA, July 2006) provides a design procedure for the riprap basin energy dissipator that is based on armoring a pre-formed scour hole. The riprap for this basin is a special gradation. Section 10.2 of HEC-14 (FHWA, July 2006) includes discussions of riprap aprons that provide a flat armored surface as the only dissipator or as additional protection at the exit of other dissipators. The riprap for these aprons is generally from the NMDOT standard classes.

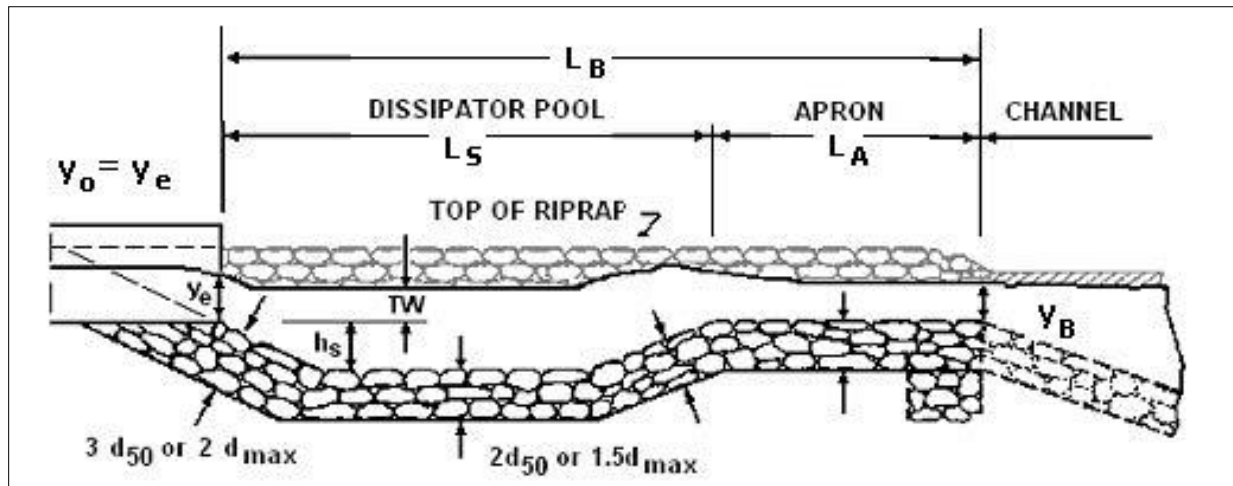
Section 10.3 of HEC-14 (FHWA, July 2006) provides additional discussions of riprap placement downstream of energy dissipators.

The design procedure for the riprap basin is based on research conducted at Colorado State University (Simons, et al., 1970; Stevens and Simons, 1971) that was sponsored by the Wyoming Highway Department. The recommended riprap basin that is shown in **Figure 608-6** and in **Figure 608-7** has the following features:

- The basin is pre-shaped and lined with riprap that is at least  $2D_{50}$  in depth.
- The riprap floor is constructed at the approximate depth of scour,  $h_s$ , that would occur in a thick pad of riprap. The  $h_s/D_{50}$  of the material should be greater than 2.
- The length of the energy dissipating pool,  $L_s$ , is  $10h_s$ , but no less than  $3W_o$ ; the length of the apron,  $L_A$ , is  $5h_s$ , but no less than  $W_o$ . The overall length of the basin (pool plus apron),  $L_B$ , is  $15h_s$ , but no less than  $4W_o$ .
- A riprap cutoff wall or sloping apron can be constructed if downstream channel degradation is anticipated as shown in **Figure 608-6**.

Further guidance and details on the design of outlet structures for scour remediation and energy dissipation can be found in the following references:

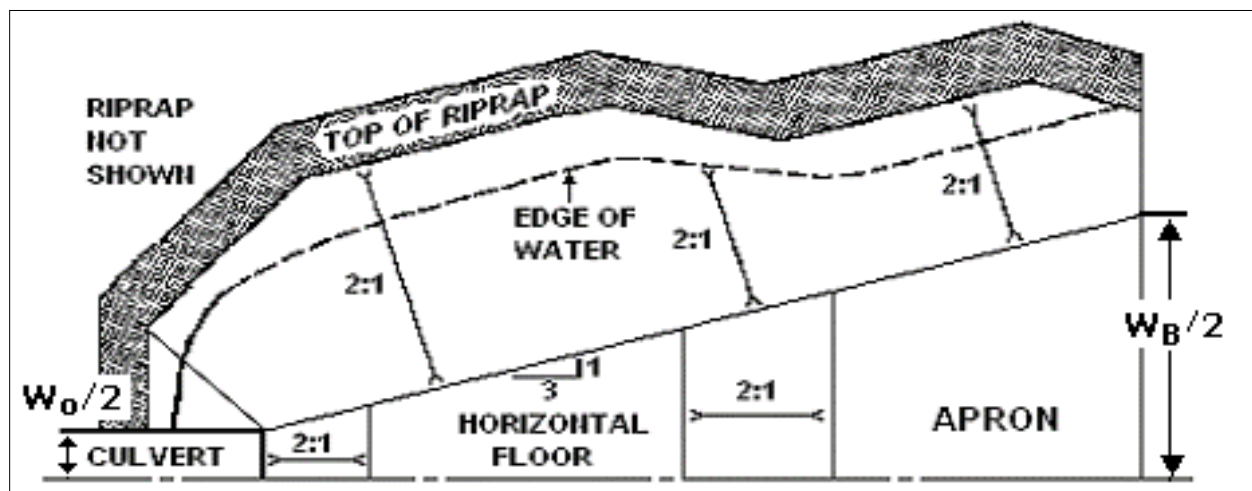
- HEC-14, "Hydraulic Design of Energy Dissipators for Culverts and Channels" (FHWA, July 2006)  
<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/06086/hec14.pdf>
- Hydraulic Design of Stilling Basins and Energy Dissipators (USBR, 1978)  
[https://www.usbr.gov/tsc/techreferences/hydraulics\\_lab/pubs/EM/EM25.pdf](https://www.usbr.gov/tsc/techreferences/hydraulics_lab/pubs/EM/EM25.pdf)
- NMDOT Standard Drawings for Outlet Erosion Protection (NMDOT Website)  
<http://dot.state.nm.us/content/nmdot/en/Standards.html>
- Denver Urban Storm Drainage Criteria Manual (June 2001)  
<http://udfcd.org/criteria-manual>



Source: FHWA, July 2006, HEC-14, Figure 10.1, p. 10-1.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/06086/hec14.pdf>

**Figure 608-6 Profile of Riprap Basin**



FHWA, July 2006, HEC-14, Figure 10.2, p. 10-2.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/06086/hec14.pdf>

**Figure 608-7 Half Plan of Riprap Basin**

## **608.8 General Discussion of Some Common Structures and Design Issues**

### **Check Dams**

A check dam (also called a grade control structure) is a dam or weir constructed across a channel to prevent degradation. The structure may be designed with various materials that typically include gabions, soil cement, concrete, and possibly other materials. A check dam provides two functions that are:

1. To stop degradation at the check dam location resulting from the downstream reach
2. To set or control the grade for the reach upstream from the dam crest

A check dam may eliminate the need for channel lining of concrete or other materials, by reducing the channel slope which reduces velocities and scour potential. The downstream reach slope must be evaluated carefully to determine the expected future long term bed slope reduction that may occur, or to determine if the downstream reach has reached a near equilibrium slope.

For guidance on how to determine check dam spacing (more than one dam) and vertical heights, and scour depth computations at the toe of the check dam, refer to the Sediment and Erosion Design Guide, (SSCAFCA, November 2008).

[http://sscafca.org/development/documents/sediment\\_design\\_guide/Sediment%20Design%20Guide%2012-30-08.pdf](http://sscafca.org/development/documents/sediment_design_guide/Sediment%20Design%20Guide%2012-30-08.pdf)

### **Wire Enclosed Riprap**

Wire enclosed riprap includes wire and beams driven in to the substrate. This may be used on channel bank slopes and as bed protection. The wire and beams may add some additional level of riprap stability by securing the wire to resist movement. Refer to the NMDOT Standard Drawings for details.

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

### **Gabions**

Gabions are wire baskets filled with stones. Gabions may be used as erosion protection and energy dissipation and may also provide structural reinforcement as soil retaining walls. The NMDOT has prepared standard gabion drawings that indicate the limits of retaining and gabion height allowed in retaining situations. Refer to the NMDOT Standard Drawings for details.

## **608.9 References**

Denver Urban Drainage and Flood Control District, June 2001, "Urban Storm Drainage Criteria Manual, Chapter 8".

<http://udfcd.org/criteria-manual>

FHWA, March 1989, "HEC-11, Design of Riprap Revetment".

<https://www.fhwa.dot.gov/engineering/hydraulics/pubs/hec/hec11sl.pdf>

FHWA, February 1990, "Highways in the River Environment".

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hire1990.pdf>

FHWA, July 2006, "HEC-14, Hydraulic Design of Energy Dissipators for Culverts and Channels, Third Edition".

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/06086/hec14.pdf>

FHWA, September 2009, "HEC-23, Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance, Third Edition, Volume 1".

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/09111/09111.pdf>

FHWA, September 2009, "HEC-23, Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance, Third Edition, Volume 2".

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/09111/09112.pdf>

Fletcher, B. P. and Grace, J. L., 1972, "Practical Guidance for Estimating and Controlling Erosion at Culvert Outlets" Misc. Paper H-72-5, U.S. Army Waterways Experiment Station, Vicksburg, Mississippi.

Lane, E.W., 1955, "The Importance of Fluvial Geomorphology in Hydraulic Engineering," ASCE Proceeding, Vol. 81, No. 745, pp. 1-17.

NMDOT, Website, "Standard Drawings", "Standard Specifications for Highway and Bridge Construction".

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

Simons, D. B., Stevens, M. A., and Watts, F. J., 1970, "Flood Protection at Culvert Outlets," Colorado State University Fort Collins, Colorado, CER 69-70 DBS-MAS-FJW4.

SSCAFCA, November 2008, Mussetter Engineering Inc., "Sediment and Erosion Design Guide".

[http://sscafca.org/development/documents/sediment\\_design\\_guide/Sediment%20Design%20Guide%2012-30-08.pdf](http://sscafca.org/development/documents/sediment_design_guide/Sediment%20Design%20Guide%2012-30-08.pdf)

Stevens, M. A. and Simons, D. B., 1971, "Experimental Programs and Basic Data for Studies of Scour in Riprap at Culvert Outfalls," Colorado State University, Fort Collins, Colorado, CER 70-71-MAS-DBS-57.

USBR, Peterka, January 1978, "Hydraulic Design of Stilling Basins and Energy Dissipators".

[https://www.usbr.gov/tsc/techreferences/hydraulics\\_lab/pubs/EM/EM25.pdf](https://www.usbr.gov/tsc/techreferences/hydraulics_lab/pubs/EM/EM25.pdf)

# 700 WATER QUALITY PROTECTIONS

One of the NMDOT's goals is to protect New Mexico's water resources from damages caused by the construction and operation of its transportation facilities. There are two distinct categories of issues addressed in this manual relating to water quality protection. First are the regulations and multiple permits associated with construction and operations relating to water quality protection. Second are water quality related design standards and the analyses that support them.

## 701 Regulation and Permitting

The realm of regulation and permitting can be further subdivided into general regulations and permits: those permits which apply to all new construction which disturb one acre or more and those permits which are tied to an MS4 Permit (Municipal Separate Stem Sewer System). The regulations and rules required by the MS4 Permits are more specific and have been tailored to address the known water quality issues within the specific permit area.

### 701.1 General Water Quality Protection Regulations and Permits

It may be generally said that water is the nexus for most of the environmental protection regulations. As a result, the Federal Water Pollution Control Act of 1972, and the Clean Water Act (CWA) of 1977 and the Water Quality Act of 1987 (which amended the 1972 act) gave broad powers to the USEPA in the regulation of the nation's rivers, streams and lakes. The Rivers and Harbors Act of 1899 gave jurisdiction to the U.S. Army Corps of Engineers (USACE) over the nation's navigable rivers ("Waters"). Subsequently, Section 404 of the Clean Water Act designated the USACE as having jurisdiction over the cutting and filling within navigable waters of the U.S. The USEPA has delegated some of its authority to the State of New Mexico Environment Department (Section 401 and 402) to assess and regulate the health of the state's streams and lakes. Lastly, New Mexico has established design guidelines for the protection of its fish and wildlife from impacts caused by construction. A summary of the general regulations and links to the appropriate websites for further information are provided below.

#### 701.1.1 USEPA Regulations Summary Relating to Stormwater and Roads

The following is a summary of the current regulations relating to stormwater and roads in New Mexico:

- a. USEPA Regulations Summary Relating to Stormwater and Roads
  - i. National Pollutant Discharge Elimination System (NPDES) Manual Revision 2, August 2012.
  - ii. <http://dot.state.nm.us/content/dam/nmdot/Infrastructure/NPDESM.pdf>
  - iii. As part of the Water Quality Act of 1987, stormwater discharge associated with industrial activity from a point source to waters of the United States is unlawful, unless authorized by an NPDES Permit. Construction activities that disturb an

- area greater than one acre by grading, clearing, grubbing, or other construction activity are subject to the requirement of an NPDES Permit.
- iv. In order to effectively manage the permit process, the EPA has produced a General Permit for Discharges from Construction Activities (CGP), which defines specific conditions and requirements to be met as part of the General Permit. The General Permit establishes the procedures required for proper coverage, the requirement for a Stormwater Pollution Prevention Plan (SWPPP), and requirements for termination of permit coverage.
  - v. The NPDES Stormwater Permitting Program in New Mexico is administered by the EPA. Requirements for the NPDES Stormwater Discharge Permit are defined by federal law in Section 402(p) of the CWA, and added by Section 405 of the Water Quality Act of 1987.
  - vi. Compliance with the requirements of the Construction General Permit consists of four major components that must be accomplished:
    - Determination of eligibility
    - Preparation and implementation of a SWPPP
    - Submission of a Notice of Intent (NOI)
    - Submission of a Notice of Termination (NOT)

**Note:** The SWPPP is usually prepared in conjunction with the construction design documents for the site, and before the submission of the NOI to the EPA, depending on local authority requirements.

- vii. In addition to NPDES Permits for construction activities, large, medium, and some small sized municipalities (as identified by the EPA) are required to obtain NPDES Permits for their MS4s to control stormwater outflow into waters of the United States.
- viii. NPDES General Permits are termed umbrella permits, and will consolidate permit compliance requirements for many common sources of pollutants, activities, and sites under one permit. The coverage of these umbrella permits is broad, with general compliance requirements, and is effective for five years.
- ix. The permit for stormwater discharges from industrial sites, the NPDES Multi-Sector General Permit for Stormwater Discharges Associated with Industrial Activity (MSGP), requires the development of a Stormwater Pollution Prevention Plan (SWPPP), which is the documentation of the measures that will be implemented to ensure that pollution does not occur. There are requirements in the Multi-Sector General Permit (MSGP) for industry-specific Best Management Practices (BMPs), and for monitoring and analytical activities, based on Standard Industrial Classification (SIC)-code determinations for the particular industrial activity. The analytical requirements ensure that industrial activity-specific pollutants aren't being transported in stormwater runoff.



### **701.1.2 U.S. Army Corps of Engineers – Albuquerque District**

The U.S. Army Corps of Engineers Permitting overview page is presented here:

<http://www.spa.usace.army.mil/Missions/RegulatoryProgramandPermits/PermittingOverview.aspx>

#### **Section 10 of the Rivers and Harbors Act of 1899**

Section 10 of the Rivers and Harbors Act of 1899 requires approval from the U.S. Army Corps of Engineers (USACE) prior to the accomplishment of any work in or over navigable waters of the United States, or which affects the course, location, condition or capacity of such waters.

Typical activities requiring Section 10 permits are:

- Construction of piers, docks, wharves, bulkheads, dolphins, marinas, ramps, floats, intake/outfall structures, and cable or pipeline crossings
- Dredging and excavation

Section 10 Waters within the Albuquerque District include the following:

- Navajo Lake (San Juan County, New Mexico)
- Rio Grande at the international border with Mexico (El Paso County, Texas)

<http://www.usace.army.mil/Portals/2/docs/civilworks/regulatory/materials/rhsec10.pdf>

#### **Section 404 of the Clean Water Act**

[http://www.usace.army.mil/Portals/2/docs/civilworks/regulatory/materials/cwa\\_sec404doc.pdf](http://www.usace.army.mil/Portals/2/docs/civilworks/regulatory/materials/cwa_sec404doc.pdf)

Section 404 of the Clean Water Act requires approval from the USACE prior to discharging dredged or fill material into waters of the United States (U.S.). Typical activities requiring Section 404 Permits include, but are not limited to the following:

- Construction of flood control and stormwater management facilities, mining, grading, intake/outfall structures, road crossings, pipelines
- Construction of boat ramps, docks, piers, shoreline or bank stabilization, and fish habitat
- Construction of revetments, groins, breakwaters, levees, dams, dikes, drop-structures and weirs
- Placement of riprap, culverts, and footings

In some cases, dredging and other excavation require approval when there is a discharge that results in more than incidental fallback ([33 CFR 323.2\(d\)\(1\)](#)) of dredged or excavated material.

#### **Regulatory Overview**

Activities requiring a permit from the USACE under Section 404 of the Clean Water Act or Section 10 of the Rivers and Harbors Act of 1899 may be permitted by General Permit or Individual Permit. General Permits are issued nationally, regionally or programmatically to authorize categories of activities that would result in minimal individual and cumulative impacts to the aquatic environment. These Permits are valid only if the terms and conditions applicable to the Permits are met. If the terms and conditions of a General Permit cannot be met, or if the USACE determines that the activity would result in more than minimal impacts, an Individual Permit would be required.



There are three types of General Permits:

#### Nationwide Permits

<http://www.spa.usace.army.mil/Missions/RegulatoryProgramandPermits/NWP.aspx>

There are fifty-four Nationwide 404 Permits with thirty-one conditions in the 2012 list of permits, ranging from “Aids to Navigation” to “Cranberry Production Activities” to “Water-Based Renewable Energy Generation Pilot Plants”. Many of the activities of the NMDOT in and around New Mexico arroyos, streams and rivers may be covered by a Nationwide 404 Permit. It is always prudent to consult with the Environmental Bureau regarding their experiences first and then with the U.S. Army Corps of Engineers, Albuquerque District, Regulatory Branch before beginning work that may affect a watercourse.

#### Regional General Permits

<http://www.spa.usace.army.mil/Missions/RegulatoryProgramandPermits/RGP.aspx>

**Regional General and Programmatic General Permits.** The Albuquerque District currently has five General Permits:

- RGP 16-01 for Utility Line Maintenance, Repair or Removal
- RGP 14-02 for NM-West TX Sediment and Debris Removal Activities
- RGP 14-01 for Stream Stabilization and Water Quality Improvement Projects Within Urban Ephemeral Channels in the State of New Mexico
- NM 12-01 Bureau of Reclamation Channel Maintenance Activities
- NM 12-02 Elephant Butte Reach of the Rio Grande

USACE does not currently have any **Programmatic General Permits** for New Mexico.

The basic form of authorization for jurisdictional activities not covered by a Nationwide Permit or one of the five Regional General Permits is the Standard Individual Permit.

#### Standard Individual Permits

Individual Permits involve evaluation of individual, project-specific applications. Individual permits are required for authorization of projects that result in more than minimal impacts to the aquatic environment. There are two types of permits: Standard Individual Permits and Letters of Permission. Standard Individual Permits require a public notice and agency coordination. Permitting elements may include coordination/consultation, mitigation and appeals.

#### **Exempted Activities**

There are a few activities which involve the placement of fill in a waterway that are not subject to the Section 404 regulatory program. However, the fill must not change the use of the water and the flow must not be impaired. These exempted activities are briefly described below, and located in the regulations at 33 CFR 323.4.

1. Normal farming, silviculture and ranching activities such as plowing, seeding, cultivating, minor drainage and harvesting.

2. Maintenance or emergency repair of a currently serviceable structure such as dams, riprap, abutments, and levees. The original design may not be changed.
3. Maintenance or construction of stock ponds or irrigation ditches. Maintenance (not construction) of drainage ditches. Discharges associated with irrigation facilities are included.
4. Construction of temporary sedimentation basins at construction sites if fill material is not placed in waters of the United States.
5. Activities for which a state has an approved program under Section 208 of the Clean Water Act for non-point pollution sources.
6. Construction or maintenance of farm roads, forest roads, or temporary mining roads. Best management practices must be followed to reduce flow pattern impairment and aquatic impacts (see regulations for more information).

### **Who Should Obtain a Permit?**

Any person, firm, or agency (including Federal, state, tribal and local governmental agencies) planning to work in waters of the United States should first contact the USACE regarding the need to obtain a permit from the Regulatory Division. Permits, licenses, variances, or similar authorization may also be required by other Federal, state and local statutes. The necessary permits are required even when land next to or under the water is privately owned.

Both the property owner and contractor may be held liable for violation of Federal law if work begins before permits have been obtained. Penalties for proceeding with work without a permit issued by the USACE may include:

- Removal of work and restoration of area
- Administrative penalties of up to \$25,000 per day for each violation
- A fine of up to \$50,000 per day for each violation
- Up to three years in prison

### **How to Obtain a Permit**

#### Section 401 Water Quality Certification (WQC)

<https://www.epa.gov/cwa-404/clean-water-act-section-401-certification>

A Section 401 Water Quality Certification (WQC) is required for all Section 404 General and Individual Permits. WQC is administered by state agencies, tribes or the Environmental Protection Agency (EPA). There are different WQC requirements depending on the state or tribal land where the project is located.

Nationwide Permits: Follow General Condition 27 Pre-Construction Notification (PCN) procedures to apply for authorization under a Nationwide Permit (NWP). Also, follow the Albuquerque District Nationwide Permit Checklist.

Regional General Permits: Follow the PCN procedures outlined in the particular RGP. WQC has been issued by the State of Texas for all current RGPs available in that state. Colorado is certified by Code of Colorado Regulation No. 82 for all current RGPs available in that state. For New Mexico, the terms and conditions of WQC are dependent on the specific RGP, which are included in the Permit itself.

Standard Individual Permits: Complete an application form following the instructions attached to the form. Also, follow the Albuquerque District Checklist of Complete Application Information. When applying for authorization under a Standard Individual Permit, the applicant must obtain Individual WQC from the appropriate state agency or tribe with regulatory authority, or EPA in the project area. An application must also include a statement describing how impacts to waters of the U.S. will be avoided and minimized. The application must include either a statement describing how impacts to waters of the U.S. will be compensated for or a statement explaining why compensatory mitigation should not be required for the proposed impacts (33 CFR 325.1).

Letters of Permission: The Albuquerque District has two Letters of Permission available, which only authorize projects located in the State of Texas. Follow the application procedures outlined in the particular Letter of Permission. WQC has been issued by the State of Texas for both Letters of Permission.

Project proponents are encouraged to contact the USACE as early as possible to determine notification requirements. If a permit is required, a pre-application consultation may be beneficial. Pre-application consultations are most beneficial when a Standard Permit is required and typically involve reviewing preliminary plans, discussing alternatives, discussing potential issues that may occur during the permitting process, providing guidance, and discussing mitigation. Consultations may occur via phone call, e-mail, or an on-site or off-site meeting. It is not always necessary to conduct consultation meetings.

Mitigation: For Individual Permits, an application must include a statement describing how impacts to waters of the U.S. will be avoided and minimized. The application must include either a statement describing how impacts to waters of the U.S. will be compensated for or a statement explaining why compensatory mitigation should not be required for the proposed impacts. The permittee must prepare a draft mitigation plan and submit it for review. A final mitigation plan must be submitted to and approved by the USACE before an Individual Permit can be issued. For General Permits, if compensatory mitigation is required, the USACE may approve a conceptual or detailed mitigation plan. However, a final mitigation plan must be approved before the permittee commences work in waters of the U.S.

### **Emergency Authorizations**

The Division Engineer of the South Pacific Division issued permitting procedures for emergency situations that require immediate authorization within the Albuquerque, Los Angeles, Sacramento, and San Francisco Districts. An emergency is defined as a situation that would result in an unacceptable hazard to life, a significant loss of property, or an immediate, unforeseen, and significant economic hardship if corrective action requiring a permit is not undertaken within a time period less than the normal time needed to process the application under standard procedures (33 CFR 325.2). In the case of an emergency where authorization from the USACE is required, contact the local district office in your area as soon as possible.

### **Permit Fees**

General Permits and Letters of Permission do not require fees. Fees for Standard Individual Permits are assessed according to the proposed use. For example, the fee for work to be done for commercial and industrial use is \$100, for private or non-commercial use, the fee is \$10. The applicant will be notified of the required fee. No fee is required for Federal, state, or local

government agencies. Fees are due when the permit decision is initially proffered to the applicant. Permit fees are subject to future changes.

### 701.1.3 New Mexico Environment Department

A list of impaired waters ("303d listed" waters) for New Mexico has been developed by the New Mexico Environment Department. The New Mexico Water Quality Control Commission approves the list. This list is submitted to USEPA as required by the Water Pollution Control Act (known as the Clean Water Act). The intended audience for this list includes but is not limited to the general public, stakeholders, interest groups, consultants, state legislators, governmental agencies at all levels, as well as universities and other educational entities.

1. The most current listing of New Mexico Clean Water Act impaired waters may be found at: <https://www.env.nm.gov/swqb/303d-305b/2016-2018/index.html>
2. CWA § 303(d)(2) requires that each state submit to the United States Environmental Protection Agency ([US EPA](#)) a listing of water quality limited segments requiring waste load allocations, load allocations and Total Maximum Daily Loads (TMDLs). CWA § 305(b) (1) requires that each state submit a biennial report to the United States Congress through the USEPA. In accordance with the above stated statutory requirements, this report contains:
  - a. An assessment of water quality;
  - b. An analysis of the extent to which the CWA §101(a)(2) goal of surface water quality which provides for protection and propagation of fish, shellfish, and wildlife and recreation in and on the water is being achieved;
  - c. an overview of progress in water pollution control and recommendations for further action; and
  - d. a description of the nature of nonpoint source pollution and of programs for nonpoint source pollution control.

Appendix A of the current report is the active *State of New Mexico CWA §303(d)/§305(b) Integrated List of Assessed Waters*. Appendix may be found at:

[https://www.env.nm.gov/swqb/303d-305b/2016-2018/documents/FINALDRAFT303d\\_305b\\_List\\_04\\_25\\_16.pdf](https://www.env.nm.gov/swqb/303d-305b/2016-2018/documents/FINALDRAFT303d_305b_List_04_25_16.pdf)

The Integrated List identifies whether or not a particular surface water of the state is currently meeting its designated uses as detailed in the [Standards for Interstate and Intrastate Surface Waters](#) (20.6.4 NMAC) through application of the State's [Assessment Protocol](#). "Category 5" waters on the Integrated List specifically constitute the CWA Section §303(d) List of Impaired Waters.

The water quality in impaired waters is evaluated in terms of the stream or water bodies hoped for use (drinking, swimming, fishing, irrigation, etc.) and lists the nature of the impairment in terms of total maximum daily load of pollutants that can be discharged to it without further impairing the water for the intended use(s).

#### **701.1.4 New Mexico Department of Game and Fish**

The New Mexico Department of Game and Fish Department (NMDGF) has policies to protect and preserve critical habitat. These policies include fragmentation of habitat, and animal travel and migration paths. Drainage projects have the potential to impact habitat as the intent of drainage projects is to manage water. Consultation with NMDGF during the planning phase of a project will inform the designers of issues that would be otherwise unknown and potentially overlooked.

#### **701.2 MS4s (Municipal Separate Storm Sewer Systems)**

A MS4 is a conveyance or system of conveyances that is:

- Owned by a state, city, town, village, or other public entity that discharges to waters of the U.S.
- Designed or used to collect or convey stormwater (e.g., storm drains, pipes, ditches)
- Not a combined sewer
- Not part of a sewage treatment plant, or publicly owned treatment works (POTW)

To prevent harmful pollutants from being washed or dumped into specific types of MS4s, operators may be required to obtain NPDES Permits and develop stormwater management programs (SWMPs).

##### **Phase I MS4s**

Issued in 1990, Phase I regulations require medium and large cities, or certain counties with populations of 100,000 or more, to obtain NPDES Permit coverage for their stormwater discharges.

- There are approximately 750 Phase I MS4s nationwide
- Generally, Phase I MS4s are covered by Individual Permits
- The Albuquerque metropolitan area is the only Phase 1 MS4 in New Mexico

##### **Phase II MS4s**

Issued in 1999, Phase II regulations require regulated small MS4s in urbanized areas, as well as small MS4s outside the urbanized areas that are designated by the permitting authority, to obtain NPDES Permit coverage for their stormwater discharges.

- There are approximately 6,700 Phase II MS4s nationwide
- Generally, Phase II MS4s are covered by General Permits
- There are five areas covered by the forthcoming New Mexico Small MS4 Permit (anticipated in 2018): Farmington/Aztec, Santa Fe, Los Lunas, Las Cruces, and the El Paso suburbs of Anthony and Sunland Park

NPDES Permits for regulated MS4s require permittees to develop a Stormwater Management Plan (SWMP), which describes the stormwater control practices that will be implemented consistent with permit requirements to minimize the discharge of pollutants from the stormwater sewer system.

On December 11, 2014, EPA Region 6 Water Quality Protection Division announced the issuance of the National Pollutant Discharge Elimination System (NPDES) General Permit for stormwater discharges from municipal separate storm sewer systems (MS4s) located in the Middle Rio Grande Watershed in the State of New Mexico (Watershed Based Permit). The permit offers discharge authorization and requires an MS4 Permit for regulated MS4s within the boundaries of the Bureau of the Census designated 2000 and 2010 Albuquerque Urbanized Areas and any other MS4s in the watershed designated by the Director. This Permit is intended to replace both the Individual NPDES Permit NMS000101 issued on January 31, 2012, and the expired General Permits NMR040000 and NMR040001 for dischargers in this watershed area.

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

The co-permittees on this permit are:

- The City of Albuquerque (COA)
- Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA)
- The University of New Mexico
- New Mexico Department of Transportation (District 3)
- Bernalillo County
- Sandoval County
- Village of Corrales
- City of Rio Rancho
- Los Ranchos de Albuquerque
- Kirtland Air Force Base
- Town of Bernalillo
- EXPO New Mexico
- Southern Sandoval County Arroyo Flood Control Authority (SSCAFCA)
- Eastern Sandoval County Arroyo Flood Control Authority (ESCAFCA)
- Sandia Laboratories, Department of Energy
- Pueblo of Sandia
- Pueblo of Isleta
- Pueblo of Santa Ana

The Permit is lengthy and complex, but may be summarized to the extent that it establishes the following minimum pollution control measures for the area covered by the Permit and for the entities covered under the Permit:

### **Minimum Pollution Control Measures**

The Phase II regulations at 40 CFR 122.34 set forth the following six minimum pollution control measures to be included in SWMPs.

1. Public Education and Outreach on Stormwater Impacts
2. Public Involvement/Participation
3. Illicit Discharge Detection and Elimination
4. Construction Site Stormwater Runoff Control
5. Post-Construction Stormwater Management in New Development and Redevelopment
6. Pollution Prevention/Good Housekeeping for Municipal Operations

This manual principally addresses pollution control measure number 5 listed above as it relates to new highway construction and roadway improvement projects. If the project disturbs one or more acre, or is a part of a larger common plan of development, the permit requires that:

“Incorporate a stormwater quality design standard that manages on-site the 90<sup>th</sup> percentile storm event discharge volume associated with new development sites and 80<sup>th</sup> percentile storm event discharge volume associated with redevelopment sites, through stormwater controls that infiltrate, evapotranspire the discharge volume, except in instances where full compliance cannot be achieved, as provided in Part I.D.5.b.(v)”.

And:

“The permittee must ensure the appropriate implementation of the structural BMPs by considering some or all of the following: pre-construction review of BMP designs; inspections during construction to verify BMPs are built as designed; post-construction inspection and maintenance of BMPs; and penalty provisions for the noncompliance with preconstruction BMP design; failure to construct BMPs in accordance with the agreed upon pre-construction design; and ineffective post-construction operation and maintenance of BMPs.”

The Permit contains many interrelated details regarding specific compliance that is beyond the scope of this manual to list or discuss in depth. Prior to beginning design within the Permit area or within the boundaries of any of the entities covered by the Permit, the engineer should consult with the Drainage Design Bureau regarding specific design requirements.

The requirement to manage the 80<sup>th</sup>/90<sup>th</sup> percentile storm on site will normally be met by the inclusion of a retention pond. The pond must drain via infiltration in less than 96 hours to meet the New Mexico Office of the State Engineer water rights requirements.

For new development, the pond volume required may be computed as the 90<sup>th</sup> % rainfall depth multiplied by the new impervious area. For re-development, the pond volume required may be computed as the 80<sup>th</sup> % rainfall depth multiplied by the increase in impervious area.

A simple example, of a redevelopment project in Santa Fe, follows:

## MS4 Permit Requirements Pre-development hydrology

- **Volume retained = (post-construction impervious area  
MINUS pre-construction impervious area)  
TIMES rainfall depth**

Post-Construction Impervious Area	25,000 sq ft
Pre-Construction Impervious Area	20,000 sq ft
Net Increase in Impervious Area	5,000 sq ft

- **Required Retention Volume  
= (increase in IA) \* (80th% rainfall depth) \* (unit conversion)  
= 5,000 sq ft \* 0.50 inches \* (1 ft/12 in)  
= 208 cubic feet**

The following **Table 701-1** (Table 2 in the report) has been copied from the report USEPA, Office of Wastewater Management, Water Permits Division, Municipal Branch—“*Estimating Predevelopment Hydrology for Urbanized Areas in New Mexico*” March 2015. It describes the results of an investigation conducted for USEPA and under their direction to determine the precipitation values for the various annual exceedance percentages listed.

**Table 701-1 80<sup>th</sup>, 90<sup>th</sup>, and 95<sup>th</sup> Percentile Rainfall Events (inches)**

NCDC ID	LOCATION NAME	80 <sup>th</sup> Percentile	90 <sup>th</sup> Percentile	95 <sup>th</sup> Percentile
290234	Albuquerque International Airport	0.48	0.65*	0.94
293142	Farmington Agricultural Science Center	0.40	0.53	0.70
295084	Los Alamos	0.53	0.69	0.93
295150	Los Lunas 3 SSW	0.48	0.71	0.92
298085	Santa Fe 2	0.50	0.68	0.87
298535	State University (Las Cruces)	0.55	0.78	0.95
412797	El Paso Airport	0.54	0.82	1.08

\*Use 0.615 inches per the following paragraph.

Notes related to **Table 701-1** and information for the Albuquerque area follow.

The previous predevelopment runoff study (Kosco et al., 2014) used data from the Albuquerque International Airport for the period 1950-2012. Because rainfall data for the other stations studied in the 2015 report did not extend back to 1950, the 2015 report used the most recent 30-year period of record (1983-2013) for all stations which resulted in a slightly higher 90<sup>th</sup> percentile event for Albuquerque. For all NMDOT projects within the Small MS4 Permit areas, use the values in **Table 701-1**.

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

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

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



### 701.3 NMDOT's Stormwater Management Plan (SWMP)

The State of New Mexico has EPA-approved water quality standards. The goal of these permits is for implementation of the approved Stormwater Management Plan (SWMP) and other permit conditions to provide a reasonable assurance that the permitted activity will be conducted in a manner which will not violate applicable Water Quality Management Plan and Water Quality Standards, including but not limited to the following:

- *No discharge of toxic pollutants in toxic amounts:* It is the National Policy that the discharge of toxic pollutants in toxic amounts be prohibited (Section 101(a)(3) of the Act). The State of New Mexico Standards for Interstate and Intrastate Surface Waters (20.6.4.13 F.) state that "Surface waters of the State shall be free of toxic pollutants from other than natural causes in amounts, concentrations or combinations that affect the propagation of fish or that are toxic to humans, livestock or other animals, fish or other aquatic organisms, wildlife using aquatic environments for habitation or aquatic organisms for food, or that will or can be reasonably expected to bio-accumulate in tissues of fish, shellfish, and other aquatic organisms to levels that will impair the health of aquatic organisms or wildlife or result in unacceptable tastes, odor or health risks to human consumers of aquatic organisms".
- *No discharge of pollutants in quantities that would cause a violation of State or Tribal water quality standards:* Section 301(b)(1)(C) of the Act and 40 CFR 122.44(d) require that NPDES Permits include "...any more stringent limitations, including those necessary to meet water quality standards, treatment standards, or schedule of compliance, established pursuant to State law or regulations. EPA is using CWA 402(p)(3)(B)(iii) authority for "other such provisions as the Administrator deems appropriate for the control of pollutants" to include permit requirements supporting protection of water quality standards and compliance with TMDLs.
- *No discharge of floatable debris, oils, scum, foam, or grease in other than trace amounts.* The State of New Mexico Standards for Interstate and Intrastate Surface Waters (20.6.4.13 B) states that "Surface waters of the State shall be free of oils, scum, grease and other floating materials resulting from other than natural causes that would cause the formation of a visible sheen or visible deposits on the bottom or shoreline, or would damage or impair the normal growth, function or reproduction of human, animal, plant or aquatic life".
- *No discharge of non-stormwater from the municipal separate storm sewer system, except in accordance with Part I.A.4:* Permits issued to MS4s are specifically required by Section 402(p)(3)(B) of the Act to "include a requirement to effectively prohibit non-stormwater discharges into the storm sewers."

40 CFR 122.26(d)(2)(iv)(B)(1) and 122.34(b)(3)(iii) allows the permittee to accept certain non-stormwater discharges where they have not been identified as significant sources of pollutants. The definition of "illicit discharge" at 40 CFR 122.26(b)(2) excludes discharges subject to its own NPDES permit, so such permitted non-stormwater discharges would not be subject to the prohibition on non-stormwater.

- *No degradation or loss of State or Tribal-designated uses of receiving waters as a result of stormwater discharges from the municipal separate storm sewer (unless authorized in accordance with the State or Tribal Antidegradation Policy):* The State of New Mexico

and the Navajo Nation have adopted Antidegradation Policies and Implementation Plans as part of their Water Quality Standards which provide for maintenance of existing in-stream water uses; existing water quality levels where existing water quality exceeds the levels necessary to support propagation of fish, shellfish, and wildlife, and recreation in and on the water (except where the State or Tribe has determined that lowering water quality is necessary to accommodate important economic or social development in the area where the waters are located); existing water quality where high-quality waters constitute an outstanding national or tribal resource (e.g., waters of National and State parks and wildlife refuges or exceptional recreational or ecological significance); and compliance with Section 316 of the Act where potential water quality impairment is associated with a thermal discharge.

EPA Region 6 is unaware of any direct discharges to waters under the jurisdiction of a Tribe (Tribal waters), but Tribal waters may be located downstream of direct discharge points. A few examples are:

- Runoff from the Farmington urbanized area discharges to the San Juan River
- Downstream of the Farmington urbanized area, the San Juan River flows through the Navajo Nation
- Some arroyos and drains may flow downstream of the Santa Fe urbanized area to the Pueblo of Cochiti's waters

Permit conditions are expected to be protective of downstream Tribal waters.

## 702 Analysis in Support of Design

The goals of design for water quality protection are different than those that drive the design of bridges, crossing structures, and storm drainage systems. Ideally, the analysis methods used for both crossing structure protection and water quality protection would be consistent. However, USEPA has directed the permittees covered by the MS4 Permit for the Albuquerque metropolitan area (now Middle Rio Grande Watershed Based Permit) to develop a set of hydrologic parameters for the design of water quality protection, and specifically Stormwater Pollution Prevention Plans (SWPPPs) that are different than those found in both the 1995 Drainage Design Manual as well as in this manual update. These methods apply to the design of both construction and post-construction BMPs.

The August 2012 edition (Rev. ed. 2) of the Stormwater Management Guidelines for Construction and Industrial Activities should be used as guidance for construction projects and industrial sites.

<http://dot.state.nm.us/content/dam/nmdot/Infrastructure/NPDESM.pdf>

The project engineer should attempt to meet all criteria presented in those guidelines. However, those guidelines should not be considered a standard that must be met regardless of impacts. Engineers must exercise good judgment on individual projects and frequently must be innovative in their approach to stormwater management. The guidelines were designed to be used in all parts of the state of New Mexico, both in urban and rural areas.

This set of guidelines contains extensive instructions regarding the permitting process, design, operations and maintenance of stormwater BMPs, covering both construction and post-construction phases of a project.

USEPA has prepared two additional technical guidance documents for use in developing the hydrology for design and analyses of stormwater BMPs in New Mexico.

USEPA “Estimating Predevelopment Hydrology for Urbanized Areas in New Mexico” EPA Publication Number 832-R-15-009 March 2015.

USEPA “Estimating Predevelopment Hydrology in the Middle Rio Grande Watershed, New Mexico” EPA Publication Number 832-R-14-007 April 2014.

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

## **702.1 Green Infrastructure/Low Impact Development**

Most engineers designing highway drainage structures are experienced in designing for storms with 2% probability (50-yr storm) and 1% probability (100-yr storm), but have little experience analyzing the runoff from or designing for small to very small storms. **Green Infrastructure/Low Impact Development (GI/LID)** treats the runoff from the 80-95<sup>th</sup> percentile rainfall event, which is the event whose precipitation total is greater than or equal to 80-95% of all storms on an annual basis. In New Mexico, this is generally less than one inch of precipitation. The stated objective of GI/LID is to design drainage facilities such that the post-developed project hydrology mimics the pre-development hydrology to the **Maximum Extent Practicable (MEP)**.

The Middle Rio Grande Watershed Based MS4 Permit requires that GI/LID principles be applied whenever they are practicable. It is anticipated that the Small MS4 (sMS4) Permits will include the same requirements. The principal differences in designing using GI/LID principles compared to providing for adequate capacity for a bridge, crossing structure or storm drainage system for flood control is the magnitude and frequency of the rainfall events that are addressed.

The Arizona Water Resources Research Center has developed the following brief description of Green Infrastructure/Low Impact Development (GI/LID) and the differences and the similarities between the two terms.

<http://wrrc.arizona.edu/sites/wrrc.arizona.edu/files/Quick%20Resouce-%20GI%20and%20LID%20%28final%29.pdf>

Green Infrastructure (GI) and Low Impact Development (LID) are two terms that often arise during discussions about water harvesting, especially with the EPA’s focus on using these techniques to help combat non-point source pollution. Non-point source pollution is water pollution that comes from diffuse sources, such as stormwater runoff in urban areas. The agency defines green infrastructure as a stormwater management technique that “uses vegetation and soil to manage rainwater where it falls. By weaving natural processes into the built environment, green infrastructure provides not only stormwater management, but also flood mitigation, air quality management, and much more. LID is defined as an approach to land development (or re-development) that works with nature to manage stormwater as close to its

source as possible. LID employs principles such as preserving and recreating natural landscape features, minimizing effective imperviousness to create functional and appealing site drainage that treat stormwater as a resource rather than a waste product. As these two definitions show, GI and LID have a lot in common, and in fact, the terms are often used interchangeably. In some ways, GI and LID can be thought of as large-scale passive water harvesting.

Several types of stormwater management applications are used in GI/LID. These are listed in **Table 702-1** below and include:

**Table 702-1 GI/LID Potential Applications**

Source: USEPA GI/LID Website

<https://www.epa.gov/green-infrastructure>

GI/LID Practice	Definition <sup>i</sup>
<b>Bioretention</b>	Soil and plant-based filtration devices that remove pollutants through a variety of physical, biological, and chemical treatment processes
<b>Green roofs</b>	Vegetated roof covers that help to mitigate the effects of urbanization on water quality by filtering, absorbing or detaining rainfall
<b>Permeable pavement</b>	Alternative paving material to locally infiltrate rainwater and reduce the runoff leaving a site. Can be composed of pavers, cement, or asphalt
<b>Rain barrels and cisterns</b>	Large and small tanks used to store rainwater collected from a catchment area such as a roof
<b>Soil amendments</b>	Used to minimize development impacts on native soils by restoring their infiltration capacity and chemical characteristics. After soils have been amended their improved physical, biological and hydrological characteristics will make them more effective agents of stormwater management
<b>Tree box filters</b>	Mini bioretention areas installed beneath trees that can be very effective at controlling runoff, especially when distributed throughout the site

The EPA and others' websites have several useful resources on GI and LID, including studies showing that these techniques can be more cost-effective than traditional, "gray infrastructure". Gray infrastructure is the term used for traditional forms of stormwater management such as pipes and storm drains. EPA resources are listed here:

USEPA Green Infrastructure

<https://www.epa.gov/green-infrastructure>

## 703 Practical Application Principles

The basic goals and principles in designing water quality protection best management practices (BMPs) are to minimize the hydrologic and water quality impacts from development activities. These goals are often articulated as “mimicking pre-development hydrology”. Land disturbance removes the protective vegetative cover and exposes the soil to the erosive effects of rainfall and runoff. Paving and buildings significantly reduce the amount of precipitation that would have been intercepted by natural landscape characteristics and thus increase the frequency, volume and flow rates of runoff. The increased frequency, volume and flow rates are better able to carry sediment, chemical and biologic contamination from the land to receiving waters.

Some runoff from roofs and paved areas flows over natural ground, lawns and landscaped areas, or through unlined arroyos and ponds – all of which may significantly affect the frequency, volume and rate of runoff and often intercept much of the sediment, chemical and biologic contamination that would otherwise reach the receiving waters. Effective BMPs are those that mimic and enhance these casual water quality treatment processes.

Design for water quality protection in the desert southwest is a relatively new field, and because precipitation is infrequent and unreliable, data regarding the effectiveness of BMPs is not readily available. Without good data upon which to base analyses, designs must be based on the common-sense application of sound engineering principles and the reality of limited resources to perform maintenance of the resulting BMPs. The more the BMP emulates natural processes, the simpler it should be to construct and maintain.

What does that look like in practice?

Some basic natural principles for review and consideration:

- In nature, initial abstraction (wetting of surfaces, capture in depressions and on plants and trees) and infiltration happens almost entirely at the micro scale – at or near where the precipitation falls.
- The role of vegetation:
  - capture precipitation (how long can a person stand under a tree in the rain?),
  - enhance infiltration by making the soil less dense,
  - slow runoff flowing across the land surface, creating more opportunity for both abstraction and infiltration
  - trap sediments
  - uptake and transpire water from the soil increasing its infiltration capacity
  - create litter which absorbs water, slows runoff as it flows across the ground and ultimately can be incorporated into the soil as organic matter which further enhance plant growth and increases infiltration
  - support microbial communities which break down oils and other contaminants
  - sequester nutrients from runoff and use them for plant growth
- In New Mexico, macro scale runoff-producing precipitation occurs infrequently on most natural landscapes (from once every two to three years up to 2 to 3 times a year). See **Table 703-1**.
- In New Mexico, runoff-producing precipitation ( $\geq 0.1$  inch) occurs 20-40 times a year on directly connected impervious surfaces. However, when viewed on a monthly basis and for the purposes of using this runoff to establish and/or support vegetation a different runoff result is formed. See **Table 703-2** on the following page.



**Table 703-1 New Mexico Precipitation Data**

Source: USEPA, Office of Wastewater Management, Water Permits Division, Municipal Branch-  
*"Estimating Predevelopment Hydrology for Urbanized Areas in New Mexico"* March 2015.

Western Regional Climate Center Local Climate Data Summaries for Western U.S.

[https://wrcc.dri.edu/Climate/comp\\_tables.php](https://wrcc.dri.edu/Climate/comp_tables.php)

New Mexico Precipitation Data	Average Number of Days in a Year with Precipitation Greater Than or Equal to: (inches)				
	0.01	0.1	0.25	0.5	1
Daily Precipitation Total >=					
Albuquerque	57.2	25.9	12.3	4.3	0.6
Carlsbad	55.3	25.4	13.1	7	2
Clayton	66.5	31.2	17.9	8.2	2.5
Clines Corner	81.3	35.9	16.5	7.1	1.2
Deming	56.2	24.5	11.9	5	0.8
Farmington	64.5	24	9	2.4	0.3
Gallup	74.8	31.5	13.1	3.9	0.5
Grants	68.4	26.4	10.1	2.6	0.2
Las Vegas	76.7	35.4	17.7	7.8	1.5
Raton	84.7	32.9	16.2	6.9	1.8
Roswell	57.2	24.4	14.3	7.6	2.2
Santa Fe	67.9	28.7	13.8	4.8	0.9
Truth or Consequences	52	22.4	11.3	4.8	1
Tucumcari	66.4	32	20.1	9.5	2.8

**Table 703-2 Monthly Runoff Producing Rainfall Events from Impervious Areas**

Source: USEPA, Office of Wastewater Management, Water Permits Division, Municipal Branch-  
*"Estimating Predevelopment Hydrology for Urbanized Areas in New Mexico"* March 2015.

Western Regional Climate Center Local Climate Data Summaries for Western U.S.

[https://wrcc.dri.edu/Climate/comp\\_tables.php](https://wrcc.dri.edu/Climate/comp_tables.php)

Runoff Producing Rainfall Events from Impervious Areas	Average Number of Days by Month with Precipitation Greater than or Equal to 0.1 inch						
	March	April	May	June	July	August	September
Albuquerque	2.1	1.9	0.8	1.5	4.4	4.1	2.3
Carlsbad	1.8	1	2.1	2.8	2.9	4.1	3.1
Clayton	2.5	2.8	2.6	4	4.5	5.6	3
Clines Corner	2.5	1.9	2.5	2.4	5.8	5.8	3.5
Deming	0.5	1.1	0.5	1.3	5.4	4.2	2.1
Farmington	2.2	1.7	1.1	0.3	2.1	2.6	1.9
Gallup	1.9	1.8	1.1	1.1	5.3	5.3	3
Grants	1.6	1	0.9	1.6	4.6	4.5	2.5
Las Vegas	1.5	2	3.4	4.1	5.5	6.8	3.6
Raton	2.9	2.6	2.9	4.5	5	4.9	3.6
Roswell	2	1.4	1.6	2.3	2.9	3.7	2.6
Santa Fe	2.1	1.7	1.5	1.8	4.4	5.1	2.9
Truth or Consequences	0.8	0.9	1.3	1.2	5	4.3	2.8
Tucumcari	2.4	2.2	3	3.8	3.7	5.8	2.5

Some basic natural principles for review and consideration (continued):

- When selecting and/or designing a BMP (which is effectively a water treatment facility), the volume, quality and frequency of the inflow must be determined
- Acknowledging the frequency and seasonal timing of an average season of runoff events should be considered in the type of facility proposed and the need for and ease of maintenance for it to remain functional

## 800 MATERIAL SELECTION AND OTHER CONSIDERATIONS

### 801 Service Life of Culverts

A significant number of NMDOT culverts are approaching the end of their serviceable life. In some cases, deterioration is evident due to heavy equipment damage, silting, abrasion and rusting. Concrete box culverts exhibit spalling, where concrete flakes away and exposes reinforcing steel. This is exacerbated by the presence of water and its interactions with the chemistry of the soils. In addition, abrasion affects the serviceability of culverts. Abrasion is the deterioration of culvert bottoms by the pummeling of cobbles, boulders, gravels and other debris over the life of a culvert. The consequence of abrasion is that it chips away at the protective coating of a culvert exposing the underlying metal, thus speeding up the corrosion process.

In the past two decades, the NMDOT has relied on a rating system for determining types of culverts based on the resistivity of soils, pH, and the amounts of dissolved salts. The rating categories, Corrosion Resistance Number (CRN), are shown in Section 570.2.3.1 in the 2014 Edition of the NMDOT Standard Specifications for Highway and Bridge Construction. CRN ranges from CR1 up to CR7. A rating of CR1 is the most benign of conditions in soil and water where corrosion is not likely. But, CR7 rating presents very harsh environments significantly affecting a culvert's serviceable life.

This section will give guidance in the selection of the CRN for culverts. CRN, gage thickness (if metal culvert), and Manning's "n" value shall be put in the structural quantities sheet in the plan set for a project. CRN shall still apply in the case of concrete culverts and others.

It should be noted that prior to determining CRN, hydraulics and hydrology for a culvert should, by this point, have already been determined such as flow rates, velocities, hydraulic grade lines, aggradation, degradation, and other factors.

The NMDOT Drainage Design Bureau can be contacted to obtain a copy of a spreadsheet used to calculate CRN. Note that the Engineer/Consultant is responsible for understanding the use of, and the accuracy of the results of, this spreadsheet.

#### 801.1 Culvert Corrosion

Corrosion of metal culverts occurs on the interior and/or exterior of the pipe. The latter is an indication of corrosive materials of soils working in tandem with the presence of water. The former is caused by a variety of factors: very low or very high pH in water caused by the presence of dissolved salts, silt build-up where dissolved salts and water are present, or abrasion has occurred. There are other factors that can contribute to pipe corrosion such as anaerobic conditions allowing certain bacteria to grow, or the degree of water hardness.

All culverts to be used in NMDOT projects must be assessed based on the criteria given in **Table 801-1**. Soil resistivity, pH, amount of salts in water and soil will need to be known to make a proper determination of the CR value to be placed on the proposed culvert.



**Table 801-1 Table of Corrosion Resistance Numbers for Culverts**

CORROSION RESISTANCE TABLE FOR 50 YEAR SERVICE LIFE							
NEW MEXICO DEPARTMENT OF TRANSPORTATION							
Date: March 4, 2018	CORROSION RESISTANCE NUMBER						
JSL-DT	CR1	CR2	CR3	CR4	CR5	CR6	CR7
METALLIC	ACCEPTABILITY / RECOMMENDATIONS						
Galvanized Steel	yes	no	no	no	no	no	no
Aluminized Steel (Type II)	yes	yes	yes	no	no	no	no
Aluminum Alloy	yes	yes	yes	yes	yes	no	no
Polymeric Precoated Galvanized Steel (250 µm both sides)	yes	yes	yes	yes	yes	yes	no
Aramid Fiber Bonded Galvanized Steel	yes	yes	yes	yes	yes	yes	yes
CONCRETE RCP & CIPCP*						if soil has a pH<5.0, provide concrete with rapid chloride permeability of ≤1200 coulombs as tested in accordance with with ASTM 1202. or if pH>12.0, use Epoxy coating (280 mils, total)	
Cement: (Ref. Spec. Section 510)							
Type II	yes	yes	yes	yes	yes	yes	no
Type V	yes	yes	yes	yes	yes	yes	yes
THERMOPLASTIC							
HDPE & PVC	yes	yes	yes	yes	yes	yes	yes
STRUCTURAL PLATE (STEEL & ALUMINUM)	Use the Service Life Expectancy methods given in 801.1 to determine thickness or gage required for a fifty year service life. See Electrochemical Criteria Table (571.5.5:1) of 2014 Edition of Standard Specifications for backfill and bedding requirements.						
CONCRETE and METAL ATTACK							
	Negligible		Positive		Considerable		Severe
CONDUCTIVITY mS/cm (MILLISIEMENS PER CENTIMETER) for BOTH SOIL & WATER**							
	≤0.5	≤ 0.67	≤ 1.0	≤ 1.0	≤ 3.64	GREATER THAN 3.64	
MINIMUM RESISTIVITY (OHM-CM) for BOTH SOIL & WATER							
	≥2000	≥1500	≥1000	≥1000	≥275	<275	
pH LEVELS							
	6.0 - 9.0	5.0 - 9.0		4.0 - 12.0		<4.0 OR >12.0	
SOIL CHARACTERISTICS (from Alkali samples)							
Soluble Salts (Cl) & SO <sub>4</sub> (% by weight)	≤0.0500		≤0.0750		≤0.1250	≤0.2000	>0.2000
WATER CHARACTERISTICS (from Water samples)							
Soluble Salts (Cl) & SO <sub>4</sub> (% by weight)	≤0.0250		≤0.0375		≤0.0625	≤0.1000	>0.1000

**\*\* NOTE \*\*** METALLIC Pipe: CR# based primarily on pH and minimum resistivity.

NON-METALLIC Pipe: CR# based primarily on pH and % salts.(1%=10,000 ppm)

\* RCP -Reinforced Concrete Pipe; CIPCP - Cast in Place Concrete Pipe

\*\* Values given for milliseimens per centimeter (mS/ cm) can be substituted with deciseimens per meter (dS/ m)

A useful tool for initial assessment of soil chemistry of a site is data from the NRCS (<https://websoilsurvey.sc.egov.usda.gov/App/HomePage.html>). Data from the NRCS website can be downloaded that give electrical conductivity, pH, and/or other chemical properties. Note that NRCS uses conductivity rather than resistivity for its soils. Units for conductivity are given in Siemens per centimeter (S/cm), and resistivity are given in ohms-centimeter ( $\Omega$ -cm). The relation between conductivity (S/cm) and resistivity ( $\Omega$ -cm) is as follows:

$$\text{Conductivity} = 1/\text{resistivity} \quad \text{OR} \quad \text{S/cm} = 1/(\Omega\text{-cm})$$

Therefore, 1000  $\Omega$ -cm is equivalent to 0.001 S/cm or 1mS/cm (milliseimens per cm). NRCS units are deciseimens per meter (1 dS/m or 100 mS/m).

It must be cautioned however that tests for conductivity and resistivity are different and are for very different aims. NRCS methods are to test for presence of dissolved salts whereas the latter is for electrical grounding. At this time, it is not recommended using NRCS conductivity to make a final determination of CR values until further research is done.

Chemical properties can be determined in a lab from soil samples or from tests completed in the field.

To begin, all culverts should be inspected. The investigator should note the condition of a culvert for rusting (inside as well as exterior of pipe), signs of abrasion, absence or presence of water, and the type of bedload in the channel. This includes all other items for a typical culvert inspection such as size and type of culvert, aggradation, degradation of channel, etc. In some cases, the interior of a culvert may need to be inspected by a robotic device equipped with a camera and other sensors to further assess the culvert condition.

## 801.2 Determination of Service Life of Culverts

It would be desirable to select the culvert with the greatest service life possible, but other factors such as cost may be prohibitive. The limit for the service life of NMDOT culverts shall therefore be 50 years or more. Anything less will be unacceptable. Appendix 9 contains **Example Problem 9-1** that presents a corrosion resistance number calculation.

### 801.2.1 Galvanized Steel Culverts

**Figure 801-1**, from the NCSPA “Corrugated Steel Pipe Design Manual” (2008), shows the method to find the Estimated Material Service Life (EMSL) for a 16 gage galvanized steel culvert. For pH of 7.3 or less, resistivity (given in  $\Omega$ -cm) and pH govern and **Equation 801-1** applies. For pH greater than 7.3 resistivity governs, and **Equation 801-2** applies.

EMSL is the “Estimated Material Service Life” of a culvert in years. EMSL will be used interchangeably with Average Service Life and is defined as 25% removal of the thickness of the culvert wall at the invert, where most damage usually occurs.

For pH of 7.3 or less, pH and resistivity govern:

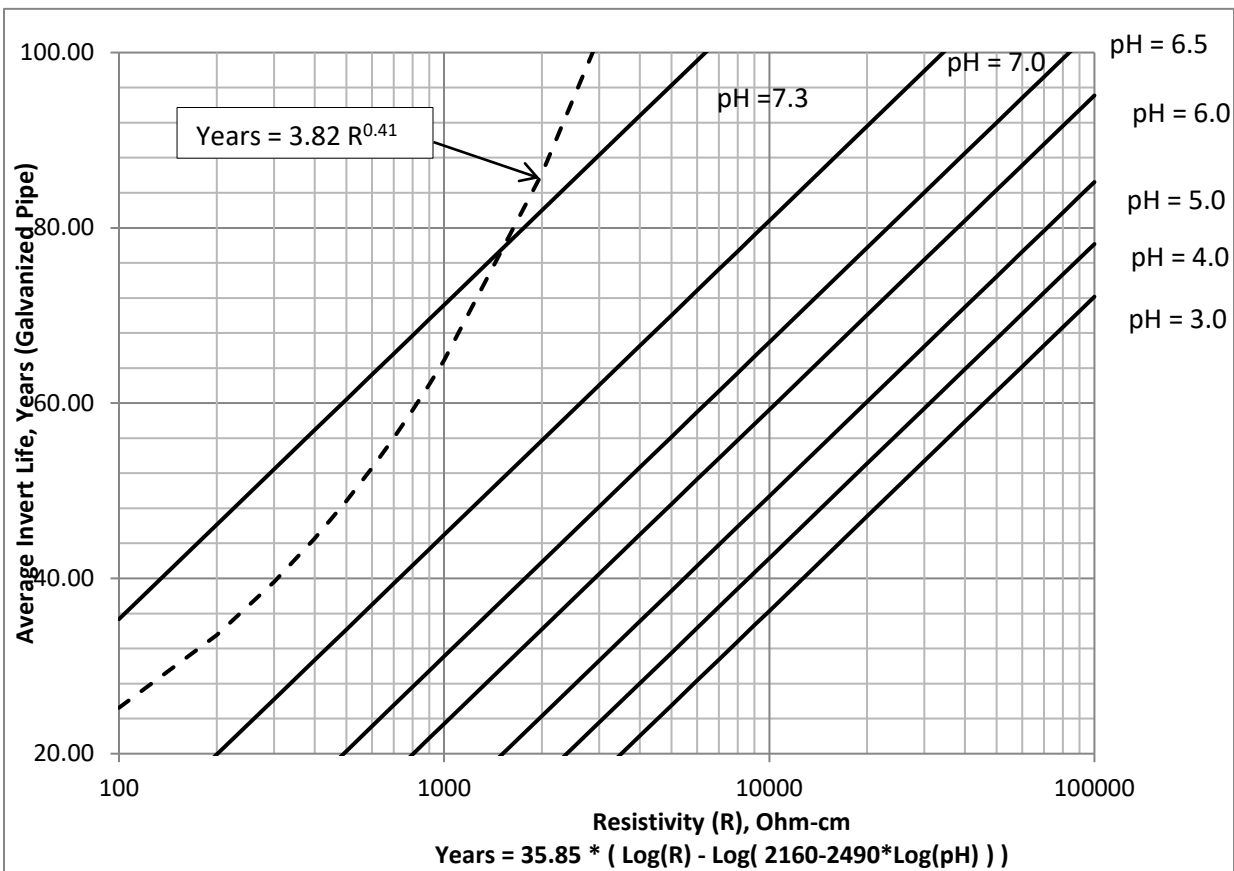
$$\text{EMSL} = 35.85 \left( \log_{10} R - \log_{10} (2160 - 2490 \log_{10} \text{pH}) \right) \quad \mathbf{801-1}$$

where:

R = resistivity

For pH greater than 7.3, resistivity governs:

$$\text{EMSL} = 3.82 R^{0.41} \quad \mathbf{801-2}$$



**Figure 801-1 Average Invert Life (years) for Galvanized Pipe**

**Table 801-2** gives the relation of gage value to its thickness and the Factor (F). The Factor (F) given here is the multiplier to apply in **Equation 801-3** for the selected gage thickness of culvert. Arithmetically:

$$\text{EMSL}_{\text{adjusted}} = F * \text{EMSL}$$

**801-3**

EMSL<sub>adjusted</sub> is the adjusted number of years, and F is the Factor to apply, and is shown in **Table 801-2**.

**Table 801-2 Gage Thicknesses for Metal Culverts.**

Factor (F) in conjunction with **Figure 801-1**.

<b>Gage</b>	<b>18</b>	<b>16</b>	<b>14</b>	<b>12</b>	<b>10</b>	<b>8</b>
Thickness (mm)	1.3	1.6	2	2.8	3.5	4.3
Thickness (inches)	0.052	0.064	0.079	0.109	0.138	0.168
Factor (F)	0.7	1	1.3	1.8	2.3	2.8

Note that if resistivity is greater than 10,000 ohm-cm, consideration must be given to water hardness:

“In the presence of hard waters ( $\text{CaCO}_3 > 50$  ppm), a process called scaling results in the deposition of a protective barrier on the pipe surface and reduces corrosion. While increasing amounts of  $\text{CaCO}_3$  protect the pipe, increasing levels of dissolved oxygen and  $\text{CO}_2$  can accelerate corrosion. The most important effect of increased levels of  $\text{CO}_2$  in water relates to its interference with the formation of protective  $\text{CaCO}_3$  scale that develops on galvanized pipe surfaces. High resistivity levels in water ( $R > 10,000$  ohms-cm) may indicate soft water ( $\text{CaCO}_3 < 50$  ppm). Soft water has the reduced ability to neutralize acid events often attributed to air pollution and acid rain. This condition in combination with minimum thickness of protective scale is conducive to accelerated corrosion rates in galvanized steel” (p. 479, Corrugated Steel Pipe Design Manual, NCSPA, 2008).

### **801.2.2 Aluminized Steel Culverts (Type II)**

For an aluminized steel culvert, a pH equal to or greater than 5.5 (up to pH 9) and a resistivity of 1500 ohms-cm or greater gives a service life of 50 years or more. Otherwise, service life is less than 50 years which is deemed unacceptable.

### **801.2.3 Concrete Culverts**

Concrete culverts are resistant to most soil conditions that pose problems with metal culverts. They are, however, sensitive to dissolved salts containing chlorine (Cl), or sulphates ( $\text{SO}_4$ ) where it affects the pH. A pH less than or equal to 5 will require further testing such as a rapid chloride permeability test to check for readings greater than 1200 coulombs (ASTM 1202) for a

Type V cement. Otherwise, additives may need to be put in the concrete mix. Use of Type V cement for a concrete pipe culvert shall be determined by the NMDOT Concrete Engineer.

#### **801.2.4 Plastic Culverts**

Plastic pipe includes High Density Polyethylene (HDPE) culverts and Polyvinyl Chloride (PVC) culverts. They can handle all soil and water conditions given in **Table 801-1**. Other types of plastic pipes have come into the market and future determination of their CRN values will need to be evaluated.

#### **801.2.5 Other Considerations, Coatings Applied to Culverts**

Addition of a bituminous coating to galvanized metal culvert will increase service life by 25 years. A polymer pre-coating will increase service life by 50 years. An interior bituminous coating is not recommended for perennial waterways or environmentally sensitive areas.

### **801.3 Culvert Abrasion**

The final step to determine service life is to analyze for potential abrasion. When very low bedloads are present (i.e., a closed system such as a storm sewer), higher velocities are not a concern.

Abrasion velocities should be determined from the flow frequency of 5 years or less (2 year, bankfull). Perennial streams with longer peaks should be given consideration for increased abrasion.

Invert protection coatings can be applied in accordance with the following abrasion criteria:

- Level 1: Non-abrasive – No bedload. Velocities can be greater than 15 ft/s
- Level 2: Low-abrasion – Minor bedloads of sand and gravel with velocities at 5 ft/s or less. Level 2 is applicable for storm drain applications.
- Level 3: Moderate abrasion – Bedloads of sands and gravels with velocities between 5 ft/s and 15 ft/s
- Level 4: Severe abrasion – Heavy bedloads of gravel and rock with velocities exceeding 15 ft/s

**Table 801-3** shows adjustments made to various culvert types under different abrasion conditions (note that 'add one gage' means to make thicker by one gage). Refer to **Table 801-2** for gage thickness.

**Table 801-3 Abrasion Criteria**

<b>Material</b>	<b>RECOMMENDED ADJUSTMENTS FOR ABRASION</b>			
	<b>Low Abrasion Level 1</b>	<b>Mild Abrasion Level 2</b>	<b>Moderate Abrasion Level 3</b>	<b>Severe Abrasion Level 4</b>
Concrete Pipe	No Addition	No Addition	No Addition	Modify Mix Design
Aluminized Steel Type II	No Addition	No Addition	Add One Gage	Add One Gage and Pave Invert
Galvanized Steel (2 & 3 oz. Coating)	No Addition	Add One Gage *	Add Two Gages *	Do Not Use
Polymer Precoated Galvanized Steel	No Addition	No Addition	Add One Gage	Add One Gage and Pave Invert
Aramid Fiber Bonded Galvanized Steel	No Addition	No Addition	No Addition	Add One Gage
Aluminum Alloy	No Addition	No Addition	Add One Gage	Add One Gage and Pave Invert
Thermoplastic Pipe (PVC & HDPE)	No Addition	No Addition	No Addition	Do Not Use
* A field applied concrete paved invert per ASTM A 849 may be substituted for one (1) gage thickness				

Appendix 9 contains **Example Problem 9-1** that presents an abrasion example to determine proper culvert wall thickness.

## 802 Other Pipe and CBC Considerations

The designer shall select the appropriate pipe or concrete box culvert (CBC) based on hydraulic requirements, geometry constraints, soil corrosion, economy, constructability, maintenance, and local government preferences. For wall thickness and cover height on all structures, refer to the NMDOT Standard Drawing(s) 511.

When fitting structures inside larger structures (also referred to as slip-lining), care should be taken to ensure a proper fit, considering proposed structure wall thickness and buffer on all sides.

When attaching structures to manholes or junction boxes, the designer must address constructability by checking the attachment angles and sizes as well as number of attachments. Manholes and junction boxes need to keep their structural integrity and should conform to the requirements of NMDOT Standard Drawings 511, 623 and 662.

### 803 Access and Development Review

For all access and development plans affecting the drainage within NMDOT right-of-way (ROW), or reviewed by the NMDOT, the developer needs to submit plans to the NMDOT Drainage Bureau for review and approval. The post-construction peak flow calculations need to show that pre-construction flow rates are not exceeded. No additional flows other than the historic flows are permitted inside of NMDOT ROW. No ponds, sub-surface detention areas, or water quality features are permitted inside of NMDOT ROW. Tie-ins to NMDOT owned and operated storm drains are not allowed. All drainage calculations must conform to the latest NMDOT drainage criteria and manuals, unless local entity requirements are more stringent.

All turnouts to NMDOT ROW must be constructed with waterstops (humps), matching the height of the adjacent curb and gutter or having a minimum height of 4" if curb and gutter is not present. If full-height waterstops are not geometrically feasible, consult with the NMDOT Drainage Engineer for alternative configurations. Turnouts or driveways may discharge runoff to the NMDOT ROW provided that the contributing runoff is included in design calculations for the roadway and storm drain system. If the NMDOT will discharge roadway runoff to private property, then drop inlets or other methods to reduce the runoff down the turnout, should be installed immediately upstream of the turnout.

The developer needs to comply with the USEPA NPDES program requirements for SWPPP preparation and MS4 mandates when discharging flows to the NMDOT ROW.

### 804 Other Considerations

Wire-enclosed riprap and gabions shall be designed according to the NMDOT Standard Drawing(s) 602. NMDOT uses wire-enclosed riprap at structure outlets, bridge abutments, and channel banks for erosion protection, except for unusual circumstances such as high velocities, large flow rates, long spans, and steep slopes. Wire-enclosed ripraps low flexibility, susceptibility to wire corrosion and abrasion, and relatively small size rocks make it unfit for erosion protection against large flows with high velocities. Other options include loose riprap of various sizes and articulated concrete revetment mats. See HEC-14, HEC-18, and HEC-23 for guidance on rock size selection.

Gabions can be used in lieu of wire-enclosed riprap for steep bank protection, bridge abutments, spur dikes, and channel protection.

Rock size selection for erosion protection needs to be designed according to design guidelines in HEC-14, HEC-18, and HEC-23. The minimum required rock size determines what class of riprap to use. If the required minimum rock size is larger than 4", NMDOT Class A wire-enclosed riprap and gabions may not be the right options.

## 805 References

FHWA, July 2006, “HEC-14, Hydraulic Design of Energy Dissipators for Culverts and Channels, Third Edition”.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/06086/hec14.pdf>

FHWA, September 2009, “HEC-23, Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance, Third Edition, Volume 1”.

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/09111/09111.pdf>

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NCHRP, 2015, “NCHRP Synthesis 474, Service Life of Culverts”.

<https://www.nap.edu/download/22140>

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

NRCS, 2017, “Web Soil Survey (WSS)”.

<https://websoilsurvey.sc.egov.usda.gov/App/HomePage.html>

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<https://ncspa.org/wp-content/uploads/2016/07/2ndedncspsacspdm.pdf>



# **APPENDIX 1 DEFINITIONS**

absorption	The act or process of taking in water by inflow of atmospheric vapor, hygroscopic absorption, wetting, infiltration, influent seepage, and gravity flow of streams into sinkholes or other large openings.
abstraction	That portion of rainfall which does not become runoff. It includes interception, infiltration, and storage in depression. It is affected by land use, land treatment and condition, and antecedent soil moisture.
accretion	1. A process of accumulation of flowing water whether of silt, sand, pebbles, etc. Accretion may be due to any cause and includes alluviation. 2. The gradual building up of a beach by wave action. 3. The gradual building of the channel bottom, bank or bar due to silting or wave action.
acre-foot	The amount of water that will cover 1 acre to a depth of 1 foot. It equals 43,560 cubic feet. Typically abbreviated as ac-ft.
aggradation	General and progressive sediment deposition of a channel bed in the longitudinal profile.
allowable headwater	The depth or elevation of impounded water at the entrance to a hydraulic structure, above which flooding, or some other unfavorable result could occur.
alluvial channel	A channel wholly in alluvium with no bedrock exposed in the channel at low flow or likely to be exposed by erosion during major flow.
alluvial fan	A sediment deposits whose surface forms a segment of a cone that radiates downslope from the point where the watercourse leaves the source area.
alluvium	Unconsolidated clay, silt, sand, or gravel deposited by a stream in a channel, floodplain, delta, alluvial fan, or pediment.
annual flood	The highest peak discharge in a water year.
annual series	A frequency series in which only the largest value in each year is used, such as annual floods.
annual yield	The total amount of water obtained in a year from a stream, spring, artesian well, etc. May be expressed in inches (depth), acre-feet, millions of gallons, or cubic feet.

Antecedent Runoff Condition	The amount of moisture in the soil and plants at the beginning of the storm. Antecedent moisture conditions affect the volume of runoff generated by a particular storm event. As storm magnitudes increase, antecedent moisture has a rapidly decreasing influence on runoff because the soils become saturated.
areal rainfall	The average rainfall over an area, usually as derived from, or discussed in contrast with point rainfall.
bank	Lateral boundaries of a channel or stream, as indicated by a scarp, or on the inside of bends, by the streamward edge of permanent vegetal growth.
base flow	Stream discharge derived from groundwater sources. Sometimes considered to include flows from regulated lakes or reservoirs. Fluctuates much less than storm runoff.
basin, drainage	The area of land drained by a watercourse.
basin lag	The time from the centroid of the excess rainfall hyetograph to the hydrograph peak.
bed (of a channel or stream)	The part of a channel without permanent vegetation, bounded by banks, over which water normally flows.
berm	A narrow shelf or ledge; also, a form of dike.
bridge	A structure including supports erected over a depression or an obstruction, such as a watercourse, highway, or railway, and having a tract or passageway for carrying traffic or moving loads. A bridge has an opening measured along the center of the roadway of more than 20 feet between under copings of abutments or spring lines of arches, or extreme ends of openings for multiple boxes. A bridge may also include multiple pipes, where the clear distance between openings is less than half of the smaller contiguous opening. A bridge is designed hydraulically using the principles of open channel flow to operate with a free water surface but may be inundated under flood conditions.
broken-back culvert	A culvert with one or more changes in slope within the length of the culvert. An alternative to a steeply sloped culvert accomplished by breaking the culvert into steeper and flatter sections.
bulking factor	A factor applied to indicate that a clear water peak discharge and runoff volume include an increase to account for sediment volume within the clear water volume.

capacity	A measure of the ability of a channel or conduit to convey water.
catch basin	A structure with a sump for inletting drainage from a gutter or median and discharging the water through a conduit. In common usage it is a grated inlet with or without a sump.
catchment	See watershed.
catchment area	The area tributary to a lake, stream, or drainage system.
channel	1. The bed and banks that confine the surface flow of a natural or artificial stream. Braided streams have multiple subordinate channels which are within the main stream channel. Anabranching streams have more than one channel. 2. The course where a stream of water runs, or the closed course or conduit through which water runs, such as a pipe.
channel routing	The process where a hydrograph is translated through a channel from its origin to a downstream location.
check dam	A low structure, dam or weir, across a channel for the control of water stage or velocity, or to control channel erosion. Also called a grade control structure.
Check Flood	The flood flow applied in hydraulic analyses that is a more intense flood (less frequent) than the corresponding Design Flood and is applied as a secondary criterion for design.
control section	A cross section, such as a bridge crossing, reach of channel, or dam, with limited flow capacity, in which the discharge is related to the upstream water-surface elevation.
conveyance	A measure, $K$ , of the ability of a stream, channel, or conduit to convey water. In Manning's formula, $K = (1.49/n) AR^{2/3}$ .
cover	The depth of soil above the crown of a pipe or culvert. The vegetation, or vegetational debris such as mulch that exists on the soil surface. The percent of ground cover and cover types are fundamental parameters for determining Runoff Curve Numbers.
criterion	A standard, rule, or test on which a judgement can be based (criteria is plural).
cross drainage	The runoff from contributing drainage areas both inside and outside the highway right-of-way and the transmission thereof from the upstream side of the highway facility to the downstream side.

cross section	The shape of a channel, stream, or valley, viewed across its axis. In watershed investigations it is determined by a line approximately perpendicular to the main path of water flow, along which measurements of distance and elevation are taken to define the cross-sectional area.
cubic feet per second	A unit of water flow rate, sometimes called “second-feet”. Typically abbreviated as cfs.
culvert	<p>The following definition is from AASHTO, 2014, “AASHTO Drainage Manual, Chapter 11”.</p> <ul style="list-style-type: none"><li>– A structure that is usually designed hydraulically to take advantage of submergence to increase hydraulic capacity.</li><li>– A structure used to convey surface runoff through embankments.</li><li>– A structure, as distinguished from bridges, that is usually covered with embankment and is composed of structural fill material around the entire perimeter, although some structures are supported on spread footings or piles with the streambed serving as the bottom of the culvert (bottomless culvert).</li><li>– A structure that conveys flow from one side of the Right-of-Way (ROW) or roadway to the other, passing under the roadway. This is generally a straight pipe or Concrete Box Culvert (CBC) with no bends or elbows. This structure may have a cattle pass or other median drain dropping in from above.</li></ul>
dead storage	Storage volume located below the lowest drain invert elevation in a pond or reservoir. This storage volume may be included in the storage design to accommodate sediment accumulation.
debris	Material transported by the stream, either floating or submerged, such as logs, brush or man-made items.
degradation	General and progressive lowering of the longitudinal profile of a channel bed by erosion.
deposition	The settling of material to the bottom from either stream flow or ponded water.
depression storage	The natural depressions within a watershed that store runoff. Generally, after the depression storage is filled runoff will commence.
design discharge or flow	The rate of flow for which a facility is designed.
Design Flood	The flood flow that is selected as the basis for hydraulic analysis and design, or evaluation of a hydraulic structure.

design flood frequency	The recurrence interval that is expected to be accommodated without contravention of the adopted design constraints. The return interval (recurrence interval or reciprocal of probability) used as a basis for the design discharge.
design storm	A specified rain event of selected frequency (25-year, 50-year, etc.) that has a specified depth, duration and distribution.
detention pond	A stormwater storage facility, where stormwater is detained and released by either a controlled or uncontrolled outlet. A controlled outlet has a gate, valve or other means to control the outlet discharge. Uncontrolled outlets have no means of controlling the outlet discharge except for the inherent structure hydraulic capacity. Detention ponds attenuate the inflow hydrograph such that the outflow hydrograph has a reduced peak discharge and longer duration.
detour	A temporary change in the roadway alignment. It may be localized at a structure or may be along an alternate route.
direct runoff	The water that enters the stream channel during a storm or soon after, forming a runoff hydrograph. May consist of rainfall on the stream surface, surface runoff, and seepage of infiltrated water (rapid subsurface flow).
discharge	The flow rate as volume per unit of time, usually expressed in cfs.
drainage area	The area draining into a stream at a given point. The area may be of different sizes for surface runoff, subsurface flow, and base flow, but generally the surface flow area is used as the drainage area.
drainage structure	A conduit for conveying storm water away from or under a roadway. Often a closed conduit such as a culvert could also be a bridge.
effective duration	The time in a storm during which the water supply for direct runoff is produced. Also used to mean the duration of excess rainfall.
ephemeral stream	A stream or reach of a stream that does not flow continuously for most of the year.
erosion	The wearing away or scouring of material in a channel, opening, or outlet works caused by flowing water.
evapotranspiration	Represents the total evaporation from water, snow, ice, vegetation and other surfaces plus transpiration.
excess rainfall	Direct runoff.

exfiltration	The process by which stormwater seeps or flows to the surrounding soil through openings in a conduit.
flood	In common usage, an event that overflows the normal banks. In technical usage, it refers to a given discharge based, typically, on a statistical analysis of an annual series of events.
flood frequency	The average time interval, in years, in which a given flood or will be equaled or exceeded.
flood of record	Reference to the maximum estimated or measured discharge that has occurred at a site.
floodplain	The alluvial land bordering a stream, formed by stream processes, that is subject to inundation by floods.
flood routing	Determining the changes in a flood hydrograph as it moves or translates downstream through a channel or through a reservoir.
flow distribution	The estimated or measured spatial distribution of the total streamflow.
freeboard	The vertical distance between the level of the water surface, usually corresponding to the design flow and a point of interest.
freeboard for a bridge	The vertical distance between the lowest hanging superstructure member of the bridge (low chord) and the water surface immediately below it.
freeboard for a pond	The vertical distance from the water surface elevation to the lowest spillway crest or the lowest point of the embankment crest.
frequency	In analysis of hydrologic data, the recurrence interval is simply called frequency.
groundwater	Subsurface water occupying the saturation zone, from which wells and springs are fed. A source of base flow in streams. In a strict sense the term applies only to water below the water table. Also called phreatic water.
guide banks	Embankments built upstream from one or both abutments of a bridge to guide the approaching flow through the waterway opening.
gutter	That portion of the roadway section adjacent to the curb which is utilized to convey storm runoff water.
headwater	The water depth at a culvert inlet as measured from the culvert invert to the water surface, abbreviated as HW.

high water elevation	The water surface elevation that results from the passage of flow. It may be "observed high water elevation" as a result of an event, or "calculated high water elevation" as part of a design process.
historical flood	A past flood event of known or estimated magnitude.
hydraulic capacity	Maximum flow a structure will convey while meeting a specific design criterion. This may vary with slope, material, headwater and other factors.
hydraulic grade line	The water surface elevation in an open channel, or non-pressure flow pipe. The hydraulic grade line in a pressure flow pipe represents the water surface elevation that would be reached if a piezometer tube was inserted into the center of the flow stream.
hydrograph	A graph showing, for a given point on a stream or for a given point in any drainage system, the discharge, stage, velocity or other property of water with respect to time.
hydrologic soil-cover complex	A combination of a Hydrologic Soil Group and a type of cover.
Hydrologic Soil Group	A group of soils having the same runoff potential under similar storm and cover conditions.
hydrology	The study of the occurrence, circulation, distribution, and properties of the waters of the earth and its atmosphere.
hyetograph	A graphical representation of average rainfall, rainfall-excess rates or volumes over specified areas during successive units of time during a storm.
impervious	Impermeable to water infiltration.
infiltration	That movement of water through the soil surface (see percolation).
infiltration rate	The rate at which water enters the soil under a given condition. The rate is usually expressed in inches per hour, or feet per day.
initial abstraction	The rainfall volume that must be satisfied, prior direct runoff. When considering direct runoff, initial abstraction (IA) consists of interception, evaporation, and the soil-water storage. Sometimes called "initial loss".
intensity	The rate of rainfall upon a watershed, usually expressed in inches per hour.



interception	Precipitation retained on plant or plant residue surfaces and finally absorbed, evaporated, or sublimated. That which flows down the plant to the ground is called "stemflow" and not counted as true interception.
isohyet	A line on a map, connecting points of equal rainfall amounts.
jurisdictional and non-jurisdictional dams	<p><i>(Definitions provided below were obtained from the New Mexico Office of the State Engineer Dam Safety Bureau (NMOSE, December 2010, "Rules and Regulations Governing Dam Design, Construction and Dam Safety"). <a href="http://www.ose.state.nm.us/DS/Regs/19-25-12-NMAC-2010.pdf">http://www.ose.state.nm.us/DS/Regs/19-25-12-NMAC-2010.pdf</a></i></p> <p><u>Jurisdictional Dam</u> – A dam that is 25 feet or greater in height which impounds more than 15 acre-feet of water, or a dam that impounds 50 acre-feet of water or more of water and is 6 feet or greater in height. Dam height is measured from the lowest point on the downstream toe of dam to the lowest dam crest elevation.</p> <p><u>Levee or Diversion Dike</u> – A structure where water flows parallel to the length of the levee or diversion dike as determined by the State Engineer.</p> <p><u>Roadway Embankment</u> – A structure designed across a watercourse for the sole purpose of supporting a roadbed or other means of conveyance for transportation as determined by the State Engineer; where the area upstream has not been enlarged to increase flood storage; and where the embankment is provided with an uncontrolled conduit of sufficient capacity to satisfy requirements of the appropriate state or local transportation authority. If no transportation authority has jurisdiction over the structure, the current drainage design criteria of the NM Department of Transportation shall apply.</p>
Lag Time	<p>Lag time (<math>T_L</math>) is time between the centroid of the excess rainfall (that rainfall producing runoff) and the peak of the runoff hydrograph. Often estimated as 60 percent of the Time of Concentration (<math>T_c</math>), <math>T_L = 0.6 T_c</math>.</p>
land use	A land classification. Cover, such as row crops or pasture, indicates a kind of land use. Roads may also be classified as a separate land use.
levee	A linear embankment outside a channel for containment of flow.
low chord	The lowest hanging superstructure member of a bridge.
major structure	A drainage conduit which is larger than a minor structure, yet smaller than a bridge.

Manning's "n"	A coefficient that represents hydraulic roughness, used in a formula for estimating the capacity of a channel to convey water.
mass inflow curve	A graph showing the total cumulative volume of stormwater runoff plotted against time for a given drainage area.
maximum embankment height	Defined as the vertical distance from the lowest point on the downstream embankment toe to the lowest point on the embankment crest as defined by the NM Office of the State Engineer Dam Safety Bureau (NMOSE, December 2010). This definition shall also apply to NMDOT pond embankments.
minor structure	A drainage conduit which less than or equal to a 48" circular pipe culvert, or equivalent hydraulic capacity.
municipal separate storm sewer system (MS4)	<p>This term was defined by the U.S. Environmental Protection Agency (USEPA) for use in addressing stormwater runoff quality. An MS4 is a conveyance or system of conveyances that are:</p> <ul style="list-style-type: none"><li>– Owned by a state, city, town, village, or other public entity that discharges to waters of the U.S.</li><li>– Designed or used to collect or convey stormwater (e.g., storm drains pipes, ditches).</li><li>– Not a combined sewer (sanitary and storm).</li><li>– Not part of a sewage treatment plant, or publicly owned treatment works (POTW).</li></ul>
new development	The process of adding improvements to a parcel of land, such as grading, subdivisions, drainage, access, roadway improvements, impervious driving surfacing and utilities. This applies to parcels of lands with little to no previous human-caused disturbances, or otherwise in a natural condition.
on-site drainage	Runoff from within the highway right-of-way, including pavement, medians, and road shoulders. Runoff may be collected by gutters, catch basins, swales, and rundowns.
overland flow	Runoff that has a shallow flow depth and is unconcentrated flow, often in the form of sheet flow.
partial duration series	A list of all events, such as floods, occurring above a selected base, without regard to the number of events, within a given period. In the case of floods, the selected base is usually equal to the smallest annual flood, in order to include at least one flood in each year.
peak discharge	Maximum discharge rate of a hydrograph.
percolation	Movement of water through the soil profile.

permeability	The property of a material that permits appreciable movement of water through it when it is saturated, and movement is actuated by hydrostatic pressure of the magnitude normally encountered in natural subsurface water.
perennial stream	A stream or reach of a stream that flows continuously for all or most of the year.
point rainfall	Rainfall at a single rain gage or location.
precipitation	The process by which water in liquid or solid state falls from the atmosphere.
pre-development runoff	Storm runoff generated from a site or an area in its natural condition, prior to development or disturbances.
probability	Probability analysis seeks to define the flood flow with a probability “p” of being equaled or exceeded in any year. Return period “r” is commonly used instead of probability “p” to define the design flood. Return period and probability are reciprocals, i.e. $p = 1/r$ . For example, an event with a 50-year return period, $r = 50$ , has a 2% probability of occurring in any given year, $p = 1/50 = 0.02$ .
rainfall excess	The water available to runoff after interception, depression storage, and infiltration rainfall losses have been satisfied.
rainfall intensity	Amount of rainfall occurring in a unit of time, typically converted to its equivalent in inches per hour.
rating curve	A graphical plot relating stage to discharge.
recession curve	The receding portion of a hydrograph, occurring after excess rainfall has stopped.
recharge basin	A basin excavated in the earth to receive the discharge from streams or storm drains for the purpose of replenishing groundwater supply.
re-development	Improvements made to a parcel of land that was previously developed (see “new development”). Examples include widened roadways and sidewalks. This does NOT apply to mill-and-pave activity within the roadway where no bare ground is exposed, but would apply to, for example, an additional sidewalk placed on previously bare ground during the same project.

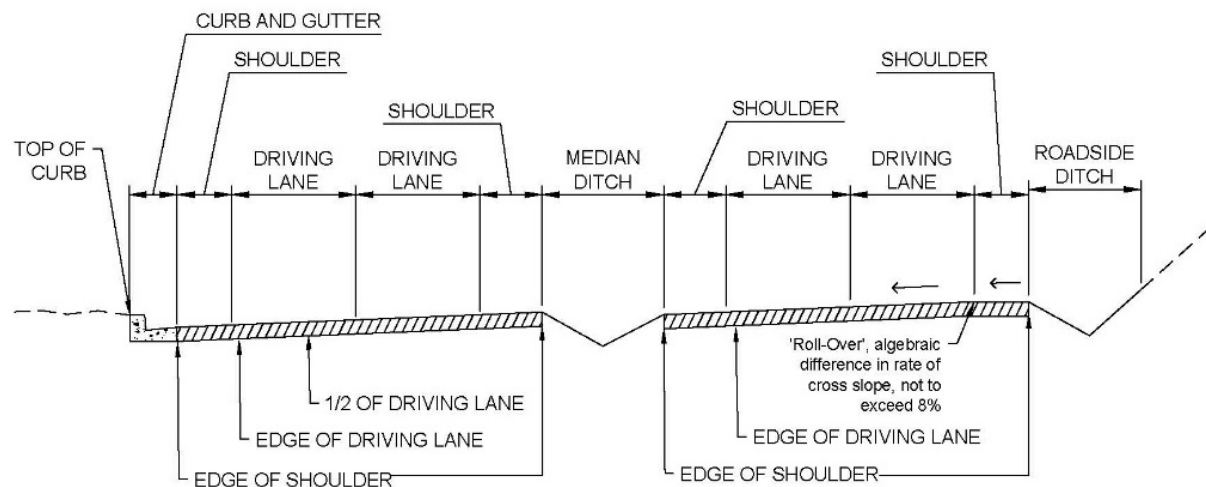
regional analysis	A regional study of gaged watersheds which produces regression equations relating various watershed and climatological parameters to discharge. Use for design of ungaged watershed with similar characteristics.
regulatory flood	Usually the 100-year flood, which was adopted by the Federal Emergency Management Agency (FEMA), as the base flood for floodplain management purposes.
regulatory floodway	The floodplain area that is reserved in an open manner by Federal, State, or local requirements, i.e., unconfined or unobstructed either horizontally or vertically, to provide for the discharge of the base flood so that the cumulative increase in water surface elevation is no more than a designated amount.
reservoir routing	The process where a hydrograph is translated through a reservoir.
retention pond	A stormwater storage facility, where stormwater is retained because the pond does not contain a positive outlet, or the pond may have a regulated or controlled outlet. All or a portion of the inflow hydrograph is retained for a prolonged period. Water loss over time may occur through evaporation and infiltration. Infiltration basins are a type of retention pond.
return period	The average number of years within which a given event will be equaled or exceeded "r". Also known as "storm frequency", or "recurrence interval" (see probability).
risk analysis/ assessment	An analysis approach used as means of justification for deviation from the design return period and criteria. Risk analysis is generally more detailed than risk assessment.
road classification	<p>The NMDOT has adopted the FHWA roadway functional classifications (FHWA, December 1995) as presented here (not defined). Refer to the American Association of State Highway and Transportation Officials (AAHSTO, 2011, "A Policy on Geometric Design of Highways and Streets, 6th Edition").</p> <ul style="list-style-type: none"><li>– Arterial, Principal</li><li>– Arterial, Minor</li><li>– Collector, Major</li><li>– Collector, Minor</li></ul>

## roll-over

A roll-over is defined (AASHTO, 2001, Fourth Edition) as the algebraic difference in cross slope between two adjacent paved areas which are separated by a crown or grade break. AASHTO, 2001, policy establishes the maximum acceptable roll-over between two adjacent travel lanes and between a travel lane and a shoulder. Maximum roll-over standards are as follows:

- Between travel lane and shoulder, any speed: 8%
- Between two travel lanes: 4%

Refer to **Figure 1** for one example of a roll-over where the shoulder is sloped into the travel way, and the roadside ditch on the high side of road section could spill into the shoulder if flow exceeds capacity. Roll-over road sections as shown in **Figure 1** should be avoided if possible to avoid excess stormwater runoff on the lower roadway section that is generated from the upper roadway / shoulder section.



**Figure 1 Roll-Over**

AASHTO, 2001, "A Policy on Geometric Design of Highways and Streets, Fourth Edition".

[http://nacto.org/docs/usdg/geometric\\_design\\_highways\\_and\\_streets\\_aashto.pdf](http://nacto.org/docs/usdg/geometric_design_highways_and_streets_aashto.pdf)

## runoff

Total rainfall minus interception, evaporation, infiltration, and surface storage which moves across the ground surface to a stream or depression.

## Runoff Coefficient

A factor representing the portion of storm rainfall that will become storm runoff. This factor represents Hydrologic Soil Group, land use type, vegetation type and density.

Runoff Curve Number	A factor representing the portion of storm rainfall that will become storm runoff. This factor represents Hydrologic Soil Group, land use type, vegetation type and density.
rural	Where the majority of the watershed area is agricultural or lightly used by human activity.
saturated soil	Soil that has its interstices or void spaces filled with water to the point at which runoff occurs.
sedimentation	The process involving the deposition of soil particles which have been carried by flood waters.
sequent depth	Flow depth immediately downstream from a hydraulic jump, also called conjugate depth.
sidewalk culvert	An enclosed rectangular concrete drainage channel located within a sidewalk section that conveys storm runoff through the sidewalk. The channel top is enclosed with a metal plate anchored to the concrete. The metal culvert plate allows sidewalk pedestrian traffic to travel across the drainage channel.
skewness	When data are plotted in a curve on log-normal paper, the curvature is skewness.
slotted pipe drain	A monolithic storm drain pipe and inlet, comprised of a narrow slot inlet (typically 1 ½ inches wide) located at the top of a vertical throat section that joins to the top of the pipe. Pipe diameters may range from 4 inches to 36 inches.
soil porosity	The percentage of the soil (or rock) volume that is not occupied by solid particles, including all pore space filled with air and water.
soil-water-storage	The amount of water the soils (including geologic formations) of a watershed will store at a given time. Amounts vary from watershed to watershed. The amount for a given watershed is continually varying as rainfall or evapotranspiration takes place.
stage	Height of water surface above a specified datum.

storm drain	A “storm drain” is a <u>pipe or system of pipes</u> with water-tight joints, that provides for the conveyance of stormwater that enters the system by means of one or more storm drain inlets, may have multiple geometric changes, or potential changes in flow regime in the barrel. A “trunk line” is the largest pipe that serves as the outfall for laterals and/or storm drain inlets. A “lateral” is a storm drain pipe that joins a storm drain inlet to the trunk line. If the system is more complicated than a simple straight pipe and a few inlets, it will be considered a storm drain.
storm drain pipe	A pipe that has a closed end (drop inlet) or is part of a network of storm drain pipes, or is connected to a manhole, or includes bends or elbows, or as determined by the NMDOT Drainage Engineer.
storm duration	Length of a storm. Use a 24-hour duration for all NMDOT analyses.
storm frequency	See “return period”.
stream channel	The bottom of a topographic area, where runoff collects and flows in a concentrated manner. May be ephemeral or perennial.
stream reach	A length of stream channel selected for use in hydraulic or other computations.
surface runoff	See “rainfall excess”.
surface storage	Stormwater that is contained in surface depressions or basins.
surface water	Water appearing on the surface in a diffused state, with no permanent source of supply or regular course, as distinguished from water appearing in water courses, lakes, or ponds.
synthetic hydrograph	A hydrograph determined from empirical rules. Usually based on the physical characteristics of the basin. A graph developed for an ungaged drainage area, based on known physical characteristics of the watershed basin.
swale	A slight depression in the ground surface where water collects and flows.
Time of Concentration	The time it takes surface runoff water to travel from the hydraulically most distant point to the watershed outlet. Time of Concentration ( $T_c$ ) varies with flood frequency but is often used as a watershed constant.
trainer dikes	Low embankments constructed to guide flows toward a drainage structure opening, often with erosion control revetments.

travel time	The average time for water to flow through a reach or other stream or valley length.
tributaries	Branches of the watershed stream system.
ungaged stream sites	Locations at which no systematic records are available regarding actual stream flows.
uniform flow	Flow of constant cross section and average velocity through a reach of channel during an interval of time.
unit hydrograph	A hydrograph of a direct runoff resulting from 1 unit of effective rainfall (inch) generated uniformly over the watershed area during a specified period of time or duration.
urban	A development condition within a watershed where large, contiguous areas have a significant quantity of impervious surfaces constructed by humans.
watercourse	Typically refers to a natural channel that has the capacity to convey water through a basin, and may be ephemeral or perennial.
watershed	The catchment or basin area for rainfall which is delineated as the drainage area producing runoff. Base flow in a stream is normally generated from the same area.
water table	The water surface of the zone of water saturation (groundwater) that is below the ground surface, except where that surface is formed by an impermeable body (perched water table).



# **APPENDIX 2   UNITS AND CONVERSION FACTORS**

<b>CONVERSION FACTORS FROM ENGLISH TO METRIC UNITS</b>			
<b>Quantity</b>	<b>To Convert From English Units</b>	<b>To Metric Units</b>	<b>Multiply by</b>
Length	mile	km	1.609347
	yard	m	0.9144
	foot	m	0.3048006
	foot	mm	304.8006
	inch	mm	25.4
Mass (weight)	pound	g	453.5924
	pound	kg	0.4535924
	pound	t (1000 kg)	0.00045359
	ton	kg	907.1847
	ton	t (1000 kg)	0.9071847
Area	square mile	km <sup>2</sup>	2.59
	acre	m <sup>2</sup>	4046.873
	acre	ha (10,000 m <sup>2</sup> )	0.4046873
	square yard	m <sup>2</sup>	0.8361307
	square foot	m <sup>2</sup>	0.09290341
	square inch	mm <sup>2</sup>	645.16
Volume	acre-foot	m <sup>3</sup>	1233.49
	cubic yard	m <sup>3</sup>	0.764559
	cubic foot	m <sup>3</sup>	0.028317
	cubic foot	cm <sup>3</sup>	28,317.02
	cubic foot	l (1000 cm <sup>3</sup> )	28.31702
	gallon	l (1000 cm <sup>3</sup> )	3.7854 l
	cubic inch	cm <sup>3</sup>	16.387162
	cubic inch	mm <sup>3</sup>	16,387.16
Velocity	feet/second	meters/second	0.305
Pressure or Stress	psi	Pa	6,894.76
	psi	kPa	6.894757
	ksi	MPa	6.894757
	lb(force)/ft <sup>2</sup>	Pa	47.88026
Force	pound-force	N	4.448222
	kip	N	4,488.22
Unit Weight	lbs/ft <sup>3</sup>	kg/m <sup>3</sup>	16.01846
	lbs/yd <sup>3</sup>	kg/m <sup>3</sup>	0.5932764
Flow	cubic ft/min	m <sup>3</sup> /s	0.000471947
	cubic ft/sec	m <sup>3</sup> /s	0.02831685
Note : Based on the U.S. survey foot where one meter = 39.37 inches			

COMMON HYDROLOGIC FACTORS		
Quantity	Equals	Comment
1 acre	43,560 sq ft	1 acre = 208.71 ft per side if square
1 mile	5,280 ft	---
1 square mile	27,878,400 sq ft	---
1 square mile	640 acres	---
1 acre-foot	43,560 cu ft	1 acre, 1 foot deep
1 cfs flowing for 1 day	1.98347 ac-ft	1 cfs for 1 day = approx. 2 ac-ft
ac-ft = acre-feet cfs = cubic feet per second cu ft = cubic feet ft = feet sq = square feet		

## SEDIMENT RELATED CONSTANTS AND CONVERSIONS

$$\text{mm} = 1 \text{ ft} \left( \frac{304.8 \text{ mm}}{1 \text{ ft}} \right)$$

$$\text{mm} = 1 \text{ in.} \left( \frac{25.38 \text{ mm}}{1 \text{ in.}} \right)$$

$$\text{Unit Weight of Water } (\gamma_w) = 62.4 \text{ lbs/ft}^3$$

$$\text{Density of Water } (\rho_w) = 1.94 \text{ slugs/ft}^3$$

$$\text{Specific Gravity of Sediment } (S_G) \approx 2.65 \text{ (Quartz)}$$

$$\text{Sediment Porosity (in-situ) } (\eta) \approx 0.4 \text{ at arroyo bed}$$

$$\text{Solid Unit Weight of Sediment } (\gamma_s) = S_G \gamma_w = 62.4 S_G \approx 165.36 \text{ lb/ft}^3 \text{ (Quartz)} = 2.65 \gamma_w$$

$$\text{Bulked Unit Weight of Sediment (dry)} = (1 - \eta) \gamma_s \approx 100 \text{ lb/ft}^3 \text{ } (\eta = 0.395)$$

$$\text{Sediment Yield: } 1 \text{ ac-ft/mi}^2 \text{ (bulk)} \approx 3.4 \text{ tons/ac } (\eta = 0.395)$$

Sediment Transport:

$$1 \text{ cfs} \approx 43.2 \gamma_s \text{ tons/day}$$

$$\text{Concentration by Volume } (C_v) = \left( \frac{V_s}{V_w + V_s} \right) * 10^6$$

$$\text{Concentration by weight } (C_w) = \left( \frac{W_s}{W_w + W_s} \right) * 10^6 = \left( \frac{S_g V_s}{V_w + S_g V_s} \right) * 10^6$$

$$C_v = \frac{C_w}{S_g - C_w(S_g - 1)} \text{ or } C_w = \frac{S_g C_v}{1 + C_v(S_g - 1)}$$

$$\text{Bulking Factor (BF)} = \frac{V_w + V_s}{V_w} = \frac{1}{(1 - C_v)}$$

$$\text{Fluid density } (\rho_m) = \rho_0 [1 + (S_g - 1)C_v]$$

where:

$V_s$  = volume of sediment

$V_w$  = volume of water

$W_w$  = weight of water

$W_s$  = weight of sediment

all other terms previously defined

The source of the sediment related constants and conversions is the following document:

AMAFCA, November 1994, "Sediment and Erosion Design Guide",

[http://www.bernco.gov/uploads/FileLinks/cbb8af72471e4746ba9e92e9a67318bb/Sediment\\_and\\_Erosion\\_Design\\_Guide\\_AMAFCA.pdf](http://www.bernco.gov/uploads/FileLinks/cbb8af72471e4746ba9e92e9a67318bb/Sediment_and_Erosion_Design_Guide_AMAFCA.pdf)

# **APPENDIX 3 CHECKLISTS FOR FIELD TRIPS, DRAINAGE REPORTS AND SURVEYS**

## Field Trip Preparation Checklist

**Date**

**Project Name:**

**Route Number:** , **Beg. Mile Post:** , **End Mile Post:** , **Bridge No. (s):**

**Project Engineer:**

**Watercourse Name**

ITEM	Yes/No /NA	Comments
Topographic map (delineate basin prior to field visit)		
Google Earth satellite photographs		
Estimate peak flow rates using StreamStats or similar method		
Record Drawings (As-built plans)		
Obtain patrol foreman/bridge inspection records (review prior to field visit)		
Email Address		
Name and telephone numbers of patrol foreman to coordinate and meet at the site		
Name and telephone numbers of local individuals to coordinate and meet at the site		
Camera		
Shovel and ax		
Sediment sample bags (need strong bags) and box to hold sediment samples		
Magic markers, pencils, pens, note pad, clip board, tape measure		
Safety vest, hard hat, identification		

### Field Trip Observations and Measurements Checklist

**Date:**

**Project Name:**

**Route Number:** , **Beg. Mile Post:** , **End Mile Post:** , **Bridge No.(s):**

**Project Engineer:**

**Watercourse Name**

ITEM	Yes/No /NA	Comments
<b><i>Patrol Foreman Interview questions:</i></b>		
What is your association or knowledge of the area and in particular this watercourse, and structure and drainage history, any photographs, or data recorded?		Name Telephone No.
<b><i>Local Individual Interview questions:</i></b>		
What is your association or knowledge of the area and in particular this watercourse, and structure and drainage history, any photographs, or data recorded?		Name Telephone No.
<b>Observations, Measurements, Notes</b>		
Take digital photographs of all observed items listed here (annotate later)		
Prepare a Sketch to record measurements/dimensions:		
No. of Bridge openings or No. of Culverts		
Diameters, dimensions, widths, thickness, etc.		
Depth(s) from bridge low chord to bed across watercourse		
Bridge deck thickness		
Bridge span / distance between piers		
Pier width, debris width and height and location		
Maximum available headwater depth		
Sediment depths in culverts		
Scour depth at culvert outlets		
Scour depth at bridge piers		

Note culvert or bridge condition and material type (RCP, CMP, etc.)		
Document locations and sizes of bridge deck drains		
Document and measure any rundowns, pipes etc. adjacent to bridge and erosion if any		
Document condition/issues of roadside ditches and rundowns		
Document existing condition, undermining if any, of wire-enclosed riprap, gabions, etc.		
Estimate average flow depth and discharge - if a perennial stream		
Estimate location of the ordinary high water mark, measure depth of low flow banks, etc.		
Document existing condition, undermining if any, of wire-enclosed rip-rap, gabions, etc.		
Use a map and/or at least prepare a sketch annotated with dimension, etc. to document the following items:		
observe, estimate and document the watershed rangeland condition (good, fair, poor) for the watershed that may be visible		
estimate the Runoff Curve Numbers (CNs) (refer to NMDOT Drainage Design Manual Appendix 2)		
has any portion of the watershed recently burned or been graded		
look for and record/photograph high watermark elevations and flood debris in overbanks		
observe water course for about 2,000 feet each direction - upstream and downstream of highway crossing and document/estimate the water course characteristics in reaches with respect to the Manning's Roughness Coefficient "n" and sediment transport regime (bed aggradation/degradation, or approximate equilibrium)		
for watercourse by designated reaches, estimate Manning's Roughness Coefficients "n" for bed, banks and overbanks (and document with photographs and on map)		
note and document naturally occurring or man-made bed armor locations		



note and document any natural or man-made grade control locations or structures		
observe and document banks for heights and stability		
obtain and document the location of sediment samples collected for gradation analyses		
estimate extent of bed that may be covered with stones, record stone size fractions and take corresponding stone diameter measurements to assist in critical shear analyses		
obtain depth to groundwater by interview or estimate by observing vegetation, particularly in valley locations (determine if shallow or too deep to be important)		
observe paleo evidence if any, of previous large flows		
<b>NOTES - OTHER ITEMS</b>		

Drainage Report Preparation Checklist		
<b>Date:</b>		
<b>Project Name:</b>		
<b>Route Number:</b> , <b>Beg. Mile Post:</b> , <b>End Mile Post:</b> , <b>Bridge No. (s):</b>		
<b>Project Engineer:</b>		<b>Watercourse Name</b>
<b>ITEM</b>	<b>Yes/No/ NA</b>	<b>Comments</b>
<b>Drainage Report Cover</b>		
1. Project Number:		
2. Project Control Number:		
3. Date:		
4. Route Number:		
5. Begin Mile Post:		
6. End Post:		
7. Bridge Number:		
8. Document Type: example - Final Drainage Report		
9. Document Description:		
<b>Executive Summary</b>		
<b>1. General Project Information</b>		
1.1 Description and Purpose		
1.2 Field Observation and Survey		
1.3 As-Builts and Plans		
1.4 Highway Classification and Average Daily Traffic (ADT)		
1.5 Drainage Design Criteria		
1.6 FEMA Floodplains		
1.7 MS4 Requirements		

<b>2. Hydrologic Analyses</b>		
2.1 Drainage Basin Delineation		
2.2 Existing Flood Control and Drainage Structures		
2.3 Storm Drain System Description (Existing and/or Proposed)		
2.4 Basin Condition - Fire Issues, Debris and Range Conditions		
2.5 Hydrologic Analyses Results from Previous Studies		
2.6 Drainage Criteria		
2.7 Hydrologic Analyses Methods		
2.8 Rainfall Data and Distributions		
2.9 Soils Data and Runoff Curve Numbers, Runoff Coefficients		
2.10 Time of Concentration and Lag Time Computations		
2.11 Channel Routing		
2.12 Pond or Reservoir Routing		
2.13 Fire, Debris and Sediment Bulking		
2.14 Diversions and Hydrograph Divides		
2.15 Computation Time Increment for Hydrologic Model		
2.16 HEC-HMS or Hydrologic Model Results		
2.17 Peak Discharge Computations (if no computer model)		
2.18 USGS Regression Equations		
2.19 USGS Gage Flood Frequency Regression Analysis		
2.20 Peak Discharge Comparisons		
2.21 Bridge Deck Hydrologic/Hydraulic Computations		
2.22 Roadside/Median Ditch Hydrologic/Hydraulic Computations		
<b>3. Stormwater Quality, Low Impact Development and Best Management Practices</b>		
3.1 The Impact of ADT on Stormwater Quality		
3.2 Nearby and Potentially Affected Wetlands and Sensitive or Important Habitat		
3.3 Waters of the U.S. Concerns		
3.4 Principles of Road Corridor Erosion and Remedies		
3.5 Choosing the "Best Management Practices"		
3.6 The Effects of Microclimate in New Mexico Highways on Water Quality		

<b>4. Storm Drain System Hydraulic Analyses</b>		
4.1 Highway/Street System and Storm Drain System Description		
4.2 Street Capacities and Storm Drain Inlet Capacities		
4.3 Storm Drain Hydraulic Data, Assumptions, Analyses and Results		
4.4 Storm Drain Hydraulic Model Results		
4.5 Street and Storm Drain Results - Flow Spread and Depth		
4.6 Culvert Analyses and Results		
4.7 Open Channel Flow for Small – Local Channels		
<b>5. Watercourse Hydraulic Analyses, Scour and Sediment Transport</b>		
5.1 Watercourse Description and Data		
5.2 Fluvial Geomorphology Considerations		
5.3 Watercourse Steady Flow Hydraulic Model - HEC-RAS Hydraulic Model Data, Assumptions and Results		
5.4 Watercourse Two-Dimensional Flow Hydraulic Model - Model Data, Assumptions and Results		
5.5 Bridge Scour Measurements, Computations and Results		
5.6 Watercourse Quasi-Steady Flow Hydraulic Model – Sediment Transport Analyses HEC-RAS Model Data, Assumptions and Results		
5.7 Summary of Hydraulic and Sediment Transport Results		
5.8 Culvert Analyses and Results		
<b>6. Recommendations and Design for Storm Drain System</b>		
6.1 Storm Drain Recommendations		
6.2 Culvert Recommendations		
6.3 Small Open Channel Recommendations		
6.4 Large Open Channel Recommendations		
6.5 Bridge Recommendations		

<b>7. Recommendations and Design For Energy Dissipation, Erosion and Grade Control Measures</b>		
7.1 Cut and Fill Slopes		
7.2 Stabilization Measures on Slopes to Reduce Erosion		
7.3 Roadside Ditch Erosion Measures		
7.4 Riprap Design, Culvert Outfall Design, Gabion Design		
<b>8. Recommendations and Design for Bridge Scour Countermeasures</b>		
8.1 Abutment, Pier and Spread Footing Recommendations		
<b>9. Stormwater Quality Design, Construction and Maintenance Considerations</b>		
9.1 Receiving Water Considerations		
9.2 First Flush		
9.3 Rural Roadways		
9.3.1 The Effects of Roadway Drainage Micro-climate		
9.4 Urban Streets with and without Storm Drains		
9.5 Maintenance Considerations		
<b>10. References</b>		
<b>Figures and Maps</b>		
Project Vicinity Map		
Drainage Basin Map(s)		
Soils Map(s)		
Storm Drain System Map(s) and Profile(s)		
Bridge No. XX Topographic and HEC-RAS Analysis Map		
Bed Profile(s) - Existing and Proposed with Erosion and Scour		
Erosion and Scour Countermeasures		
Drainage Structure(s) - Plan(s) and Profile(s)		
Drainage Structure(s) - Detail(s)		
Best Management Practices		

<b>Tables</b>		
Drainage Criteria		
Rainfall Data		
Runoff Curve Number (CN) Assumptions, Data and Calculations		
Basin Hydrologic Data and Time of Concentration Method - Kirpich Method, Upland Method, Stream Hydraulic Method Data and Computations		
Channel Routing Data		
Pond Routing Data		
Hydrograph Divide Data		
USGS Regresssion Equations and Results		
Bridge Deck Hydrologic/Hydraulic Analysis Results Summary		
Roadside/Median Ditch Hydrologic/Hydraulic Analysis Results Summary		
Peak Discharge and Runoff Volume Results Summary		
Street and Storm Drain Hydraulic Results Summary		
Culvert Hydraulic Analysis Results Summary		
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Critical Shear and Incipient Motion Analysis Results Summary		
Scour Analysis Results Summary		
Sediment Transport Analysis Results Summary		
MS4 Analysis and Results Summary		
<b>Appendix 1 - Annotated Photographs</b>		
<b>Appendix 2 - Record Drawings (highways, bridges, culverts), Bridge Inspection Reports, Patrol Foreman Records</b>		

<b>Appendix 3 - Hydrologic Data, Calculations and References</b>		
Data and Calculations		
1. Select Pages from the NMDOT Drainage Design Manual		
2. NOAA Atlas 14 Point Precipitation Frequency Estimates (printed from NOAA Atlas 14 internet site)		
3. Figure 14, Depth-Area Curves (Source: NOAA Atlas 2 Vol. IV, 1973)		
4. Urban Hydrology for Small Watersheds, US Dept. of Agricultural Soil Conservation Service, Technical Release 55, June 1986. * Figure B-2, Approximate Geographic Boundaries for SCS Rainfall Distributions * Table 2-2a. Runoff Curve Numbers for Urban Areas. * Table 2-2d. Runoff Curve Numbers for Arid and Semiarid Rangelands.		
5. Soils Data Summary (Tables and Figures) * Table – Soil Map Unit Descriptions and Hydrologic Soil Groups * Soil Survey of XX * NRCS Internet Soils Data Program Output		
6. Sediment Bulking Factors were obtained (list source)		
7. Analysis of the Magnitude and Frequency of Peak Discharge and Maximum Observed Peak Discharge in New Mexico and Surrounding Areas. USGS Scientific Investigations Report 2008-5119 (or latest document)		
8. USGS Stream Flow Gaging Station (USGS XX) and Map of Location, Peak Streamflow, Mean Monthly Discharge, Annual Statistics		
9. Program PeakFq (latest version) Output - USGS Flood Frequency Regression Program – Annual Peak Flow Frequency Analysis for "Station XXX"		
10. Table - Time Increment Computation and Selected Pages from the HEC-HMS Reference Manual		
11. Table - Equilibrium Slope Computation(s), (List Source)		
<b>Appendix 4 - HEC-HMS Models or Other Models and Hydrologic Summary Output</b>		
HEC-HMS Models and Hydrologic Summary Output		

<b>Appendix 5 - MS4, Storm Water Quality, Low Impact Development and Best Management Practices</b>		
Present Computations, Sources for LID and BMP Selection, List Important Websites		
<b>Appendix 6 - Hydraulic Data, Calculations, FlowMaster Models, or Other Models and Hydraulic Summary Output</b>		
Hydraulic Data and Calculations		
Open Channel Data, Roadway Section Data, Inlet Types		<input type="checkbox"/>
FlowMaster Open Channel Flow Models and Hydraulic Summary Output		
<b>Appendix 7 - StormCAD Storm Drain Models, or Other Models, and Hydraulic Summary Output</b>		
Storm Drain Data		
StormCAD Models or Other Models and Hydraulic Summary Output		
<b>Appendix 8 - CulvertMaster Models or HY-8 Culvert Models and Hydraulic Summary Output</b>		
Culvert Data <input type="checkbox"/>		
CulvertMaster Models or HY-8 Models and Hydraulic Summary Output		
<b>Appendix 9 - HEC-RAS Steady Flow Models, or Other Models and Hydraulic Summary Output</b>		
Figure - Watercourse Data, Soil Sample Locations - Reach Map		
Table - Watercourse Reach Data (Manning's "n" bed armoring, etc.)		
Manning's "n" References:	*	
NMDOT Drainage Design Manual	*	
Other Source		
Table - Slope Computations - HEC-RAS Model Starting Water Surface Data		
HEC-RAS Steady Flow Models or Other Models and Hydraulic Summary Output		



<b>Appendix 10 - HEC-RAS Quasi - Steady Flow Models, Hydraulic Summary Output</b>		
Sediment Gradation Data, Bed Armoring Limits and Erosion Assumption Limits		
HEC-RAS Quasi - Steady Flow Models and Model Output		
SMS-SRH2D Models and Hydraulic Summary Output		
<b>Appendix 11 - Two-Dimensional - HEC-RAS Flow Models, or SMS-SRH2D Flow Models, and Hydraulic Summary Output</b>		
HEC-RAS Two-Dimensional Flow Models and Hydraulic Summary Output		
SMS-SRH2D Models and Hydraulic Summary Output		
<b>Appendix 12 - Scour and Bridge Elevation Computations</b>		
Table CS - Critical Shear and Incipient Motion Computations		
Table TS - Total Scour Depths 100-yr. and 500-yr. for Bridge #		
Table DS - Determination of Either Clear Water or Live Bed Contraction Scour Bridge #		
Table CW - Clear-Water Contraction Scour Bridge #		
Table PS - Pier Scour Bridge #		
Table AS - Abutment Scour Bridge #		
Table BB - Bridge to Bed Elevation Calculations		
<b>Appendix 13 - Project Digital Archive Provided to NMDOT Drainage Design Bureau</b>		
Contains PDFs of entire Drainage Report/Appendices and Digital Files of the Hydrologic and Hydraulic Computer Models		

**DRAINAGE REPORT TABLE OF CONTENTS TEMPLATE**

This document presents a fairly comprehensive drainage report template or outline that should be generally followed during preparation of NMDOT Drainage Reports. Depending on the scope of each particular drainage project, some Sections and/or Sub-Sections and/or Appendices may not be required.

**DRAINAGE REPORT  
COVER**Items Required on the Cover Include:

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

**Certification Page****Certification and Professional Engineer Signature and Seal****DRAINAGE REPORT  
TABLE OF CONTENTS****SECTION****EXECUTIVE SUMMARY****1. GENERAL PROJECT INFORMATION****1.1 Description and Purpose****1.2 Field Observation and Survey****1.3 As-Builts and Plans****1.4 Highway Classification and Average Daily Traffic (ADT)****1.5 Drainage Criteria****1.6 FEMA Floodplains****1.7 MS4 Requirements**

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- 2.1 Drainage Basin Delineation
- 2.2 Existing Flood Control and Drainage Structures
- 2.3 Storm Drain System Description (Existing and/or Proposed)
- 2.4 Basin Condition – Fire Issues, Debris and Range Conditions
- 2.5 Hydrologic Analyses Results from Previous Studies
- 2.6 Drainage Criteria
- 2.7 Hydrologic Analyses Methods
- 2.8 Rainfall Data and Distributions
- 2.9 Soils Data and Runoff Curve Numbers, Runoff Coefficients
- 2.10 Time of Concentration and Lag Time Computations
- 2.11 Channel Routing
- 2.12 Pond or Reservoir Routing
- 2.13 Fire, Debris and Sediment Bulking
- 2.14 Diversions and Hydrograph Divides
- 2.15 Computation Time Increment for Hydrologic Model
- 2.16 HEC-HMS or Hydrologic Model Results
- 2.17 Peak Discharge Computations (if no computer model)
- 2.18 USGS Regression Equations
- 2.19 USGS Gage Flood Frequency Regression Analysis
- 2.20 Peak Discharge Comparisons
- 2.21 Bridge Deck Hydrologic/Hydraulic Computations
- 2.22 Roadside/Median Ditch Hydrologic/Hydraulic Computations

## 3. STORMWATER QUALITY, LOW IMPACT DEVELOPMENT and BEST MANAGEMENT PRACTICES

- 3.1 The Impact of ADT on Stormwater Quality
- 3.2 Nearby and Potentially Affected Wetlands and Sensitive or Important Habitat
- 3.3 Waters of the U.S. Concerns
- 3.4 Principles of Road Corridor Erosion and Remedies
- 3.5 Choosing the “Best Management Practices”
- 3.6 The Effects of Microclimate in New Mexico Highways on Water Quality

#### 4. STORM DRAIN SYSTEM HYDRAULIC ANALYSES

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- 4.2 Street Capacities and Storm Drain Inlet Capacities
- 4.3 Storm Drain Hydraulic Data, Assumptions, Analyses and Results
- 4.4 Storm Drain Hydraulic Model Results
- 4.5 Street and Storm Drain Results - Flow Spread and Depth
- 4.6 Culvert Analyses and Results
- 4.7 Open Channel Flow for Small – Local Channels

#### 5. WATERCOURSE HYDRAULIC ANALYSES, SCOUR AND SEDIMENT TRANSPORT

- 5.1 Watercourse Description and Data
- 5.2 Fluvial Geomorphology Considerations
- 5.3 Watercourse Steady Flow Hydraulic Model –  
HEC-RAS Hydraulic Model Data, Assumptions and Results
- 5.4 Watercourse Two-Dimensional Flow Hydraulic Model – Model Data, Assumptions and Results
- 5.5 Bridge Scour Measurements, Computations and Results
- 5.6 Watercourse Quasi-Steady Flow Hydraulic Model – Sediment Transport Analyses HEC-RAS Model Data, Assumptions and Results
- 5.7 Summary of Hydraulic and Sediment Transport Results
- 5.8 Culvert Analyses and Results

#### 6. RECOMMENDATIONS and DESIGN FOR STORM DRAIN SYSTEM

- 6.1 Storm Drain Recommendations
- 6.2 Culvert Recommendations
- 6.3 Small Open Channel Recommendations
- 6.4 Large Open Channel Recommendations
- 6.5 Bridge Recommendations

#### 7. RECOMMENDATIONS and DESIGN FOR ENERGY DISSIPATION, EROSION AND GRADE CONTROL MEASURES

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- 7.2 Stabilization Measures on Slopes to Reduce Erosion
- 7.3 Roadside Ditch Erosion Measures

## 7.4 Riprap Design, Culvert Outfall Design, Gabion Design

## 8. RECOMMENDATIONS and DESIGN FOR BRIDGE SCOUR COUNTERMEASURES

### 8.1 Abutment, Pier and Footing Recommendations

## 9. STORMWATER QUALITY DESIGN, CONSTRUCTION AND MAINTENANCE CONSIDERATIONS .

### 9.1 Receiving Waters Considerations

### 9.2 First Flush

### 9.3 Rural Roadways

#### 9.3.1 The Effects of Roadway Drainage Micro-climate

### 9.4 Urban Streets with and without Storm Drains

### 9.5 Maintenance Considerations

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Figure 2	Drainage Basin Map(s)
Figure 3	Soils Map(s)
Figure 4	Storm Drain System Map(s) and Profile(s)
Figure 5	Bridge No. XX Topographic and HEC-RAS Analysis Map
Figure 6	Bed Profile(s) - Existing and Proposed with Erosion and Scour
Figure 7	Erosion and Scour Countermeasures
Figure 8	Drainage Structure(s) - Plan(s) and Profile(s)
Figure 9	Drainage Structure(s) - Detail(s)
Figure 10	Best Management Practices

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Table 5	Channel Routing Data
Table 6	Pond Routing Data
Table 7	Hydrograph Divide Data
Table 8	USGS Regression Equations and Results
Table 9	Bridge Deck Hydrologic / Hydraulic Analysis Results Summary
Table 10	Roadside/Median Ditch Hydrologic / Hydraulic Analysis Results Summary
Table 11	Peak Discharge and Runoff Volume Results Summary
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Table 13	Culvert Hydraulic Analysis Results Summary
Table 14	Bridge Freeboard Summary
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APPENDIX 2 - RECORD DRAWINGS (HIGHWAYS, BRIDGES, CULVERTS), BRIDGE INSPECTION REPORTS, PATROL FORMAN RECORDS

APPENDIX 3 - HYDROLOGIC DATA, CALCUALTIONS AND REFERENCES

Data and Calculations

Selected pages from the NMDOT Drainage Design Manual

NOAA Atlas 14 Point Precipitation Frequency Estimates (printed from NOAA Atlas 14 internet site)

Figure 14, Depth-Area Curves (Source: NOAA Atlas 2 Vol. IV, 1973).

Urban Hydrology for Small Watersheds, US Dept of Agricultural Soil Conservation Service, Technical Release 55, June 1986.

Figure B-2, Approximate Geographic Boundaries for SCS Rainfall Distributions

Table 2-2a. Runoff Curve Numbers for Urban Areas.

Table 2-2d. Runoff Curve Numbers for Arid and Semiarid Rangelands.

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Table – Soil Map Unit Descriptions and Hydrologic Soil Groups

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USGS Stream Flow Gaging Station (USGS XX) and Map of Location, Peak Streamflow, Mean Monthly Discharge, Annual Statistics

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Table - Time Increment Computation and Selected Pages from the HEC-HMS Reference Manual

Table - Equilibrium Slope Computation(s), (list source)

APPENDIX 4 - HEC-HMS MODELS or OTHER MODELS and HYDROLOGIC SUMMARY OUTPUT

HEC-HMS Models and Hydrologic Summary Output

APPENDIX 5 - MS4, STORM WATER QUALITY, LOW IMPACT DEVELOPMENT and BEST MANAGEMENT PRACTICES

Present computations, sources for LID and BMP selection, list important websites

APPENDIX 6 - HYDRAULIC DATA, CALCULATIONS, FLOWMASTER MODELS, or OTHER MODELS and HYDRAULIC SUMMARY OUTPUT

Hydraulic Data and Calculations

Open Channel Data, Roadway Section Data, Inlet Types

FlowMaster Open Channel Flow Models and Hydraulic Summary Output

APPENDIX 7 - STORMCAD STORM DRAIN MODELS, or OTHER MODELS, and HYDRAULIC SUMMARY OUTPUT

Storm Drain Data

StormCAD Models or Other Models and Hydraulic Summary Output

APPENDIX 8 - HY-8 CULVERT MODELS or CULVERTMASTER MODELS and HYDRAULIC SUMMARY OUTPUT

Culvert Data

HY-8 Models or CulvertMaster Models and Hydraulic Summary Output

APPENDIX 9 - HEC-RAS STEADY FLOW MODELS, or OTHER MODELS and HYDRAULIC SUMMARY OUTPUT

Figure – Watercourse Data, Soil Sample Locations – Reach Map

Table – Watercourse Reach Data (Manning’s “n” bed armoring, etc.)

## Manning's "n" References:

NMDOT Drainage Design Manual

## Other Sources

Table – Slope Computations – HEC-RAS Model Starting Water Surface Data

HEC-RAS Steady Flow Models or Other Models and Hydraulic Summary Output

APPENDIX 10 - HEC-RAS QUASI - STEADY FLOW MODELS, or SMS-SRH2D MODELS and  
HYDRAULIC SUMMARY OUTPUT

Sediment Gradation Data, Bed Armoring Limits and Erosion Assumptions Limits

HEC-RAS Quasi – Steady Flow Models and Model Output

SMS-SRH2D Models and Hydraulic Summary Output

APPENDIX 11 - TWO-DIMENSIONAL – HEC-RAS FLOW MODELS, or SMS-SRH2D FLOW  
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HEC-RAS Two-Dimensional Flow Models and Hydraulic Summary Output

SMS-SRH2D Models and Hydraulic Summary Output

## APPENDIX 12 - SCOUR AND BRIDGE ELEVATION COMPUTATIONS

Table CS - Critical Shear and Incipient Motion Computations

Table TS - Total Scour Depths 100-yr and 500-yr for Bridge #

Table DS - Determination of Either Clear Water or Live Bed Contraction Scour Bridge #

Table CW - Clear-Water Contraction Scour Bridge #

Table PS - Pier Scour Bridge #

Table AS - Abutment Scour Bridge #

Table BB - Bridge to Bed Elevation Calculations

APPENDIX 13 - PROJECT DIGITAL ARCHIVE PROVIDED TO NMDOT DRAINAGE DESIGN  
BUREAUContains PDFs of entire Drainage Report/Appendices and Digital Files of all Hydrologic and  
Hydraulic Computer Models



### Survey Checklist

**Date:**

**Project Name:**

**Route Number:** , **Beg. Mile Post:** , **End Mile Post:** , **Bridge No.(s) :**

**Project Engineer:**

**Watercourse Name:**

ITEM	Yes/No /NA	Comments
<b>Watercourse with Existing and Proposed Bridges</b>		
Complete topographic survey (to high banks or high ground adjacent to watercourse) upstream and downstream of highway crossing for about 2000 feet if possible. However, that distance may be less depending on the watercourse characteristics and property/right-of-way access issues. Survey should identify water course low flow centerline or centerlines if multiple low flow paths. For each bank, survey the toe of low bank in the bed and the top of low flow bank. For overbank, survey the overbank limits if any, and top of a high bank or high ground if any. Survey all locations of significant watercourse character changes in plan (change from narrow to wide cross-section) and profile (visible bed slope changes). If only cross-sections are taken, survey sections approximately every 100 to 200 ft and/or depending on locations of watercourse character changes in plan (change from narrow to wide cross-section) and profile (visible bed slope changes).		
Survey all fence lines and utility locations adjacent to and pertinent to the project		
Elevations and locations of all main features		
Cross-sections about 50 ft from bridge deck and/or at the abutment transition section from natural watercourse to beginning of abutment transition (upstream and downstream)		
Cross-sections directly under the bridge deck (upstream and downstream)		
Spot elevations at top and toe of any abutment transitions and treatments (upstream and downstream) for the full extent of the treatment		
Spot elevations at all piers (upstream and downstream)		
Centerline or upstream edge of lane profile for 200 ft minimum each direction from edge of bridge deck		
Elevations of: bridge deck at all four corners, center line, top of barrier railings, bridge deck drains, rundowns (top and toe), any other drainage related features		

<b>Watercourse with Existing and Proposed Culverts</b>		
Complete topographic survey upstream and downstream of highway crossing for about <u>1000 feet</u> if possible. However, that distance may be less depending on the culvert size and the watercourse size and characteristics. The distance is also dependent on property/right-of-way access issues. Survey should identify water course low flow centerline. For each bank, survey the toe of low bank in the bed and the top of low flow bank. For overbank, survey the overbank limits if any, and top of a high bank or high ground if any. Survey all locations of significant watercourse character changes in plan (change from narrow to wide cross-section) and profile (visible bed slope changes). If only cross-sections are taken, survey sections approximately every 100 to 200 ft and/or depending on locations of watercourse character changes in plan (change from narrow to wide cross-section) and profile (visible bed slope changes).		
Survey all fence lines and utility locations adjacent to and pertinent to the project.		
Elevations and locations of all main features:		
Cross-sections about 30 to 50 ft from edge of pavement and/or at the embankment transition section from natural watercourse at beginning of a wing wall or grading transition to culvert inlet and outlet (upstream and downstream)		
Edge of pavement and edge of lane (upstream and downstream)		
Number and size dimensions of box culverts and pipe diameter		
Invert and soffit elevations (upstream and downstream) to allow computation of sediment depth if any		
Cross-sections directly under the bridge deck (upstream and downstream)		
Spot elevations at top and toe of any wing walls or grading transition or abutment type transitions and treatments (upstream and downstream) for the full extent of the treatment		
Centerline or upstream edge of lane profile for 200 ft minimum each direction from edge of culvert		
Elevations of: roadway at all four corners above culvert(s), center line, top of barrier railings, rundowns (top and toe), any other drainage related features		

<b>Urban Areas</b>		
Complete topographic survey of proposed roadway/vicinity		
Survey all fence lines and utility locations adjacent to and pertinent to the project		
Elevations and locations of all main features		
Edge of pavement and edge of lane, centerline and median elevations		
Top and toe of wall barriers		
Top of curbs		
Gutters		
Inlet grate elevations and drop inlet basin invert elevations		
Manhole rim and invert elevations		
Storm drain invert elevations at outfalls (record material type)		
<b>Detention or Retention Ponds</b>		
Complete topographic survey of pond bottom, embankment crest elevations (upstream and downstream), toe of embankment elevations inside and outside of pond		
Watercourse inlet and outlet survey for about 100 ft upstream of pond and 200 ft or more downstream of either the toe of embankment, the principal or emergency spillway outfall locations, whichever of these is furthest downstream in the watercourse		

[illegible]

# **APPENDIX 4 PHOTOGRAPHIC GUIDE TO HYDROLOGIC CONDITIONS**



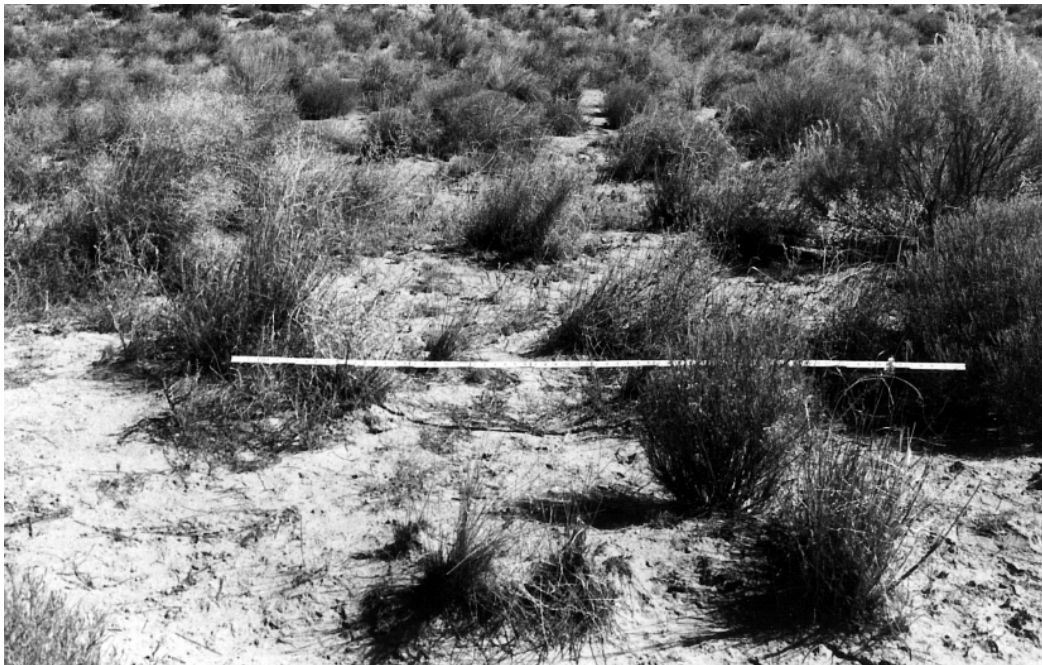
Photo No.:	<b>1A, General View</b>	Photo Date: <b>August, 1995</b>
Location:	<b>Near Santa Fe, Santa Fe County</b>	
Aspect:	<b>South Facing</b>	
Cover Description:	<b>Piñon and Juniper with Bunch Grass</b>	
Hydrologic Condition:	<b>Poor</b>	HSG: <b>B</b>
Soil Series:	<b>Pojoaque</b>	Percent Cover: <b>30%</b>
Estimated Runoff Curve Number:	<b>75</b>	Soil Description: <b>Gravelly Sand</b>



**Photo No. 1B, Ground View**



Photo No.:	<b>2A, General View</b>	Photo Date: <b>June, 1995</b>
Location:	<b>Northwest Albuquerque, Bernalillo County</b>	
Aspect:	<b>East Facing</b>	
Cover Description:	<b>Sagebrush, Saltbush, Weeds, Bunch Grasses</b>	
Hydrologic Condition:	<b>Fair</b>	HSG: <b>A</b>
Soil Series:	<b>Bluepoint</b>	Percent Cover: <b>40%</b>
Estimated Runoff Curve Number:	<b>65</b>	Soil Description: <b>Sandy</b>



**Photo No. 2B, Ground View**





Photo No.	<b>3A, General View</b>	Photo Date: <b>June, 1995</b>
Location:	<b>Near Hondo, Lincoln County</b>	
Aspect:	<b>South Facing</b>	
Cover Description:	<b>Juniper and Bunch Grasses</b>	
Hydrologic Condition:	<b>Fair</b>	HSG: <b>D</b>
Soil Series:	<b>Deama</b>	Percent Cover: <b>40%</b>
Estimated Runoff Curve Number:	<b>80</b>	Soil Description: <b>Rocky</b>



**Photo No. 3B, Ground View**





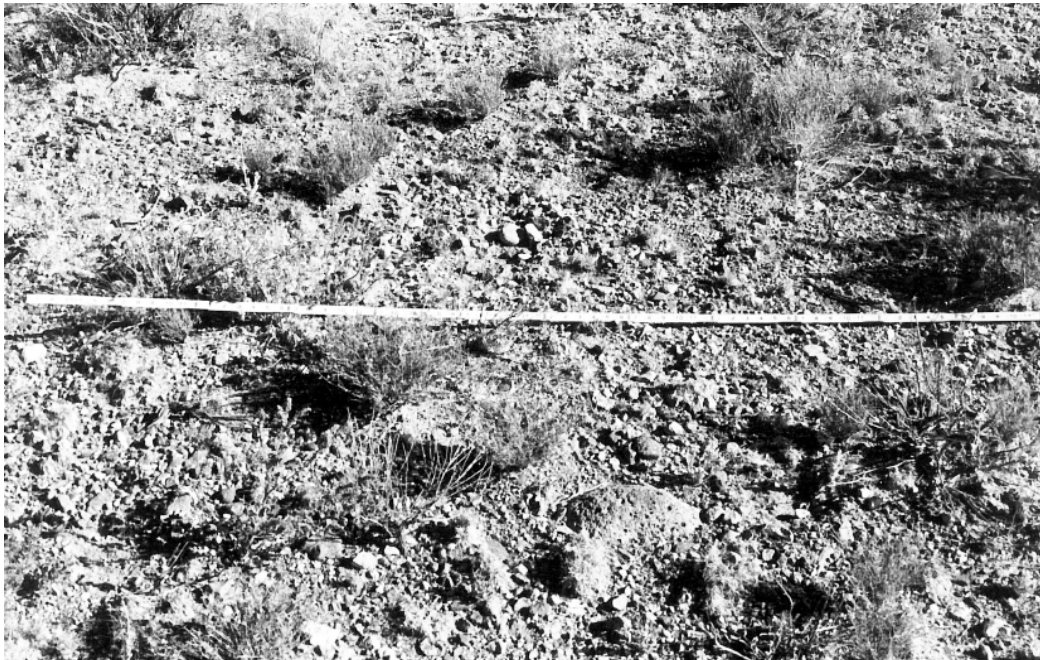
Photo No.	<b>4A, General View</b>	Photo Date: <b>June, 1995</b>
Location:	<b>Near Radium Springs, Doña Ana County</b>	
Aspect:	<b>Gently Sloping West</b>	
Cover Description:	<b>Desert Brush, Yucca, Cactus, Bunch Grasses</b>	
Hydrologic Condition:	<b>Fair</b>	<b>HSG: A</b>
Soil Series:	<b>Bluepoint</b>	Percent Cover: <b>30%</b>
Estimated Runoff Curve Number:	<b>60</b>	Soil Description: <b>Sandy Loam</b>



**Photo No. 4B, Ground View**



Photo No.	<b>5A, General View</b>	Photo Date: <b>June, 1995</b>
Location:	<b>West of San Marcial, Socorro County</b>	
Aspect:	<b>Southeast Sloping</b>	
Cover Description:	<b>Desert Shrub, Mesquite</b>	
Hydrologic Condition:	<b>Poor</b>	HSG: <b>C</b>
Soil Series:	<b>Nickel</b>	Percent Cover: <b>10%</b>
Estimated Runoff Curve Number:	<b>85</b>	Soil Description: <b>Gravelly</b>



**Photo No. 5B, Ground View**



Photo No.	<b>6A, General View</b>	Photo Date: <b>June, 1995</b>
Location:	<b>West Alamogordo, Otero County</b>	
Aspect:	<b>West Sloping</b>	
Cover Description:	<b>Desert Brush, Cactus, Bunch Grasses</b>	
Hydrologic Condition:	<b>Fair</b>	HSG: <b>B</b>
Soil Series:	<b>Nickel</b>	Percent Cover: <b>40%</b>
Estimated Runoff Curve Number:	<b>74</b>	Soil Description: <b>Sand and Gravel</b>



**Photo No. 6B, Ground View**





Photo No.	<b>7A, General View</b>	Photo Date: <b>August, 1995</b>
Location:	<b>South of Taos, Taos County</b>	
Aspect:	<b>West Sloping</b>	
Cover Description:	<b>Sagebrush and Piñon</b>	
Hydrologic Condition:	<b>Fair</b>	HSG: <b>C</b>
Soil Series:	<b>Not available</b>	Percent Cover: <b>50%</b>
Estimated Runoff Curve Number:	<b>67</b>	Soil Description: <b>Sandy Clay</b>



**Photo No. 7B, Ground View**



Photo No.	<b>8A, General View</b>	Photo Date: <b>August, 1995</b>
Location:	<b>North of Taos, Taos County</b>	
Aspect:	<b>Level</b>	
Cover Description:	<b>Sagebrush and Bunch Grasses</b>	
Hydrologic Condition:	<b>Fair</b>	HSG: <b>C</b>
Soil Series:	<b>Not available</b>	Percent Cover: <b>60%</b>
Estimated Runoff Curve Number:	<b>63</b>	Soil Description: <b>Silty Clay</b>



**Photo No. 8B, Ground View**



Photo No.	<b>9A, General View</b>	Photo Date: <b>August, 1995</b>
Location:	<b>South of Espanola, Rio Arriba County</b>	
Aspect:	<b>West Facing</b>	
Cover Description:	<b>Rangeland</b>	
Hydrologic Condition:	<b>Poor</b>	HSG: <b>A</b>
Soil Series:	<b>Not available</b>	Percent Cover: <b>20%</b>
Estimated Runoff Curve Number:	<b>78</b>	Soil Description: <b>Silty Sand</b>



**Photo No. 9B, Ground View**



Photo No.	<b>10A, General View</b>	Photo Date: <b>August, 1995</b>
Location:	<b>North Central Mountains Near Taos, Taos County</b>	
Aspect:	<b>South Sloping</b>	
Cover Description:	<b>Pine Forest</b>	
Hydrologic Condition:	<b>Good</b>	HSG: <b>C</b>
Soil Series:	<b>Not mapped</b>	Percent Cover: <b>70%</b>
Estimated Runoff Curve Number:	<b>70</b>	Soil Description: <b>Thin Gravelly Topsoil</b>



**Photo No. 10B, Ground View**





Photo No. **11A, General View** Photo Date: **June, 1995**  
Location: **Near Cloudcroft, Otero County**  
Aspect: **West Facing**  
Cover Description: **Pine Forest with Partial Brush Understory**  
Hydrologic Condition: **Good** HSG: **D**  
Soil Series: **Not mapped** Percent Cover: **80%**  
Estimated Runoff Curve Number: **77** Soil Description: **Rocky**



Photo No. **11B, Ground Cover**





Photo No.	<b>12</b>	Photo Date:	<b>June, 1995</b>
Location:	<b>North of Ragland, Quay County</b>		
Aspect:	<b>West Facing</b>		
Cover Description:	<b>Juniper, Shrubs and Grasses, Rangeland</b>		
Hydrologic Condition:	<b>Fair</b>	HSG:	<b>B</b>
Soil Series:	<b>Quay Loam</b>	Percent Cover:	<b>60%</b>
Estimated Runoff Curve Number:	<b>72</b>	Soil Description:	<b>Silty Loam</b>



Photo No.	<b>13</b>	Photo Date:	<b>June, 1995</b>
Location:	<b>Near Grady, Curry County</b>		
Aspect:	<b>Level</b>		
Cover Description:	<b>Grassed Rangeland, Heavily Grazed</b>		
Hydrologic Condition:	<b>Fair</b>	HSG:	<b>C</b>
Soil Series:	<b>Potter</b>	Percent Cover:	<b>75%</b>
Estimated Runoff Curve Number:	<b>79</b>	Soil Description:	<b>Clay Loam</b>



Photo No.	<b>14</b>	Photo Date:	<b>June, 1995</b>
Location:	<b>South of Broadview, Curry County</b>		
Aspect:	<b>Nearly Level</b>		
Cover Description:	<b>Mixed Prairie Grasses, Some Weeds</b>		
Hydrologic Condition:	<b>Good</b>	HSG:	<b>C</b>
Soil Series:	<b>Potter</b>	Percent Cover:	<b>100%</b>
Estimated Runoff Curve Number:	<b>71</b>	Soil Description:	<b>Clay Loam</b>



Photo No.	<b>15</b>	Photo Date:	<b>June, 1995</b>
Location:	<b>North of Clovis, Curry County</b>		
Aspect:	<b>Nearly Level</b>		
Cover Description:	<b>Dense Prairie Grass</b>		
Hydrologic Condition:	<b>Good</b>	HSG:	<b>C</b>
Soil Series:	<b>Potter</b>	Percent Cover:	<b>75%</b>
Estimated Runoff Curve Number:	<b>66</b>	Soil Description:	<b>Clay Loam</b>



Photo No. **16** Photo Date: **June, 1995**  
Location: **North of Clovis, Clovis County**  
Aspect: **Nearly Level**  
Cover Description: **Agricultural, Tilled, Bare Soil**  
Hydrologic Condition: **Poor** HSG: **B** Percent Cover: **0%**  
Soil Series: **Amarillo** Soil Description: **Clayey Loam**  
Estimated Runoff Curve Number: **85**



Photo No. **17** Photo Date: **June, 1995**  
Location: **Near Doña Ana, Doña Ana County**  
Aspect: **Nearly Level**  
Cover Description: **Row Crops, Young Chili**  
Hydrologic Condition: **Poor** HSG: **B**  
Soil Series: **Harkey** Percent Cover: **10%**  
Estimated Runoff Curve Number: **79** Soil Description: **Silty Loam**



Photo No.	<b>18</b>	Photo Date:	<b>June, 1995</b>
Location:	<b>North of Clovis, Clovis County</b>		
Aspect:	<b>Nearly Level</b>		
Cover Description:	<b>Row Crop, Young Corn</b>		
Hydrologic Condition:	<b>Fair</b>	HSG:	<b>B</b>
Soil Series:	<b>Amarillo</b>	Percent Cover:	<b>30%</b>
Estimated Runoff Curve Number:	<b>78</b>	Soil Description:	<b>Clayey Loam</b>

# **APPENDIX 5 NRCS SIMPLIFIED PEAK DISCHARGE METHOD**

**PREFACE** - This Appendix is a typed version of the Soil Conservation Service (SCS) Chapter 2 Estimating Runoff (2/85) document. The (2/85) document is not available on the internet and is hard to find as a paper copy. The Figures, Exhibits and Tables referenced are not included here as some of them are outdated, and most have been updated by more recent NRCS hydrology documents that include these methods. The relevant updated Figures and Tables from the (2/85) document are included in **Section 404** of this Drainage Design Manual. In April 2014, Supplemental Notice No. NM-36 was developed as a modification to the 1985 document. NM-36 only prescribed to replace the previous rainfall data (1985 document) with NOAA Atlas 14 rainfall data.

Figure 2-4 "Unit Peak Discharge" from the 1985 document referenced in this Appendix, was presented as a log-log graph, and was developed to compute the unit peak discharge ( $q_u$ ) in cfs/ac-in. as a function of Time of Concentration ( $T_c$ ). **Figure 404-1** in **Section 400** of this Drainage Design Manual was developed from the log-log graph as a Cartesian graph, for the purpose of providing better grid lines that will minimize interpolation errors from interpolation on the log-log graph. **Table 404-1** in **Section 400** presents the tabular values of the **Figure 404-1**  $q_u$  versus  $T_c$  curve.

**Peak Rates of Discharge for Small Watersheds, Chapter 2**  
**(revised 10/73 for New Mexico, updated 2/85),**  
**Engineering Field Manual for Conservation Practices,**  
**U.S. Department of Agriculture, Soil Conservation Service**

## GENERAL

This update (2/85) incorporated the following changes:

1. Channel-loss factors may be used to adjust direct runoff from drainage areas greater than one square mile.
2. The rain distributions were reduced from four to one with the maximum hour containing 60 percent of the total rainfall.
3. An analysis of New Mexico gages was performed to verify the estimates of New Mexico, Chapter 2.
4. Various editorial changes were made.

## INTRODUCTION

This chapter presents procedures to estimate peak rates and volumes of runoff for a range of rainfalls and soil-cover conditions. These procedures may be used for drainage areas up to 10 square miles.

## LIMITATIONS

These methods are simplified for general situations. If suitable streamflow data are available for the area being studied, these data should be used in preference to the procedures in this chapter. Streamflow data from similar areas should be used to check reasonableness of results obtained by the chapter procedures. More detailed analysis using NEH-4 and Technical Release 20 should be used to estimate peak rates of runoff for drainage areas greater than 10 square miles or for special situations.

## FACTORS AFFECTING SURFACE RUNOFF

### GENERAL

Precipitation is the source of runoff from small watersheds. The soils and vegetation of the watershed affect the amount of precipitation that runs off. Mechanical treatment on a watershed, along with its topography and shape, also affect the rate at which water runs off. Runoff Curve Numbers (CNs) represent the combined effect of soil, vegetative cover, and conservation practices in runoff determinations.

### PRECIPITATION

The highest rates of runoff from small watersheds are usually caused by intense rainfall. The intensity of rainfall affects the rate of runoff more than it does the volume of runoff. The melting of accumulated snow in the mountains may result in a greater volume of runoff, but usually at a lesser rate than runoff caused by rainfall. The melting of a winter's snow accumulation over a large area may cause major flooding along rivers. Intense rainstorms that produce high rates of runoff in small watersheds usually do not extend over a large area. The same intense rainstorm that causes flooding in a small tributary is not likely to be the one that will cause major flooding in a main stream that drains several hundred square miles.

Data from recording rain gages were studied to determine an appropriate rainfall distribution for New Mexico. The maximum one hour contained an average of 60 percent of the total rainfall. Generally, New Mexico has more intense, shorter duration rainfalls than other parts of the U.S.

### ANTECEDENT MOISTURE CONDITION

The amount of precipitation occurring in the five days preceding the storm in question is an indication of the antecedent moisture condition (AMC) of the soil. The CNs in Table 2-2 are for an average AMC II. Runoff Curve Numbers can be adjusted to account for different antecedent moisture conditions according to the procedure in Chapter 10, NEH-4.

### HYDROLOGIC SOIL GROUPS

Soils have been classified into four Hydrologic Soil Groups; Table 2-1 shows New Mexico soils assigned to each of the four groups. The Hydrologic Soil Groups, according to their infiltration and transmission rates, are:

- A. (Low runoff potential). Soils having high infiltration rates even when thoroughly wetted. These consist chiefly of deep, well to excessively drained, sands or gravels. These soils have a high rate of water transmission in that water readily passes through them.
- B. Soils having moderate infiltration rates when thoroughly wetted. These consist chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to

moderately coarse textures such as loamy sands and coarse sandy loams. These soils have a moderate rate of water transmission.

C. Soils having slow infiltration rates when thoroughly wetted. These consist chiefly of soils with a layer that impedes downward movement of water or soils with moderately fine to fine textures such as loams and fine sandy loams. These soils have a slow rate of water transmission.

D. (High runoff potential). Soils having very slow infiltration rates when thoroughly wetted. These consist chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very slow rate of water transmission.

## HYDROLOGIC CONDITIONS OF SOILS

The soil and its hydrologic condition, in most cases, affect the volume of runoff more than any other single factor. The hydrologic condition of the soil is determined by its moisture content at the time of the storm, its humus and organic content and its temperature, and whether or not it is frozen.

## VEGETATIVE COVER

Vegetation affects runoff in several ways. The foliage and its litter maintain the soil's infiltration potential by preventing the sealing of the soil surface from raindrop impact. Foliage, retains some of the raindrops, increasing their chance of being evaporated. Some of the moisture is intercepted on the plant and withheld from the initial period of runoff. Vegetation transpires soil moisture leaving a greater void in the soil to be filled. Vegetation, including its ground litter, forms numerous barriers along the path of the water flowing over the surface of the land. This lengthens the travel time and increases opportunity for infiltration.

The following information can be used as a guide in determining the vegetative cover conditions for range sites. Grass cover is evaluated on plant basal area while trees and shrubs are evaluated using canopy cover. Litter can be an effective cover and should be considered.

## Cover Condition Classes

<u>Condition</u>	<u>Vegetative Cover</u>
Poor	Less than 30% ground cover
Fair	About 30% to 70% ground cover
Good	More than 70% ground cover

Percent of cover can be obtained by running the standard line intercept (refer to New Mexico Range Tech. Note #28 – Revised 8/12/71). After a few transects have been made, visual estimates are usually adequate.



## CONSERVATION PRACTICES

Conservation practices, in general, reduce sheet erosion and thereby maintain an open structure of the soil surface. This reduces the volume of runoff, but the effect diminishes rapidly with increase in storm magnitude.

Crop residues tilled into the soil and the residual root system from grasses that have been in the crop rotations produce a condition favoring greater infiltration and water storage in the soil profile. The effect of conservation tillage on reducing runoff ranges from slight to substantial.

Contouring and terracing reduce sheet erosion and increase the amount of rainfall withheld from runoff by the small reservoirs they form. Land areas in which level terraces have been constructed may be excluded from the drainage area above downstream measures if they store the design depth of runoff. Gradient terraces increase the distance water must travel and thereby increase the time-of-concentration. This, in turn, reduces the peak rate of discharge.

## TOPOGRAPHY

Watershed slopes affect the rate of runoff and the peak discharge rate at downstream points. Slopes have a smaller effect on the volume of runoff.

Small potholes may trap an initial amount of rain, thus reducing the amount of expected runoff. Where ponding or swampy areas occur in the watershed, a considerably amount of surface runoff may be retained in temporary storage. Technical Release 55 contains a procedure to adjust the peak discharge for ponding area.

## WATERSHED YIELD

Long term watershed yields are important to consider for some conservation practices. For example, when designing ponds for livestock water, the technician should consider a simplified water budget such as: available water equals runoff minus evaporation minus seepage. Livestock demand should be related to available water, and storage should be provided to accommodate demand and sedimentation. Exhibit 2-1 shows average annual runoff for New Mexico.

## PEAK RATE AND VOLUME OF DISCHARGE

### RAINFALL

The 24-hour rainfall depths for a desired location and frequency can be obtained from the appropriate 24-hour rainfall maps of Exhibit 2-2. The values shown on the maps are partial-duration series amounts and should be multiplied by the following factors to obtain annual series amounts for design purposes.

Factors for Converting Map Values to Annual Series

Recurrence Interval	Conversion Factor
2 year	0.88
5 year	0.96
10 year	0.99

## ESTIMATING DEPTH OF RUNOFF

The volume of runoff from a watershed may be expressed as the average depth of water that would cover the watershed. The depth is usually expressed in inches. The direct runoff in inches may be read from Figure 2-5, using the 24-hour runoff and Runoff Curve Number. Figure 2-5 gives the graphical solution of the SCS Runoff Equation. For calculator use, the Runoff Equation may be algebraically rearranged as follows:

$$Q = \frac{[P - (200/CN) + 2]^2}{P + (800/CN) - 8}$$

where:

Q	=	runoff in inches depth over the drainage area
P	=	rainfall in inches depth over the drainage area
CN	=	Curve Number

Refer to NEH-4 for details on development of the SCS Runoff Equation. The Runoff Curve Numbers for various watershed conditions are listed on Table 2-2 and Figure 2-1. These CNs reflect an average antecedent moisture condition II. A weighted Curve Number for a watershed can be estimated as shown in the following example:

### Example:

Given a 1,000-acre watershed. About 600 acres of the watershed is in Hydrologic Soil Group C, within which two-thirds of the area is row crops in good hydrologic condition and the other one-third is small grains in good condition. The remaining 400 acres of the watershed is in Hydrologic Soil Group D, all of which is juniper grass with 50 percent cover density.

Calculate weighted CN:

2/3 of 600 ac = 400 ac @ CN 85 and  $(400/1000) 85 = 34.0$

1/3 of 600 ac = 200 ac @ CN 83 and  $(200/1000) 83 = 16.6$

400 ac @ CN 87 and  $(400/1000) 87 = 34.8$

1000                      weighted CN = 85.4

(Round to 85)

## TRANSMISSION LOSSES

Direct runoff may be decreased to account for transmission losses. Table 2-3 presents channel-loss factors associated with drainage area size and climatic index. If adequate local data are available, the more detailed methods of NEH-4, Chapter 19 may be used to estimate losses.

## TIME OF CONCENTRATION

The time of concentration ( $T_c$ ) is the time it takes for runoff to travel from the hydraulically most distant part of the watershed area to the site location or point of reference.

Figure 2-2 can be used to determine  $T_c$  for all watersheds with typical gullying. This method applies to most New Mexico watersheds. The use of Figure 2-2 is explained in the following example:

Example:

Given a small agricultural watershed with typical gullying, determine the  $T_c$  using Figure 2-2. U.S.G.S. quadrangles provide the topographic information and a base for watershed delineation.

The length ( $L$ ) of the longest waterway from the site to the ridge, 7,250 feet.

The difference in elevation ( $H$ ) between the site location and the farthest most ridge, in feet, omitting vertical drops due to gully overfalls, etc., is 130 feet.

$T_c$  in hours using Figure 2-2 is 0.57 hours.

For small watershed where overland flow exists, Figure 2-3 may be used to estimate velocity in feet per second. The average slopes in percent may be obtained by surveying or from U.S.G.S. quadrangles.

$T_c$  can then be computed using:

$$T_c = \frac{L}{3600 V}$$

where:

$T_c$  = time of concentration (hr)

$L$  = length of water course (ft)

$V$  = velocity (ft/sec)

Example:

Given a small upland watershed where overland flow exists, determine  $T_c$  by using Figure 2-3. The slope of the watershed is 4%. The cover condition is nearly bare and untilled. The length of the water course is 7,250 feet.

The average velocity from Figure 2-3 is 2.0 fps

$$T_c = \frac{L}{3600 V} = \frac{7,250}{(3600)(2.0)} = 1.0 \text{ hr}$$

Refer to NEH-4, Technical Release 55, or other hydrology references for other methods to calculate  $T_c$ .

## PEAK DISCHARGE

Figure 2-4 was developed by computing hydrographs for a range of times of concentration. The rainfall distribution had a duration of 24 hours, and the maximum hour, which contained 60 percent of the total, was placed at the 6-hour interval.

The peak discharge can be obtained by determining the cfs per acre-inch of runoff from Figure 2-4. This value can then be multiplied by the discharge area in acres and the runoff in inches to obtain peak discharged in cfs.

Example No. 1: Not Gullied

A 1,000-acre watershed near Albuquerque has a point rainfall of 2.8 inches (100-year event). About 600 acres of the watershed is in Hydrologic Soil Group C; two-thirds of the 600 acres has crops in good hydrologic condition while the other one-third is in small grains in good hydrologic condition. The remaining 400 acres of the watershed is in Hydrologic Soil Group D and vegetative cover is juniper-grass with 50 percent cover density. The longest watercourse is 7,250 feet and the drop is 130 feet. Estimate the peak discharge.

1. Estimate the weighted Runoff Curve Number

$$400 \text{ ac @ CN } 85 = 34$$

$$200 \text{ ac @ CN } 83 = 16.6$$

$$40 \text{ ac @ CN } 87 = 34.8$$

$$85.4 \text{ say } 85$$

2. Estimate the  $T_c$  using Figure 2-3 since the watershed is not gullied.

$$\text{Slope in \%} = \frac{(130)(100)}{7,250} = 1.8\%$$

$$\text{Velocity} = 1.2 \text{ fps}$$

$$T_c = \frac{7,250}{(3600)(1.2)} = 1.68 \text{ hr say } 1.7 \text{ hr}$$

3. The unit peak discharge from Figure 2-4 is 0.39 cfs/acre-in.

4. The volume of runoff ( $Q$ ) from Figure 2-5 is 1.4 inches

5. Multiply runoff by channel-loss factor from Table 2-3.

$$(1.4 \text{ in.}) (0.93) = 1.3 \text{ in.}$$

6. Calculate peak discharge  $(0.39 \text{ cfs/ac-in.}) (1,000 \text{ ac}) (1.3 \text{ in.}) = 507 \text{ cfs}$ , say 510 cfs

Example No. 2: Typical Range (Gullied) near Santa Rosa, NM

See Hydrology Data Sheet, Figure 2-6

From maps or field investigation: 2,800 ac. vegetative cover is herbaceous, 20% cover density Soil Group C = 87%; 400 ac. vegetative cover is herbaceous, 20%, cover density, Soil Group B = 13%; 15,000 ft length of drainage; 420 ft elevation difference. Estimate the 50-yr peak discharge.

1. Determine the CN – Figure 2-1

CN of herbaceous, 20% cover, Hydrologic Soil Group C = 86

CN of herbaceous, 20% cover, Hydrologic Soil Group B = 79

$$87\% \times 86 = 74.82$$

$$13\% \times 79 = 10.27$$

85.09 use 85

2. Determine Time of Concentration ( $T_c$ ).

From Figure 2-2 using  $L = 15,000$  ft;  $H = 420$  ft

Find  $T_c = 0.8$  hours.

3. Rainfall – See Exhibit 2-2, 50-year, 24-hour precipitation map (NM-N-0374).

Find location of watershed and read  $P_{24} = 3.9$  in.

4. Cfs/ac-in. – This is the peak flow expected from each acre for each 1 in. of runoff for the given  $T_c$ . Figure 2-4 gives 0.65 cfs/ac-in.

5. Direct runoff Using Figure 2-5, for  $P = 3.9$  in. and  $CN = 85$  direct runoff  $Q = 2.37$  in.

6. Net runoff = Multiply direct runoff ( $Q$ ) times channel-loss factor (CLF).

$$\text{in.} \times 0.73 = 1.73 \text{ in.}$$

7. To determine peak discharge – Multiply area of watershed ( $A$ ) by net runoff ( $Q_n$ ) by cfs/ac-in.

$$3200 \text{ ac} \times 1.73 \text{ in.} \times 0.65 \text{ cfs/ac-in.} = 3598 \text{ cfs, say } 3600 \text{ cfs}$$

## ANALYSIS OF NEW MEXICO GAGES

Forty-nine gages, all with drainage areas less than 20 square miles, were analyzed. The shortest period of record contained 16 years, and the average period consisted of 24 years. The data extended through the year 1979.

Peak discharges estimated by this Chapter 2 method were compared to the discharges obtained from a statistical analysis of the gaged data. At the 100-year recurrence interval, 55 percent of NM Chapter 2 estimates were higher than gage estimates. At the 10-year recurrence interval, 65 percent of Chapter 2 estimates were higher than gage estimates. The analysis showed much variability and conclusions were not obvious. It was obvious that Chapter 2 overestimates at elevations greater than 7500 feet. Consult that SCS state hydrologist for more refined estimates at higher elevations. In general, the gage analysis supported the changes of this update to Chapter 2.

# **APPENDIX 6 EXAMPLE HYDROLOGIC CALCULATIONS**

## **CONTENTS:**

<b>Example Problem 6-1</b>	<b>Rational Formula Method</b>
<b>Example Problem 6-2</b>	<b>Rational Formula Method</b>
<b>Example Problem 6-3</b>	<b>Simplified Peak Discharge Method</b>
<b>Example Problem 6-4</b>	<b>Simplified Peak Discharge Method</b>
<b>Example Problem 6-5</b>	<b>Iterative Method with the Stream Hydraulic Method</b>
<b>Example Problem 6-6</b>	<b>HEC-HMS Model – NRCS Unit Hydrograph Method</b>
<b>Example Problem 6-7</b>	<b>Hatch Site 6 Hydrology - Runoff Determination Discussion</b>

**Example Problem 6-1 (Rational Formula Method)**

**Note:** The equation, figure, and table numbers referenced with 3 digits (ex. Table 403-1) refer to the same item numbers identified within the Drainage Design Manual.

**Project Information**

Location: Lemitar, New

Mexico Elevation: 4,740 ft

Watershed: Truck Stop, Scattered Semi-rural development, limited access urban highway,  
Pavement Runoff – 55% impervious

Design Frequency Flood: 10-yr

Watershed Area: 33.8 acres

**1. Compute the Time of Concentration using the Kirpich Equation.**

Total distance to most hydraulically remote distance to the drainage divide = 1,469 ft

Elevation difference = 4,751 – 4,718 = 33 ft

Slope  $S = 33 \text{ ft} / 1469 \text{ ft} = 0.0225 \text{ ft/ft}$

The Kirpich Equation is given as:

$$T_c = 0.0078 L^{0.77} S^{-0.385} \quad 403-2$$

where:

$T_c$	=	Time of Concentration (minutes)
$L$	=	maximum length of water travel (ft)
$S$	=	surface slope, given by $H/L$ (ft/ft)
$H$	=	difference in elevation between the most hydraulically remote point in the drainage basin and the outlet (ft)

$$T_c = 0.0078 \times (1469)^{0.77} \times (0.0225)^{-0.385} = 9.22 \text{ minutes} < 10 \text{ minutes}$$

Use the minimum allowable time of concentration.

$$T_c = 10.0 \text{ minute}$$

**2. Compute the 1-hr rainfall depth and design storm intensity.**

From **Table 403-1** below, develop the Intensity-Duration-Frequency Curves shown in **Figure 403-1**.

**Table 403-1 NOAA Data Server Sample IDF Spreadsheet-Lemitar NM**

Point precipitation frequency estimates (inches/hour)

NOAA Atlas 14 Volume 1 Version 5

Data type: Precipitation intensity

Time series type: Partial duration

Project area: Southwest

Location name: Lemitar, New Mexico, US\*

Station Name: -

Latitude: 34.1580°

Longitude: -106.9181°

Elevation: 4712 ft\*

\* source: Google Maps

## PRECIPITATION FREQUENCY ESTIMATES

by

duration

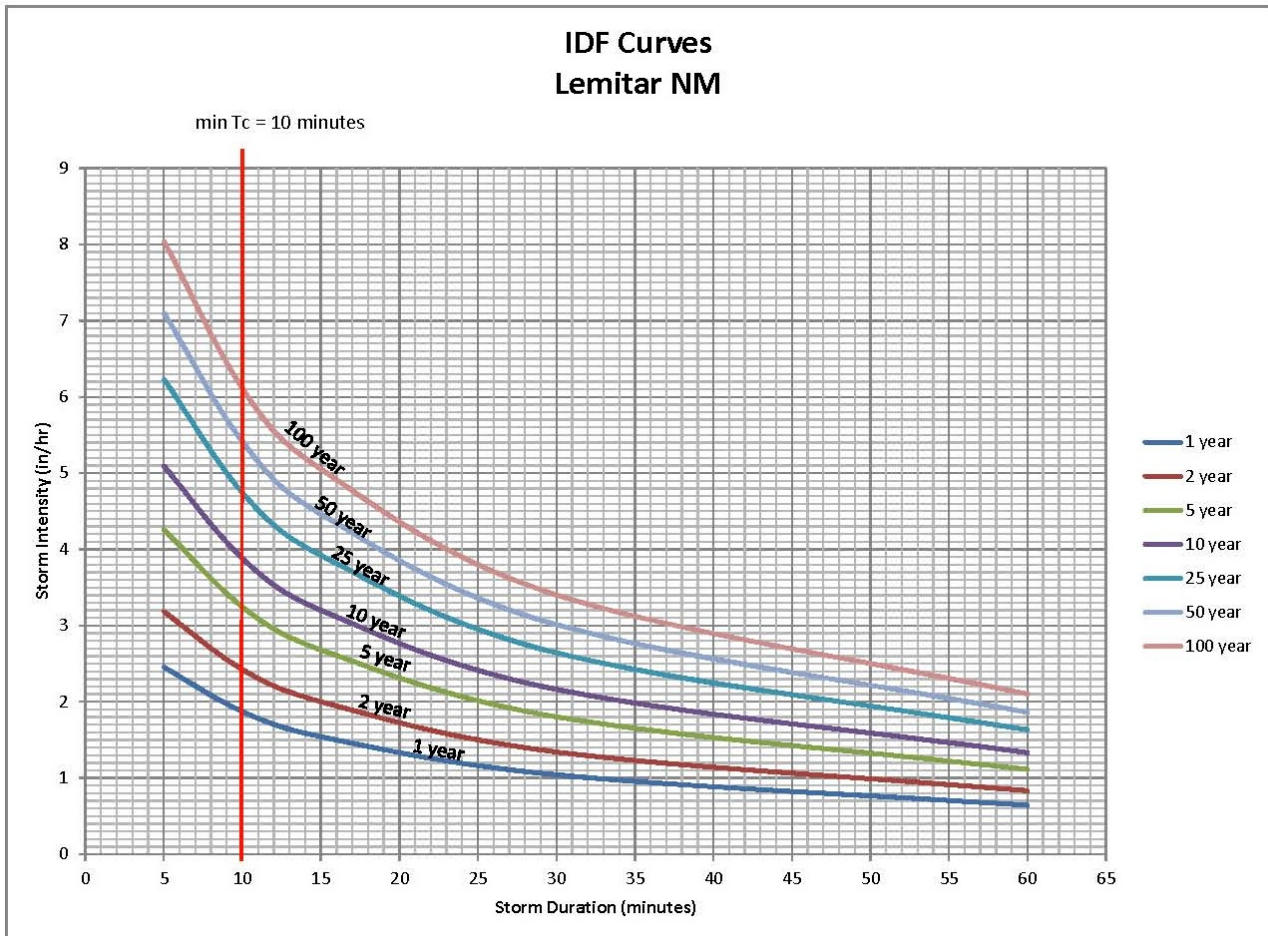
for ARI:		1	2	5	10	25	50	100	200	500	1000 years
5-min:	5	2.45	3.18	4.26	5.09	6.23	7.1	8.04	9	10.31	11.35
10-min:	10	1.87	2.42	3.24	3.88	4.74	5.41	6.11	6.85	7.84	8.64
15-min:	15	1.54	2	2.68	3.2	3.92	4.46	5.05	5.66	6.48	7.14
30-min:	30	1.04	1.34	1.8	2.16	2.64	3.01	3.4	3.81	4.36	4.81
60-min:	60	0.64	0.83	1.11	1.33	1.63	1.86	2.1	2.36	2.7	2.98
2-hr:		0.37	0.48	0.64	0.76	0.95	1.11	1.29	1.49	1.8	2.06
3-hr:		0.27	0.34	0.45	0.54	0.67	0.78	0.9	1.04	1.25	1.43
6-hr:		0.16	0.2	0.25	0.3	0.36	0.42	0.48	0.55	0.66	0.75
12-hr:		0.08	0.11	0.13	0.16	0.19	0.22	0.25	0.29	0.34	0.38
24-hr:		0.05	0.06	0.08	0.09	0.11	0.12	0.14	0.16	0.18	0.2
2-day:		0.03	0.03	0.04	0.05	0.06	0.07	0.07	0.08	0.1	0.11
3-day:		0.02	0.02	0.03	0.03	0.04	0.05	0.05	0.06	0.07	0.08
4-day:		0.02	0.02	0.02	0.03	0.03	0.04	0.04	0.05	0.05	0.06
7-day:		0.01	0.01	0.02	0.02	0.02	0.02	0.03	0.03	0.03	0.04
10-day:		0.01	0.01	0.01	0.01	0.02	0.02	0.02	0.02	0.03	0.03
20-day:		0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.02	0.02
30-day:		0	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01
45-day:		0	0	0	0.01	0.01	0.01	0.01	0.01	0.01	0.01
60-day:		0	0	0	0	0.01	0.01	0.01	0.01	0.01	0.01

Date/time (GMT): Fri Nov 13 22:14:03 2015

pyRunTime: 0.127875804901

Create the IDF curves using the data in **Table 403-1** and following the process outlined in **Section 403.2**.





**Figure 403-1 IDF Curves from NOAA Data Server-Lemitar, NM**

Read the rainfall intensity from **Figure 403-1** for the 10-yr storm frequency for 10 minutes.

$$i = 3.9 \text{ in./hr}$$

3. Find the 10-yr 1-hr precipitation  $P_1$  using the procedures in Section 403.2 and using the example (Table 403-2) below:

**Table 403-2 NOAA Depth-Duration-Frequency Table from NOAA Data Server**

Point precipitation frequency estimates (inches)  
 NOAA Atlas 14 Volume 1 Version 5  
 Data type: Precipitation depth  
 Time series type: Partial duration  
 Project area: Southwest  
 Location name: Lemitar, New Mexico, US\*  
 Station Name: -  
 Latitude: 34.1584°  
 Longitude: -106.9189°  
 Elevation: 4713 ft\*  
 \* source: Google Maps

PRECIPITATION FREQUENCY ESTIMATES

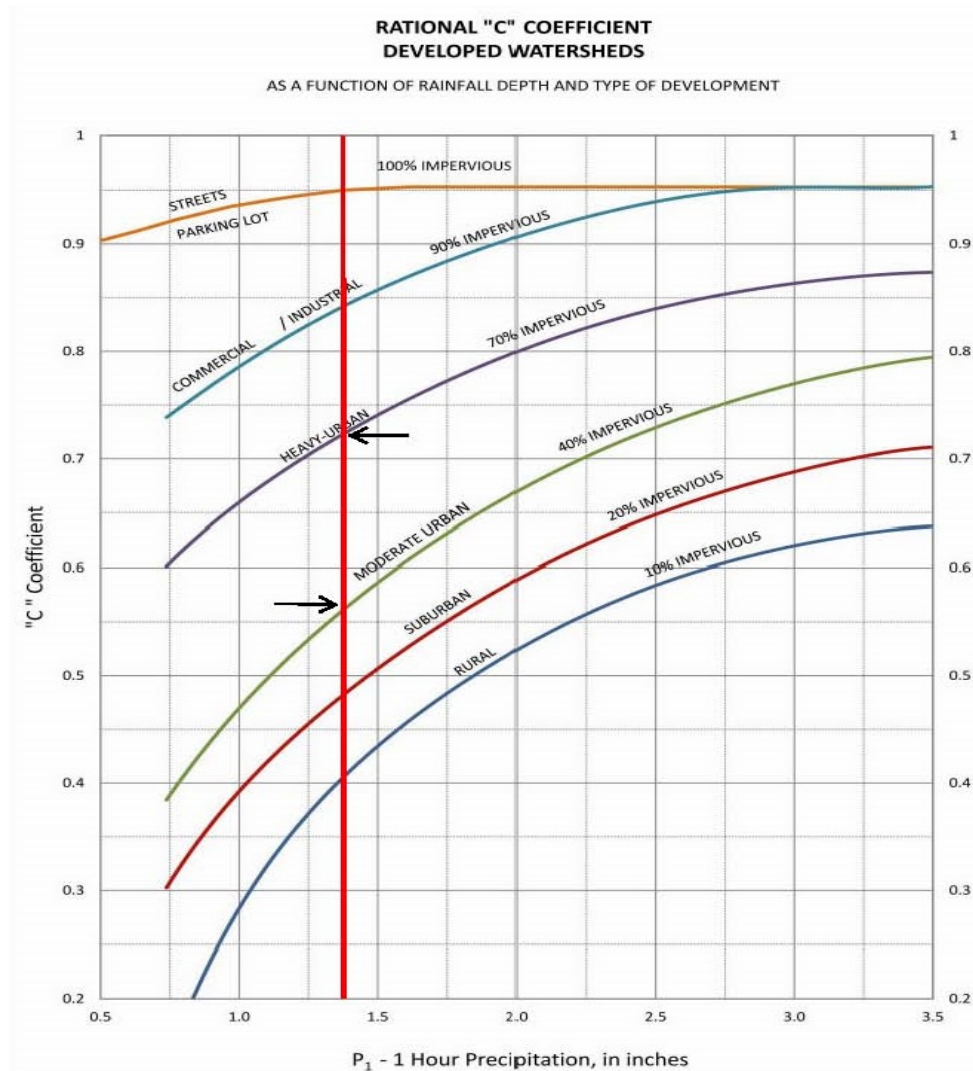
by duration	1	2	5	10	25	50	100	200	500	1000 years
5-min:	0.2	0.27	0.35	0.42	0.52	0.59	0.67	0.75	0.86	0.95
10-min:	0.31	0.4	0.54	0.65	0.79	0.9	1.02	1.14	1.31	1.44
15-min:	0.39	0.5	0.67	0.8	0.98	1.12	1.26	1.41	1.62	1.78
30-min:	0.52	0.67	0.9	1.08	1.32	1.5	1.7	1.9	2.18	2.4
60-min:	0.64	0.83	1.11	1.33	1.63	1.86	2.1	2.36	2.7	2.98
2-hr:	0.75	0.96	1.27	1.52	1.9	2.22	2.58	2.98	3.59	4.13
3-hr:	0.81	1.03	1.35	1.61	2	2.33	2.7	3.12	3.75	4.3
6-hr:	0.93	1.18	1.51	1.78	2.18	2.52	2.9	3.31	3.93	4.48
12-hr:	1.01	1.28	1.63	1.91	2.31	2.65	3.03	3.44	4.06	4.59
24-hr:	1.16	1.45	1.82	2.12	2.55	2.9	3.29	3.72	4.35	4.88
2-day:	1.27	1.59	1.98	2.3	2.76	3.13	3.54	3.99	4.64	5.2
3-day:	1.36	1.7	2.12	2.46	2.94	3.34	3.78	4.25	4.95	5.55
4-day:	1.45	1.81	2.25	2.61	3.12	3.55	4.01	4.51	5.25	5.89
7-day:	1.67	2.08	2.57	2.96	3.52	3.97	4.46	4.99	5.77	6.41
10-day:	1.84	2.3	2.84	3.29	3.91	4.41	4.96	5.56	6.43	7.17
20-day:	2.33	2.9	3.54	4.03	4.71	5.25	5.81	6.39	7.2	7.89
30-day:	2.81	3.5	4.23	4.78	5.53	6.11	6.7	7.3	8.12	8.81
45-day:	3.41	4.23	5.08	5.7	6.51	7.12	7.71	8.29	9.11	9.78
60-day:	3.9	4.84	5.8	6.52	7.44	8.13	8.81	9.47	10.33	10.98

Date/time (GMT): Mon Nov 16 19:12:46 2015  
 pyRunTime: 0.126244068146

The 1-hr rainfall depth,  $P_1$ , is obtained from the Depth-Duration-Frequency **Table 403-2** for the design storm frequency. Read the value for the 10-yr depth: 1.33 inches.

4. Find Hydrologic Soils Group (HSG) from the NRCS Web Soil Survey Web site:  
<http://websoilsurvey.nrcs.usda.gov/>.

From **Figure 403-2** the engineer will find that there is not a curve for 55% impervious.



**Figure 403-2 Rational 'C' Coefficient Developed Watersheds**

Read the 'C' Factor for  $P_1 = 1.33$  inches for 70% impervious ( $C = 0.72$ ) and for 40% impervious ( $C = 0.56$ ). Interpolate linearly between the two C Factors:

$$C_a = (C_1 - C_2) / 2 + C_2 = [(0.72 - 0.56) / 2] + 0.56 = 0.64$$

5. **Compute the 10-yr design frequency peak discharge using the Rational Formula, Equation 403-1.**

$$Q = C i A$$

$$Q_{p \text{ 10-yr}} = 0.64 \times 3.9 \times 33.8 = 84.4 \text{ cfs}$$

**Example Problem 6-2 (Rational Formula Method)**

**Note:** The equation, figure, and table numbers referenced with 3 digits (ex. Table 403-1) refer to the same item numbers identified within the Drainage Design Manual.

Project Information

Location: Near Roswell

Elevation: 4,000 ft

Watershed: Upland range

Design Frequency Flood: 50-yr

Area: 80 acres

**1. Compute the Time of Concentration using the Kirpich Equation.**

Total distance to the hydraulically most remote point in the drainage basin = 12,100 ft

Assume: overland flow for 400 ft at  $S = 0.050$  ft/ft

shallow concentrated flow 1,700 ft at  $S = 0.030$  ft/ft

channel flow 10,000 ft at  $S = 0.021$  ft/ft

Find average slope for basin:

$$S = \frac{L_1 \times S_1 + L_2 \times S_2 + L_3 \times S_3}{L_1 + L_2 + L_3}$$

$$S = \frac{400 \times 0.050 + 1700 \times 0.030 + 10,000 \times 0.021}{400 + 1700 + 10,000} = 0.0232 \frac{\text{ft}}{\text{ft}}$$

The Kirpich Equation is given as:

$$T_c = 0.0078 L^{0.77} S^{-0.385}$$

**403-2**

where:

$T_c$	=	Time of Concentration, minutes
$L$	=	maximum length of water travel, ft
$S$	=	surface slope, given by $H/L$ , ft/ft
$H$	=	difference in elevation between the most hydraulically remote point in the drainage basin and the outlet, ft

$$T_c = 0.0078 \times (12,100)^{0.77} \times (0.0232)^{-0.385} = 46.3 \text{ minutes}$$

Check  $T_c$  from the Kirpich Equation using the Upland Method using **Figure 402-15**.

For Overland flow use Manning's Equation and roughness values from Table 15-1 and Short-grass prairie  $n=0.15$

**Table 15-1** Manning's roughness coefficients for sheet flow (flow depth generally  $\leq 0.1$  ft)

Surface description	$n^{1/}$
Smooth surface (concrete, asphalt, gravel, or bare soil).....	0.011
Fallow (no residue).....	0.05
Cultivated soils:	
Residue cover $\leq 20\%$ .....	0.06
Residue cover $> 20\%$ .....	0.17
Grass:	
Short-grass prairie.....	0.15
Dense grasses <sup>2/</sup> .....	0.24
Bermudagrass.....	0.41
Range (natural).....	0.13
Woods: <sup>3/</sup>	
Light underbrush.....	0.40
Dense underbrush.....	0.80

1 The Manning's  $n$  values are a composite of information compiled by Engman (1986).

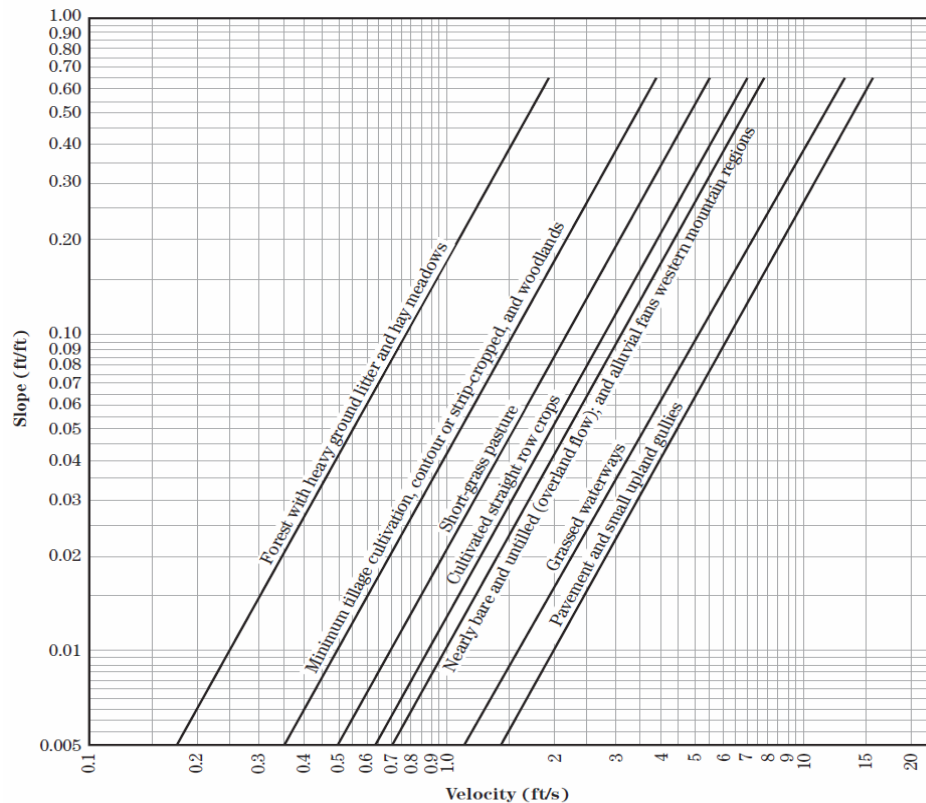
2 Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures.

3 When selecting  $n$ , consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.

Ref: NRCS Part 630 Chapter 15 Time of Concentration May 2010 Page 15-6

Overland Flow Velocity =

$$V = (1.486 \times R^{2/3} \times S^{1/2}) / n = (1.486 \times (1)^{2/3} \times (0.05)^{1/2}) / 0.15 = 2.2 \text{ ft/sec}$$



**Figure 402-15 Velocity Versus Slope for Shallow Concentrated Flow**

For Shallow Concentrated Flow use the line labeled: “Nearly Bare and Untilled, Alluvial Fans in Western Mountain Regions” for  $S=3\%$ ,  $V=2.5$  ft/sec

For Channel Flow use the Iterative Method within the Stream Hydraulic Method

- Estimate an approximate trapezoidal cross section for the majority of the stream reach and use one of the many on-line Manning’s equation solvers available to determine the velocity for the channel reach.
- For  $S=2.1\%$ , Bottom Width of 10 ft, average side slope of 2 H to 1 V, Manning’s ‘n’ of 0.035, and a flow depth of 1 ft - results in an average  $V=5.4$  ft/sec

Calculate  $T_c$  as the sum for all three reaches:

$$T_c = L_1/V_1 + L_2/V_2 + L_3/V_3 = 400 \text{ ft}/2.2 \text{ ft/sec} + 1700 \text{ ft}/2.5 \text{ ft/sec} + 10,000 \text{ ft}/5.4 \text{ ft/sec} = 2713 \text{ sec} = 45.2 \text{ min (close, use 46.3 minutes)}$$

Use  $T_c = 46.3$  minutes

**Note** – This also equals the  $T_c$  computed previously with the Kirpich Equation.



## 2. Compute the 1-hr rainfall depth and design storm intensity.

Find  $P_1$  (1-hr precipitation, inches) and (intensity, "i" inches/hour) following the procedures outlined in **Section 403.2**.

Go to NOAA Precipitation Frequency Data Server (PFDS)

[http://hdsc.nws.noaa.gov/hdsc/pfds/pfds\\_map\\_cont.html?bkmrk=nm](http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=nm)

Find the location on the map at the NOAA web site and produce a Depth-Duration-Frequency Table as described in **Section 403.2 Rainfall** (see **Table 403-1ex** below).

Point precipitation frequency estimates (inches)  
 NOAA Atlas 14 Volume 1 Version 5  
 Data type: Precipitation depth  
 Time series type: Partial duration  
 Project area: Southwest  
 Location name: Roswell, New Mexico, US\*  
 Station Name: -  
 Latitude: 33.3953°  
 Longitude: -104.6256°  
 Elevation: 3843 ft\*  
 \* source: Google Maps

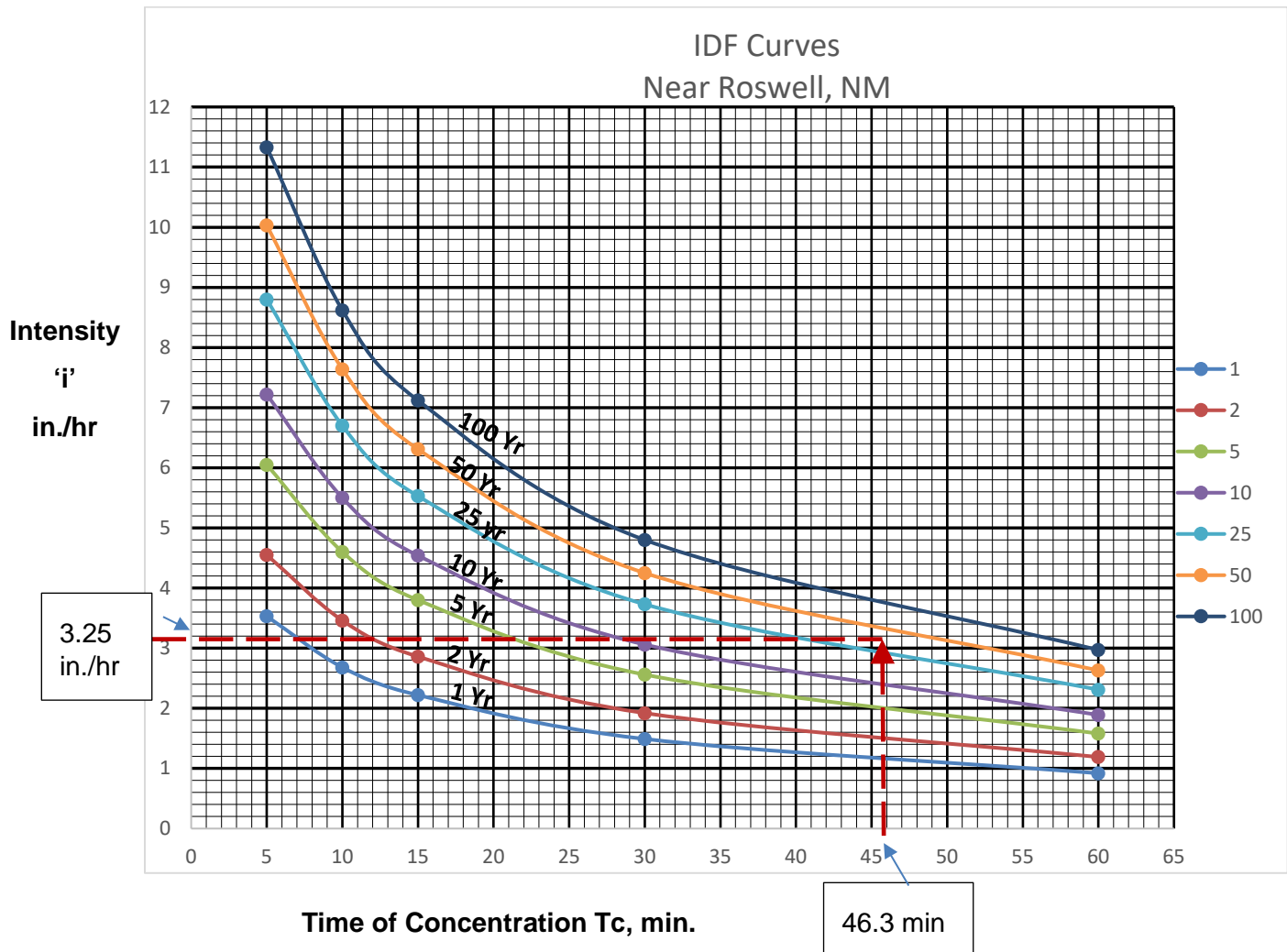
PRECIPITATION FREQUENCY ESTIMATES										
by duration	1	2	5	10	25	50	100	200	500	1000 years
5-min:	0.29	0.37	0.5	0.59	0.72	0.83	0.93	1.04	1.19	1.31
10-min:	0.44	0.57	0.76	0.9	1.1	1.26	1.42	1.59	1.81	1.99
15-min:	0.55	0.71	0.94	1.12	1.37	1.56	1.76	1.97	2.25	2.47
30-min:	0.74	0.95	1.26	1.51	1.84	2.1	2.37	2.65	3.03	3.33
60-min:	0.91	1.18	1.57	1.87	2.28	2.6	2.94	3.28	3.75	4.12
2-hr:	1.05	1.36	1.83	2.2	2.71	3.12	3.55	3.99	4.61	5.11
3-hr:	1.11	1.44	1.93	2.32	2.85	3.29	3.74	4.22	4.89	5.42
6-hr:	1.27	1.63	2.17	2.59	3.19	3.67	4.19	4.72	5.48	6.09
12-hr:	1.42	1.82	2.41	2.88	3.53	4.05	4.61	5.2	6.02	6.69
24-hr:	1.57	2.01	2.67	3.19	3.94	4.54	5.17	5.84	6.79	7.56
2-day:	1.71	2.2	2.94	3.54	4.41	5.13	5.9	6.74	7.94	8.93
3-day:	1.83	2.35	3.14	3.78	4.7	5.46	6.27	7.14	8.39	9.41
4-day:	1.95	2.51	3.34	4.02	4.99	5.78	6.63	7.54	8.84	9.9
7-day:	2.26	2.9	3.85	4.61	5.67	6.52	7.42	8.37	9.7	10.78
10-day:	2.51	3.22	4.27	5.11	6.28	7.22	8.22	9.27	10.74	11.92
20-day:	3.2	4.08	5.28	6.2	7.43	8.37	9.33	10.3	11.61	12.61
30-day:	3.77	4.79	6.11	7.1	8.4	9.37	10.33	11.29	12.53	13.46
45-day:	4.47	5.67	7.22	8.37	9.86	10.97	12.07	13.16	14.57	15.62
60-day:	5.16	6.54	8.25	9.47	11.03	12.16	13.26	14.31	15.63	16.6

Date/time (GMT): Mon Nov 16 22:15:58 2015  
 pyRunTime: 0.129033088684

**Table 403-1ex Depth Duration Frequency Table near Roswell, NM**

The 1-hr rainfall depth,  $P_1$ , is obtained from the Depth-Duration-Frequency **Table 403-1ex** for the design storm frequency. The value for the 50-yr depth is 2.6 inches.

3. Determine the storm intensity 'i'.



**Figure 403-2ex Intensity Duration Frequency Curves for near Roswell, NM**

Read from **Figure 403-2ex** to find the rainfall intensity 'i' for the 50-yr storm and a  $T_c$  of 46.3 minutes.

$i = 3.25$  in./hr

4. Compute an area-weighted Runoff Coefficient "C" for the drainage basin.

$P_1 = 2.6$  inches (from **Figure 403-1ex**)

Subarea 1: 30 acres, upland range, HSG = C, percent vegetation cover estimated as 30%. From **Figure 403-4** below,  $C = 0.46$



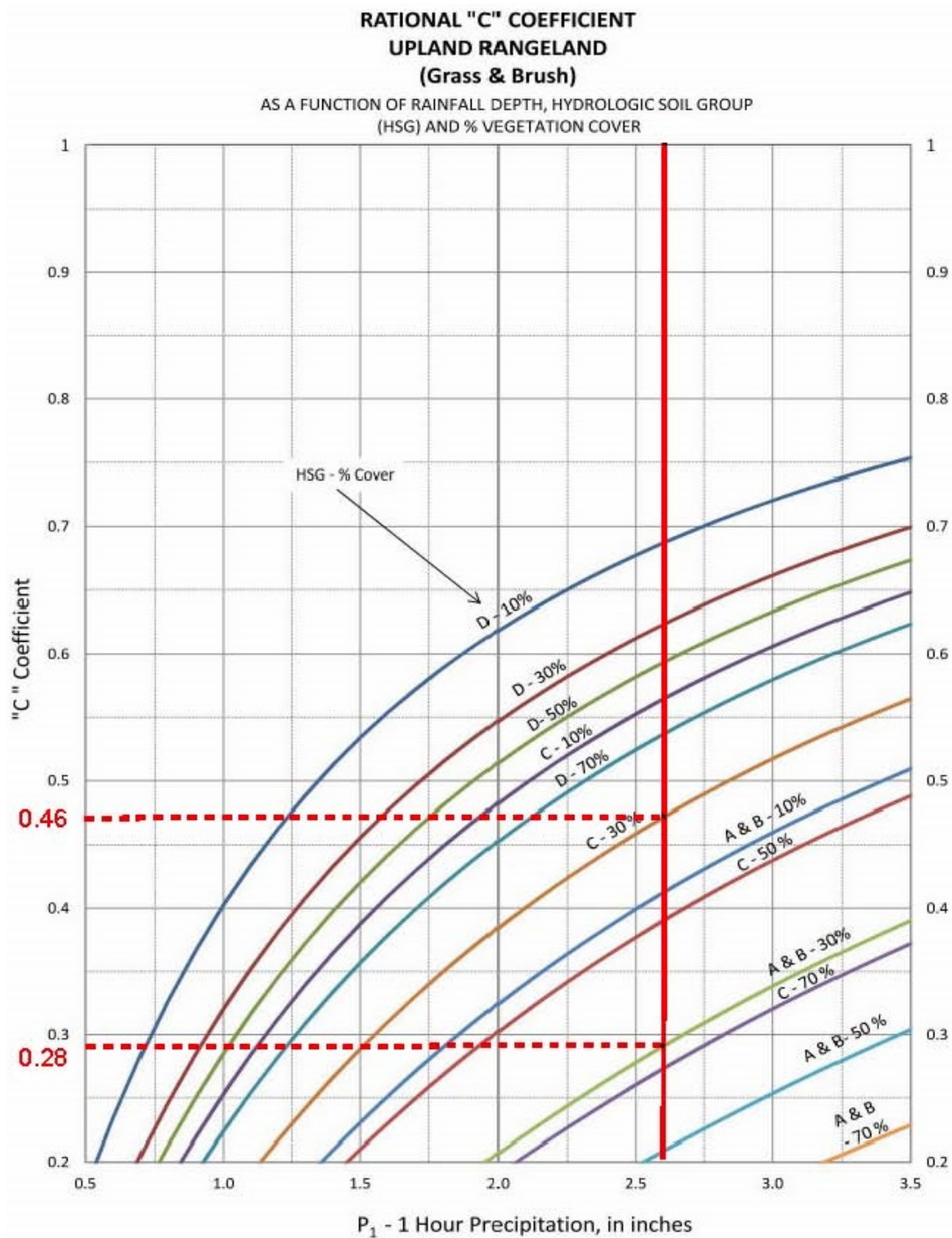


Figure 403-4 Rational 'C' Coefficient Upland Rangeland (Grass & Brush)

Subarea 2: 50 acres, upland range, HSG = B, 30% vegetation cover.

From **Figure 403-4**,  $C = 0.28$

The area weighted Runoff Coefficient is computed as

**Weighted C**

$$C = [(0.46)(30) + (0.28)(50)] / (30 + 50) = 0.35$$

**5. Compute the 50-yr design frequency flood peak discharge using the Rational Formula.**

$$Q = C i A$$

$$Q_{p \text{ 50-yr}} = 0.35 \times 3.25 \times 80 = 91 \text{ cfs}$$

**Example Problem 6-3 (Simplified Peak Discharge Method)**

**Note:** The equation, figure, and table numbers referenced with 3 digits (ex. Table 403-1) refer to the same item numbers identified within the Drainage Design Manual.

**Project Information**

Location: I-25 Arroyo crossing South of Lemitar, New Mexico

Elevation: 5,073 ft

Watershed: Arid mountain range and desert escarpment

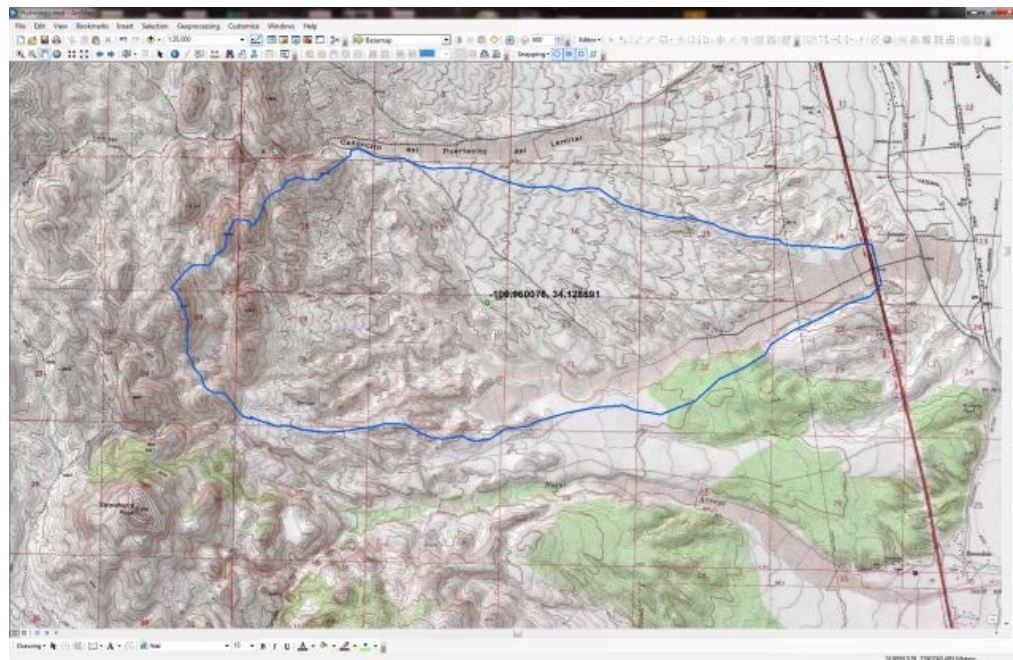
Design Frequency Flood: 50-yr

Check Flood: 100-yr

Watershed area: 4,856 acres (7.6 mi<sup>2</sup>)

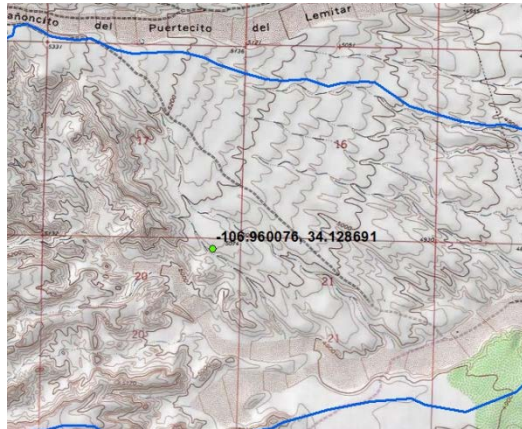
**1. Delineate the watershed boundary & Locate the Centroid**

Develop the contributing area to the project site Using USGS 1:24,000 quad sheets, see **Figure 1**.



**Figure 1 Basin Delineation**

Using a GIS or Drafting tool (or the “plumb line method”) to determine the centroid of the basin **Figure 2**.

**Figure 2 Centroid Location****2. Determine the rainfall depth for the watershed**

Using the centroid found for the watershed, determine rainfall depths for the 50-yr and 100-yr 24-hr storm events using the NOAA PFDS website **Figure 3**.

**NOAA ATLAS 14 POINT PRECIPITATION FREQUENCY ESTIMATES: NM**

**DATA DESCRIPTION**  
Data type: precipitation depth Units: english Time series type: partial duration

**SELECT LOCATION**  
1. Manually:  
a) Enter location (decimal degrees, use "-" for S and W): latitude: 34.128691 longitude: -106.960076 submit  
b) Select station (click here for a list of stations used in frequency analysis for NM): select station

2. Use map:

**LOCATION INFORMATION:**  
Name: Lemitar, New Mexico, US  
Latitude: 34.1287  
Longitude: -106.9601  
Elevation: 5073 ft

**Figure 3 NOAA Website**

From the server, create a hard copy record of the output (preferably a .pdf document) and note the 50-yr and 100-yr 24-hr rainfall depths.

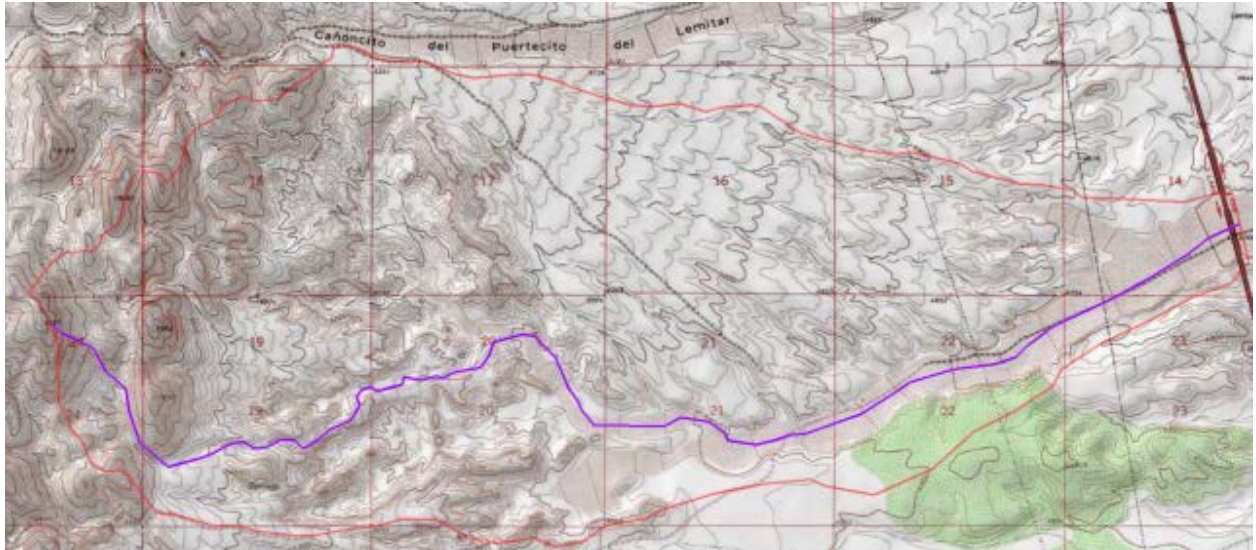
50-yr 24-hr rainfall depth = 3.0 in.

100-yr 24-hr rainfall depth = 3.4 in.



### 3. Calculate the Time of Concentration

Measure the longest flow path of the basin **Figure 4**.



**Figure 4 Flow Path Length**

Flow path length:  $L = 33,570$  ft

Upstream elevation: 6,357 ft

Downstream elevation: 4,700 ft

Elevation change:  $6,357 \text{ ft} - 4,700 \text{ ft} = 1,657 \text{ ft}$

Slope  $S$ :  $\text{Elevation change} / \text{Flow path length} = 1,657 \text{ ft} / 33,570 \text{ ft} = 0.0493 \text{ ft/ft}$

Choose  $T_c$  method using **Table 402-6**. From examination of aerial photography and local topography, use gullied watershed method.

Calculate  $T_c$ , using Kirpich Equation (**Eq. 403-2**)

$$T_c = 0.0078 L^{0.77} S^{-0.385}$$

$$T_c = 0.0078 * 33,570^{0.77} 0.0493^{-0.385} = 75.9 \text{ minutes (1.27 hrs)}$$

### 4. Determine the Runoff Curve Number

Define an AOI (Area of Interest) at the Web Soil Survey (**Figure 5**) using the basin delineated (this can be manually inputted, or a shapefile can be uploaded).

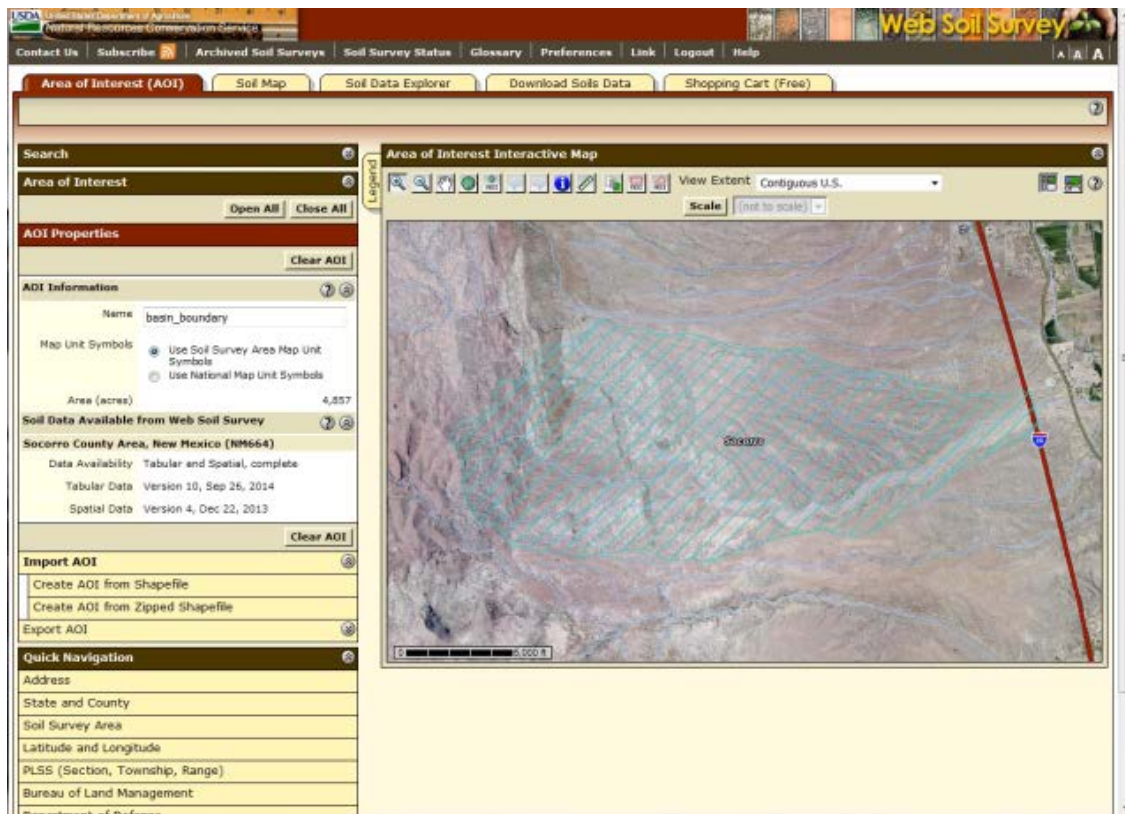


Figure 5 Defining AOI

5. Generate a report for the soils Group using the AOI is defined

Click on the Soil Data Explorer tab and select under “Soil Qualities and Features” select Hydrologic Soil Group, click on “View Ratings” (**Figure 6**) and produce a custom pdf document (**Figure 7**), this can be part of the supplemental information provided in the reports.

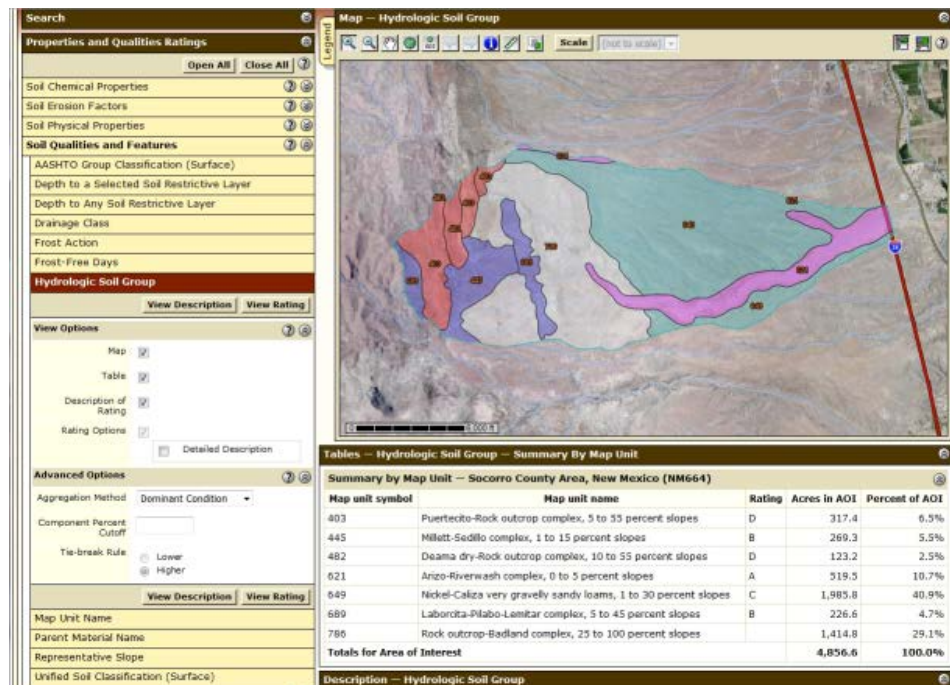


Figure 6 Hydrologic Soil Group Ratings

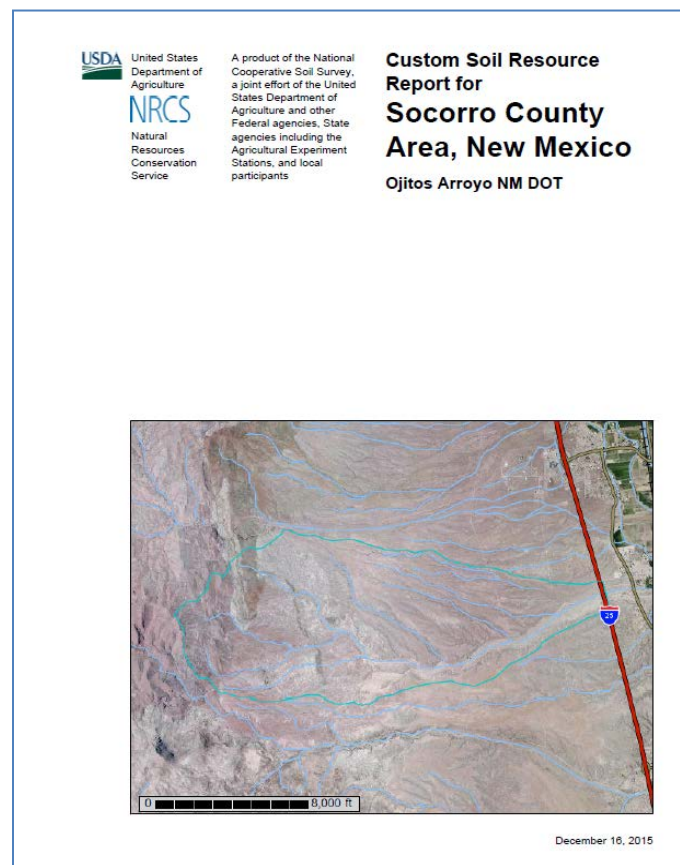


Figure 7 Custom Soils Report

**6. Assign a Curve Number based on the land use for each specific soil group and Table 402-5 and compute the weighted runoff depth and corresponding runoff weighted Curve Number**

Take the table of Hydrologic Soil Group percentages from the Soils report and assign a Curve Number. Based on the vegetative cover observed from aerial photography in the watershed, use the values for "Desert shrub" in "Fair" condition. Because all 4 hydrologic soil groups are represented, and Curve Numbers vary by more than 10, the weighted runoff method should be applied to calculate the basin composite Curve Number, or runoff weighted Curve Number.

**Table 1** presents the data applied to compute the weighted runoff depth, and that was applied to determine the runoff weighted Curve Number.

**Table 1 Soils Data, Weighted Runoff Depth Calculation and Runoff Weighted CN**

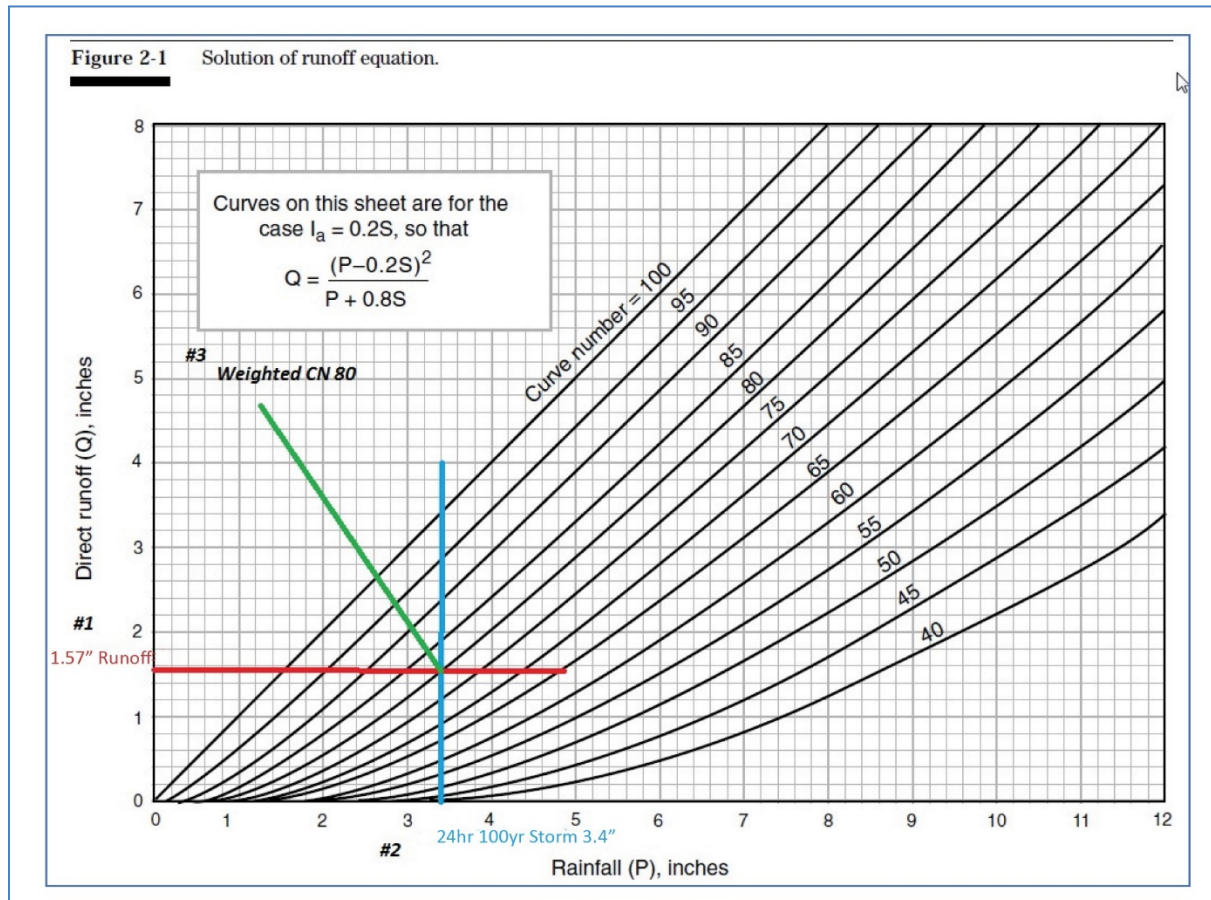
Soil Group Unit Symbol	Area	Percent	Hyd. Soil Group	CN Assumed	50-yr 24-hr rainfall depth	100-yr 24-hr rainfall depth	S	50- yr Q (Runoff Depth)	100- yr Q (Runoff Depth)
	acres	%			in.	in.		in.	in.
							a	b	b
403	317.4	6.54%	D	86	3.0	3.4	1.63	1.66	2.01
445	269.3	5.55%	B	72	3.0	3.4	3.89	0.81	1.06
482	123.2	2.54%	D	86	3.0	3.4	1.63	1.66	2.01
621	519.5	10.70%	A	55	3.0	3.4	8.18	0.19	0.31
649	1985.8	40.89%	C	81	3.0	3.4	2.35	1.31	1.63
689	226.6	4.67%	B	72	3.0	3.4	3.89	0.81	1.06
786	1414.8	29.13%	D	86	3.0	3.4	1.63	1.66	2.01
Total	4856.6								
50-yr Weighted Runoff Depth =				1.28	50-yr Runoff Weighted CN = 77				c
100-yr Weighted Runoff Depth =				1.57	100-yr Runoff Weighted CN = 80				c
a - S = potential maximum soil moisture retention after runoff begins, Equation 402-2									
b - Q = runoff depth, in., Equation 402-1									
c - Estimated from Figure 402-8									

The runoff weighted CN depth was computed to be 1.28 inches for the 50-yr storm (3.0 inch rainfall depth).

The runoff weighted CN depth was computed to be 1.57 inches for the 100-yr storm (3.4 inch rainfall depth).

The 50-yr and 100-yr runoff weighted Curve Numbers presented in **Table 1** were determined from **Figure 8 (Figure 402-8)** using the runoff weighted depths and corresponding rainfall depths.





**Figure 8 (Figure 402-8)**

The runoff weighted CN = 80 for the 100-yr storm, and CN = 77 for the 50-yr storm (determined from **Figure 8**), however assume CN= 80 for remaining calculations.

## 7. Determine the Runoff Peak flow and Runoff volume for the required storms

From above the Direct runoff was found to be:

50-yr 24-hr  $Q_d = 1.28$  inches  
 100-yr 24-hr  $Q_d = 1.57$  inches

Using **Figure 9 (404-1)** find the unit peak discharge ( $q_u$ ) based on the  $T_c = 1.27$  hours

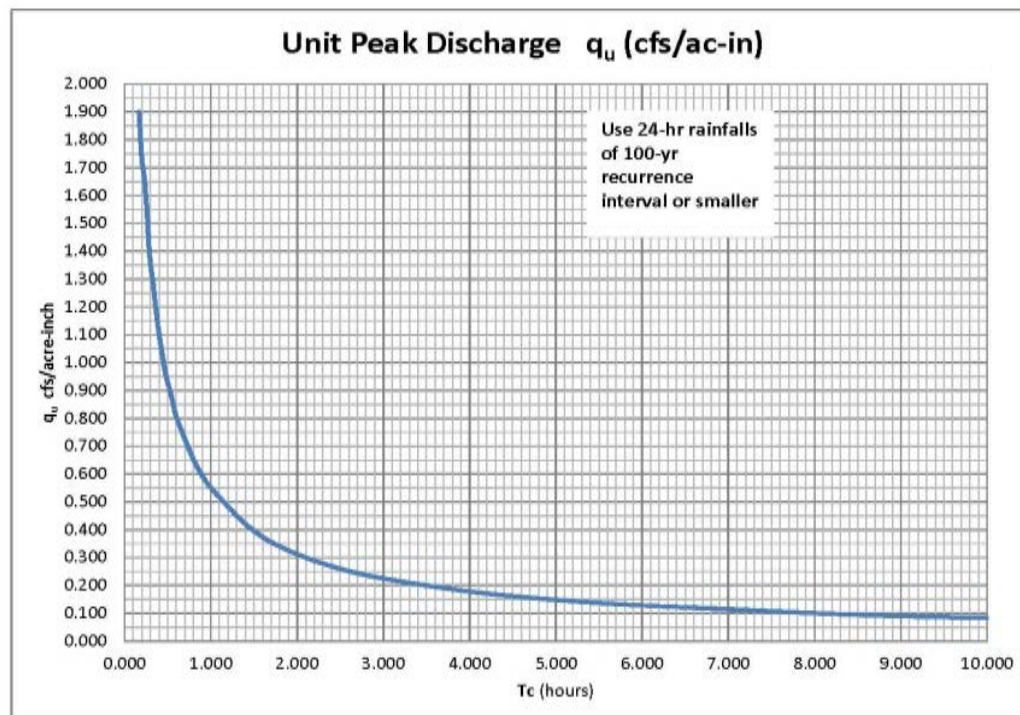


Figure 9 (Figure 404-1)

$$q_u = 0.46 \text{ cfs/acre-in.}$$

8. Solve for Peak Discharge using equation 404-2  $Q_p = A * Q_d * q_u$

$$Q_p \text{ 50-yr} = 2,859 \text{ cfs}$$

$$Q_p \text{ 100-yr} = 3,507 \text{ cfs}$$

9. Solve for Discharge per acre:

$$50\text{-yr} = 2,859 \frac{\text{cfs}}{4,856} \text{ acres} = 0.59 \frac{\text{cfs}}{\text{ac}}$$

$$100\text{-yr} = 3,507 \frac{\text{cfs}}{4,856} \text{ acres} = 0.72 \text{ cfs/ac}$$

10. Calculate the runoff volume for watershed using Equation 404-3:

$$Q_v = A * Q_d / 12:$$

$$50\text{-yr} = (1.28 \text{ inches} \times 4856 \text{ acres}) / 12 \text{ in./ft} = 518 \text{ ac-ft}$$

$$100\text{-yr} = (1.57 \text{ inches} \times 4856 \text{ acres}) / 12 \text{ in./ft} = 635 \text{ ac-ft}$$

11. Complete the worksheet below:

**Simplified Peak Discharge Method Worksheet**

Structure Location: MP: 154.6 County: Socorro  
 District: 1- Deming  
 Structure Description: Interstate 25 Bridges North and South Bound  
 Drainage Area:  $A = 4856$  acres, 7.6 mi<sup>2</sup>  
 Elevation at Centroid of Watershed: Elev = 5073 ft \*  
 Location of Centroid: Lat: 34.1287 Long: 106.9601  
 Time of Concentration:  $T_c = 1.27$  hours  
 Method: ☐ Upland ☒ Kirpich ☐ Mixed  
 Weighted Runoff Curve Number: CN = 80  
 Method: ☐ Area ☒ Runoff  
 Unit Peak Discharge (from Figure 404-1):  $q_u = 0.46$  cfs/ac-in

<b><u>Design Frequency Flood</u></b>	<u>50</u> - year	<u>100</u> - year
24-hour Rainfall Depth (NOAA PFDS):	$P_{24} = 3.0$ in.	$P_{24} = 3.4$ in.
Direct Runoff (Figure 402-8):	$Q_d = 1.28$ in.	$Q_d = 1.57$ in.
Peak Discharge, $Q_p = A \cdot Q_d \cdot q_u$ :	$Q_p = 2859$ cfs	$Q_p = 3507$ cfs
Discharge per acre	<u>0.59</u> cfs/ac	<u>0.72</u> cfs/ac
Runoff Volume, $Q_v = A \cdot Q_d/12$ :	$Q_v = 518$ ac-ft	$Q_v = 635$ ac-ft

Project Location: MP 154.6 Near Lemitar NM  
 CN#: \_\_\_\_\_  
 Date: 3/12/18  
 Computed By: KCE  
 Checked By: CME

\* If elevation is greater than 7500 ft, use NRCS Unit Hydrograph method

**Example Problem 6-4 (Simplified Peak Discharge Method)**

**Note:** The equation, figure, and table numbers referenced with 3 digits (ex. Table 403-1) refer to the same item numbers identified within the Drainage Design Manual.

**Project Information**

Location: Existing Culvert Crossing NM 26 NW of Deming, New Mexico (MP 25.4)

Elevation: 4,720 ft.

Watershed: Arid mountain range and desert escarpment

2014 AADT (Annual Average Daily Total) for road 2,207

(From Section 200 Design Criteria)

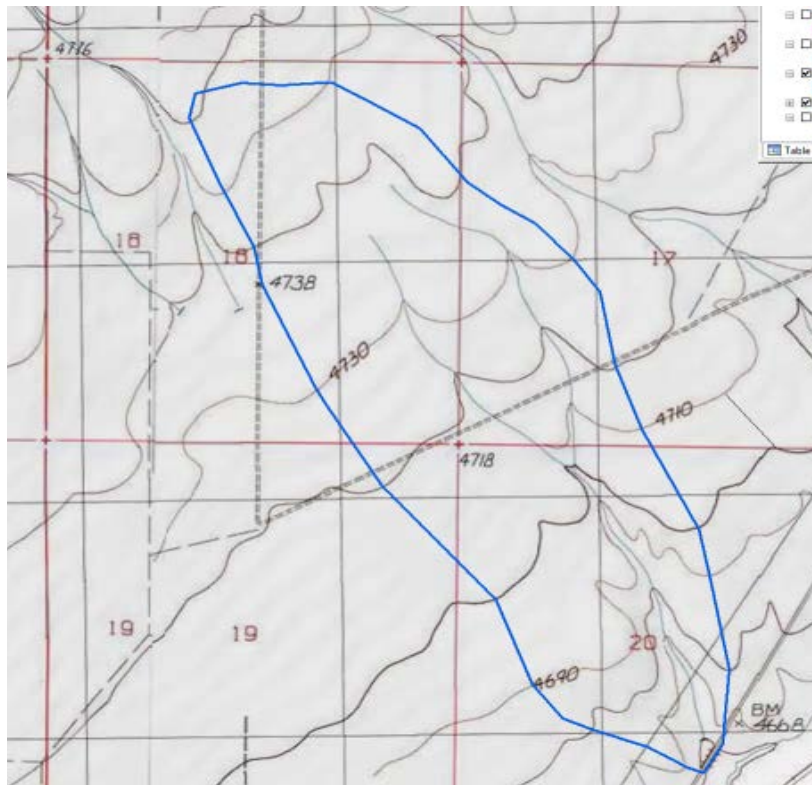
Design Frequency Flood: 50-yr

Check Flood: 100-yr

Watershed area: 690 acres (1.07 mi<sup>2</sup>)

**1. Delineate the watershed boundary & Locate the Centroid**

Develop the contributing area to the project site Using USGS 1:24,000 quad sheets, see **Figure 1**.



**Figure 1 Basin Delineation**

Using a GIS or Drafting tool (or the “plumb line method”) to determine the centroid of the basin (**Figure 2**).



Figure 2 Centroid Location

## 2. Determine the rainfall depth for the watershed

Using the centroid found for the watershed, determine rainfall depths for the 50-yr and 100-yr 24-hr storm events using the NOAA PFDS website (**Figure 3**).



Figure 3 NOAA Website

From the server, create a hard copy record of the output (preferably a .pdf document) and note the rainfall depths.

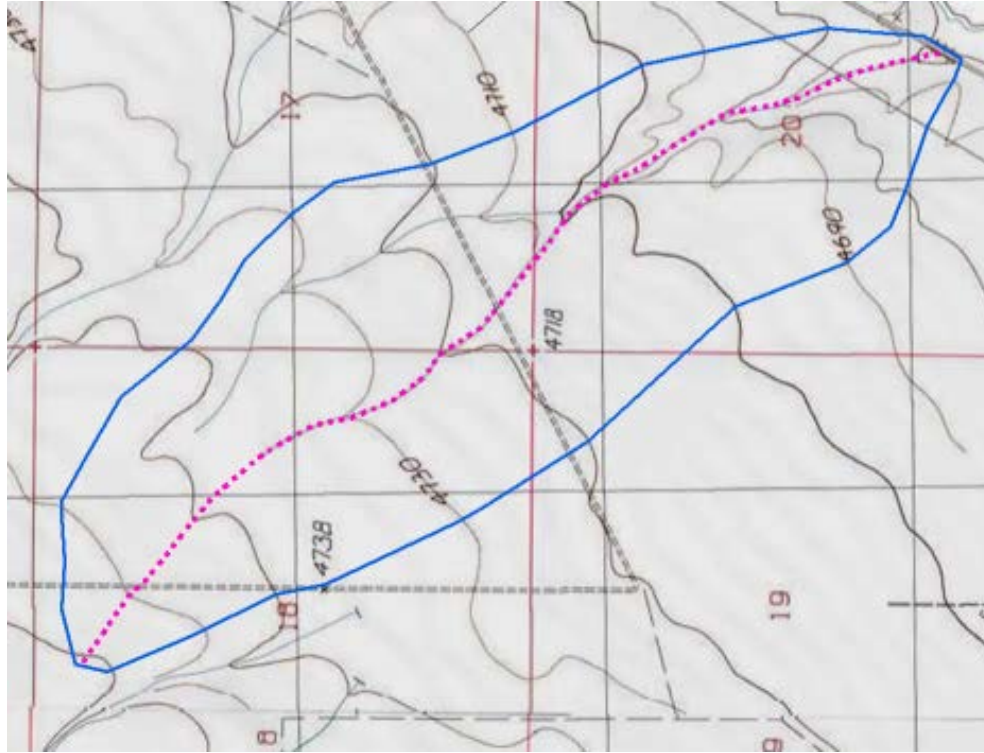


50-yr 24-hr rainfall depth = 3.21 in.

100-yr 24-hr rainfall depth = 3.56 in.

### 3. Calculate the Time of Concentration

Measure the longest flow path of the basin (**Figure 4**).



**Figure 4 Flow Path Length**

Flow path length total:  $L = 11,580$  ft

Upstream elevation: 4,760 ft

Downstream elevation: 4,660 ft

Elevation change:  $4,760 \text{ ft} - 4,660 \text{ ft} = 100 \text{ ft}$

Slope  $S$ :  $\text{Elevation change} / \text{Flow path length} = 100 \text{ ft} / 11,580 \text{ ft} = 0.0086 \text{ ft/ft}$

Determine  $T_c$  method. Examining aerial photography and local topography, use gullied watershed method (using Kerby-Kirpich Method).

$$T_c = T_{ov} + T_{ch}$$

Calculate  $T_{ov}$ , use the Kerby Equation (**Eq. 402-9**) for the upland part (1,200ft)

Slope  $S$ :  $(4,760 \text{ ft} - 4,745 \text{ ft}) / 1,200 \text{ ft} = 0.0125 \text{ ft/ft}$

Based on examination of the aerial photography,  $N = 0.2$

$$T_{ov} = K(L \times N)^{0.467} \times S^{-0.235}$$

$$T_{ov}=0.828(1200 \times 0.2)^{0.467} \times 0.0125^{-0.235}=30 \text{ minutes}$$

Calculate  $T_{ch}$ , use the Kirpich Equation (Eq. 403-2) for the gullied part (10,380ft)

Slope S:  $(4,745\text{ft} - 4,660\text{ft})/10,380\text{ft} = 0.0081\text{ ft/ft}$

$$T_{ch}=0.0078L^{0.77}S^{-0.385}$$

$$T_{ch}=0.0078 \times 10,380^{0.77} 0.0081^{-0.385} = 61.6 \text{ minutes}$$

Compute  $T_c$  as with the Kerby-Kirpich Method as follows:

$$T_c = T_{ov} + T_{ch} = 30 \text{ min} + 61.6 \text{ min} = 91.6 \text{ minutes} = 1.53 \text{ hrs}$$

#### 4. Determine the Runoff Curve Number

Define an AOI (Area of Interest) at the Web Soil Survey (**Figure 5**) using the basin delineated (this can be input manually or a shapefile can be uploaded).

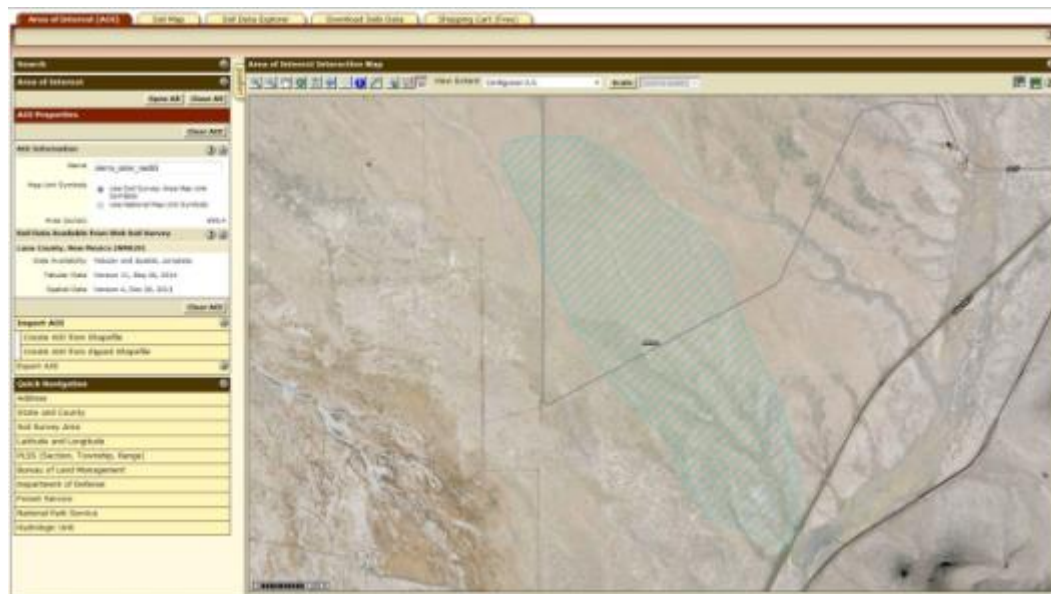


Figure 5 Defining AOI

#### 5. Generate a report for the soils using the AOI as defined

Click on the Soil Data Explorer tab and select under “Soil Qualities and Features” select Hydrologic Soil Group, click on “View Ratings” (**Figure 6**) and produce a custom pdf document (**Figure 7**), this can be part of the supplemental information provided in the reports.

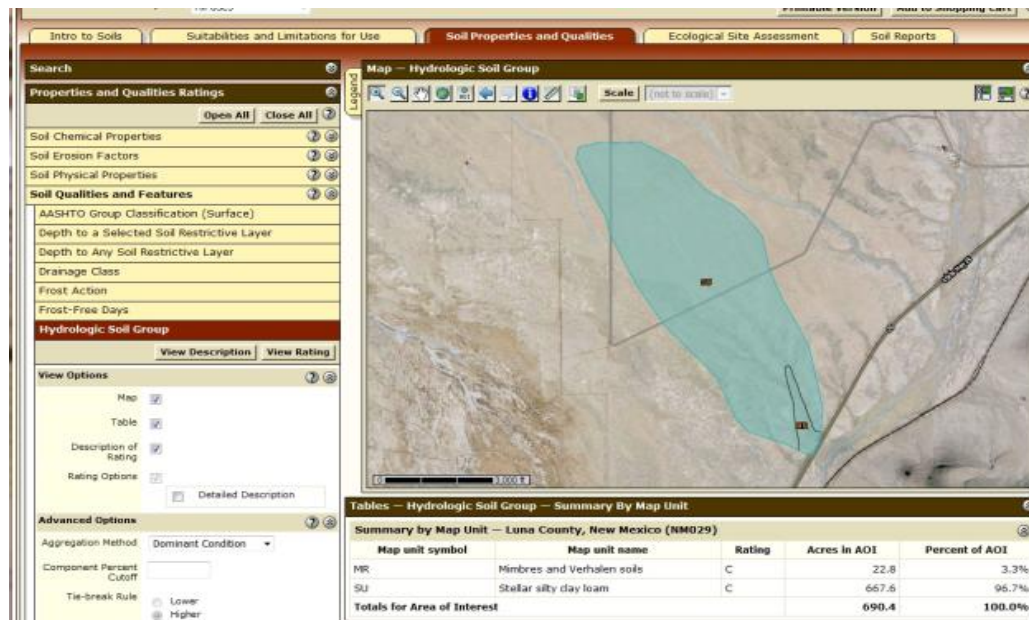


Figure 6 Hydrologic Soils Group Ratings

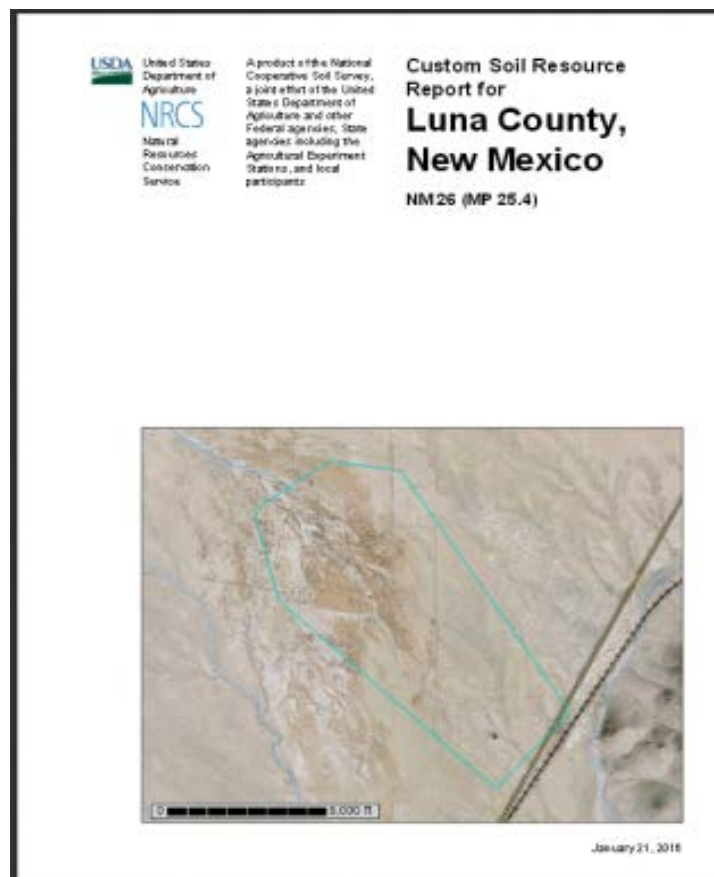


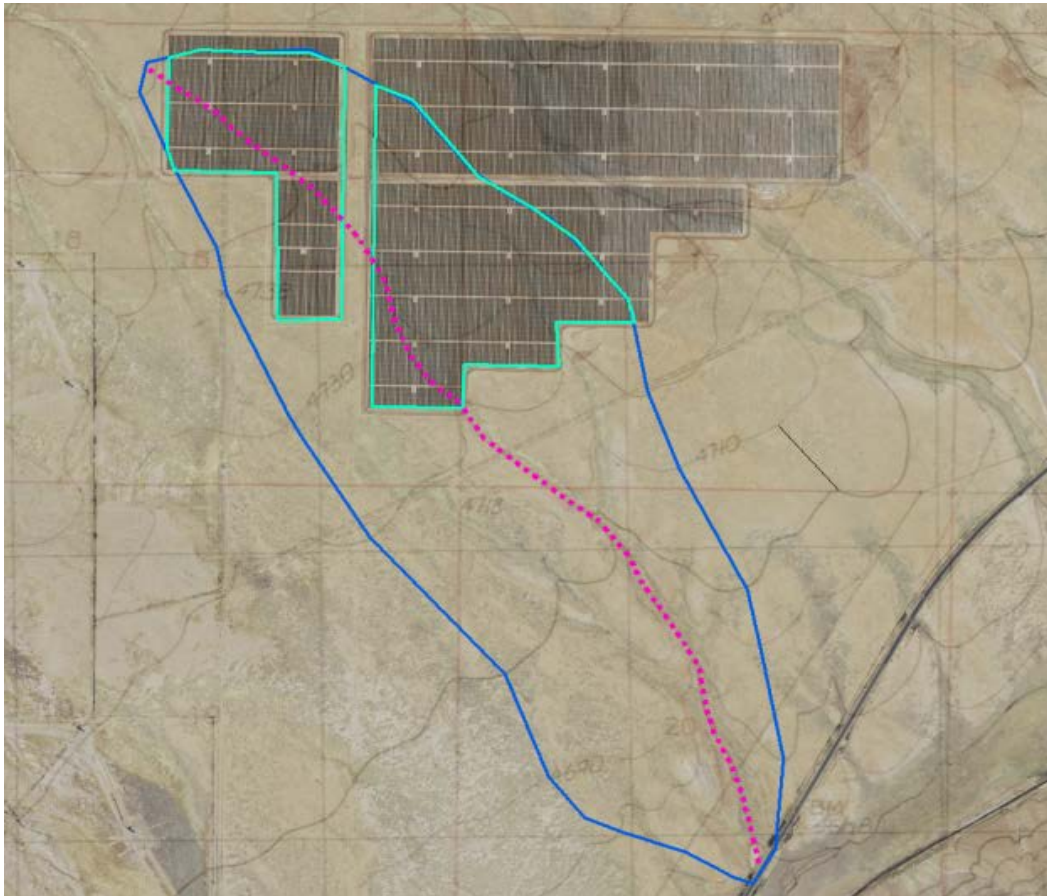
Figure 7 Custom Soils Report



**6. Assign a Curve Number** based on the land use for each specific soil group and **Table 402-5**.

Take the table of Hydrologic soil group percentages from the Soils report and assign. Using current aerial photography (**Figure 8**), a new solar farm has been installed in the watershed.

Based on the land use observed from aerial photography in the watershed, use the values for "Desert shrub" in "Fair" condition and industrial use with 70-75% imperviousness.



**Figure 8 Land Use and Cover**

Since there is only one soil group, and the Curve Numbers are within 10 or less, the composite Curve Number can be calculated using the area weighted method for the basin.

Soils Group	Landuse	Area	CN
C	Desert Shrub (fair)	475	81
C	Industrial	215	91

Calculating the basin average:

$$\text{Composite CN} = (\text{Area}_1 * \text{CN}_1 + \text{Area}_2 * \text{CN}_2 + \text{Area}_x * \text{CN}_x) / \text{Area}_{\text{tot}}$$

$$(475 \text{ acres} * 81 + 215 \text{ acres} * 91) / 690 \text{ acres} = \mathbf{84 \text{ CN}}$$

Determine the runoff volume with the composite CN for the watershed, either by equations (402-1 & 402 -2) or using **Figure 402-8**.

$$\text{Eq. 402-1 } Q = \frac{[(P - 0.2S)^2]}{P + 0.8S}$$

$$\text{Eq. 402-2 } S = \left( \frac{1000}{\text{CN}} \right) - 10$$

Solving for S:

$$S = \left( \frac{1000}{84} \right) - 10 = 1.9$$

Solving for Q (runoff depth)

$$Q_{50} = \frac{[(3.21 - 0.2 \cdot 1.9)^2]}{3.21 + 0.8 \cdot 1.9} = 1.69 \text{ in.}$$

$$Q_{100} = \frac{[(3.56 - 0.2 \cdot 1.9)^2]}{3.56 + 0.8 \cdot 1.9} = 1.99 \text{ in.}$$

**7. Determine the Runoff Peak flow and Runoff volume for the required storms.**

From above the Direct runoff depth was found to be:

$$50\text{-yr } 24\text{-hr } Q = 1.69 \text{ in.}$$

$$100\text{-yr } 24\text{-hr } Q = 1.99 \text{ in.}$$

Using **Figure 9 (404-1)** find the unit peak discharge ( $q_u$ ) based on  $T_c = 1.53$  hours

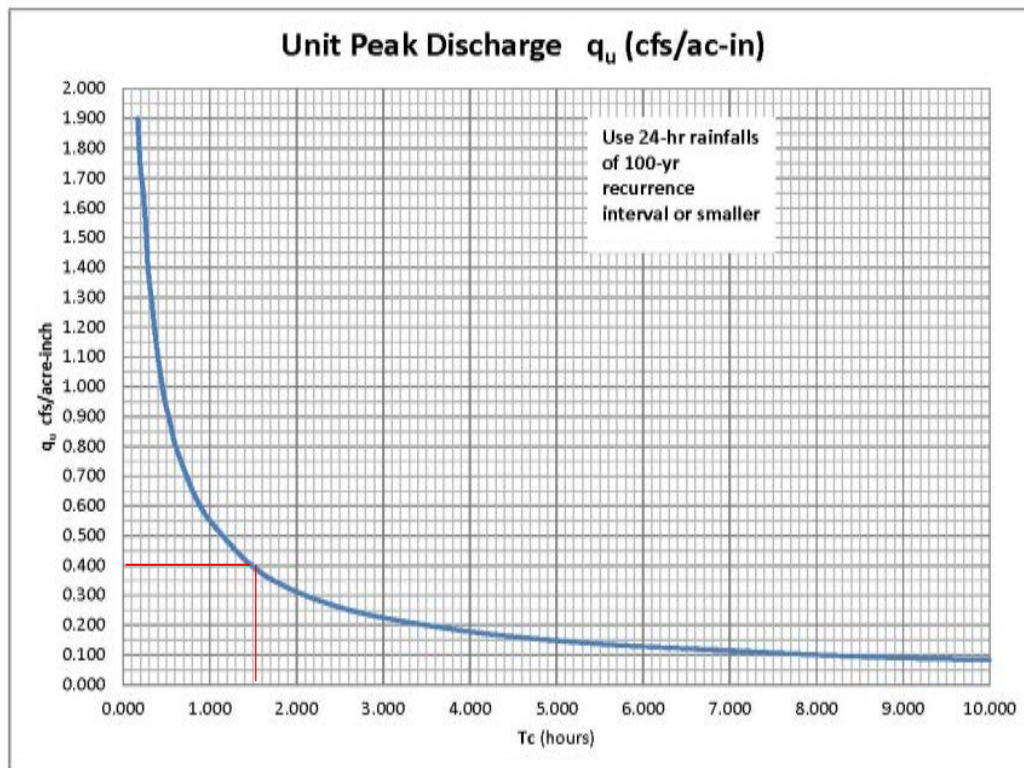


Figure 9 (Figure 404-1)

$q_u = 0.38$  cfs/acre-in.

**8. Solve for Peak Discharge using equation 404-2  $Q_p = A * Q_d * q_u$**

$Q_p$  50-yr = 690 acres x 1.69 in. x 0.38 cfs/acre-in. = 443 cfs

$Q_p$  100-yr = 690 acres x 1.99 in. x 0.38 cfs/acre-in. = 522 cfs

**9. Solve for Discharge per acre**

50-yr =  $443 \frac{\text{cfs}}{690} \text{ acres} = 0.64 \frac{\text{cfs}}{\text{ac}}$

100-yr =  $522 \text{ cfs} / 690 \text{ acres} = 0.76 \text{ cfs/ac}$

**10. Calculate the runoff volume for watershed using Equation 404-3**

$$Q_v = A * Q_d / 12$$

50-yr =  $(1.69 \text{ inches} \times 690 \text{ acres}) / 12 \text{ in./ft} = 97 \text{ ac-ft}$

100-yr =  $(1.99 \text{ inches} \times 690 \text{ acres}) / 12 \text{ in./ft} = 114 \text{ ac-ft}$

**11. Complete the worksheet below**

**Simplified Peak Discharge Method Worksheet**

Structure Location: MP: 25.4 County: Luna  
 District: 1- Deming  
 Structure Description: NM 26 Culvert  
 Drainage Area:  $A = 690$  acres,  $1.07$  mi<sup>2</sup>  
 Elevation at Centroid of Watershed: Elev = 4720 ft \*  
 Location of Centroid: Lat: 32.55 Long: 107.48  
 Time of Concentration:  $T_c = 1.53$  hours  
 Method: ☐ Upland ☒ Kirpich ☐ Mixed  
 Weighted Runoff Curve Number:  $CN = 84$   
 Method: ☐ Area ☒ Runoff  
 Unit Peak Discharge (from Figure 404-1):  $q_u = 0.38$  cfs/ac-in

<u>Design Frequency Flood</u>	<u>50</u> - year	<u>100</u> - year
24-hour Rainfall Depth (NOAA PFDS):	$P_{24} = 3.21$ in.	$P_{24} = 3.56$ in.
Direct Runoff (Figure 402-8):	$Q_d = 1.69$ in.	$Q_d = 1.99$ in.
Peak Discharge, $Q_p = A \cdot Q_d \cdot q_u$ :	$Q_p = 443$ cfs	$Q_p = 522$ cfs
Discharge per acre	$0.64$ cfs/ac	$0.76$ cfs/ac
Runoff Volume, $Q_v = A \cdot Q_d / 12$ :	$Q_v = 97$ ac-ft	$Q_v = 114$ ac-ft

Project Location: NM 26 NW of Deming, NM  
 CN#: \_\_\_\_\_  
 Date: 3/12/18  
 Computed By: KCE  
 Checked By: CME

\* If elevation is greater than 7500 ft, use NRCS Unit Hydrograph method

**Example Problem 6-5 (Iterative Method within the Stream Hydraulic Method)****(Manual Section 402.9.4)**

**Note:** The equation, figure, and table numbers referenced with 3 digits (ex. Table 403-1) refer to the same item numbers identified within the Drainage Design Manual.

**Project Information**

Determine the discharge at a new road crossing required to safely pass the 100-yr peak discharge using the iterative procedure to determine the  $T_c$ . See the **Watershed Map** below.

Location: Northeast Albuquerque, New Mexico

Drainage Area: 1047 acres

Basin Averaged CN = 83



**Watershed Map**

**Calculate Time of Concentration  $T_c$**  using the Upland Method found in **Section 402.9.1**.

Basin  $T_c$  Hydraulic Characteristics:

**Total Length of Longest Flow Path = 9,118 ft**

- Sheet Flow = 150 ft
- Shallow Concentrated Flow = 2,268 ft
- Channel Flow = 6,700 ft

**Calculate Sheet Flow Contribution to  $T_c$** 

Sheet Flow Length = 150 ft

Slope: 0.29 ft/ft

Manning's: 0.15

2-yr 24-hr storm depth = 1.65 in.

(Note this is the mountainous regions below Sandia Crest with steep, rocky terrain)

Using **Eq. 402-7** below:



$$T_t = \frac{0.007(nl)^{0.8}}{(P_2)^{0.5}S^{0.4}} = \frac{0.007(0.15 \cdot 150)^{0.8}}{(1.65)^{0.5}0.29^{0.4}} = 0.11 \text{ hrs}$$

$$T_{\text{sheet}} = 0.11 \text{ hrs}$$

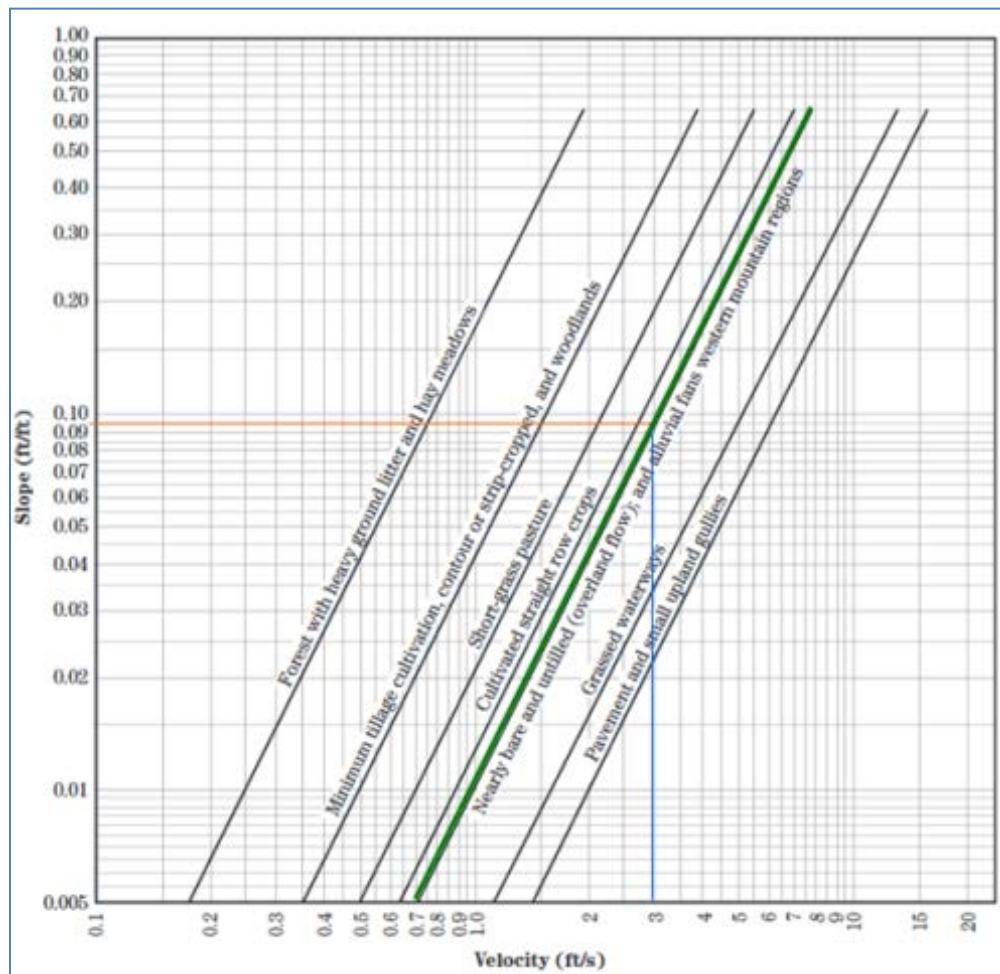
### Calculate Shallow Concentrated Flow Contribution to $T_c$

Shallow Concentrated Flow Length = 2268 ft

Slope: 0.095 ft/ft

Type: Western Mountain regions

From **Figure 402-15 Velocity Versus Slope for Shallow Concentrated Flow**



**Figure 402-15 Velocity Versus Slope for Shallow Concentrated Flow**

Velocity = 3.0 ft/sec

2,268 ft / (3 ft/sec) = 756 sec (0.21 hrs)

$T_{\text{shallow}} = 0.21 \text{ hrs}$

**Calculate Channel Flow Contribution to Tc**

Channel Length = 6700 ft

Channel Characteristics:

Slope: 0.077 ft/ft (very steep!)

Manning's 'n': 0.055

Shape: Trapezoid

Bottom Width: 30 ft

Side Slope: 15:1 Horizontal : Vertical

**1. Estimate 1<sup>st</sup> Iteration Average Channel Velocity for Channel Reach**

A discharge is needed to estimate flow velocity using Manning's Equation:

Assume 1 cfs per acre for 100-yr discharge = 1047 cfs

Calculate peak discharge for average velocity (2/3 Peak Q) through the channel flow reach

$$Q = 1047 \text{ cfs} \times 2/3 = 698 \text{ cfs}$$

For calculating the Tc with the first iteration.

$$T_{\text{Csheet}} = 0.11 \text{ hrs}$$

$$T_{\text{Cshallow}} = 0.21 \text{ hrs}$$

Estimating Channel - Tc<sub>Channel</sub> = using discharge values above,

Using a Manning's Calculator =

The screenshot shows the ManningSolver software interface. The title bar is 'ManningSolver - Untitled'. The menu bar includes File, Edit, Options, Channel Type, Output, and Help. The toolbar has icons for New, Open, Save, Print, Graph, Table, and Section. The main window is titled 'Trapezoidal Channel' and is divided into two panes: 'Input Data' and 'Results'.

Input Data	
Flow	698 cfs
Slope	0.077 ft/ft
Manning's n	0.055
Base Width	30 ft
Side Slope Rt. (H:1)	15
Side Slope Lt. (H:1)	15

Results	
Depth	1.606 ft
Flow Area	86.9 sf
Velocity	8.04 fps
Velocity Head	1.00 ft
Top Width	78.2 ft
Froude No.	1.34
Critical Depth	1.879 ft
Critical Slope	0.0408 ft/ft

Flow Velocity = 8.04 ft/sec, Froude no. = 1.34

$$T_{\text{Channel}} = 6700 \text{ ft} / 8.04 \text{ ft/sec} = 833 \text{ sec} = 0.23 \text{ hrs}$$

$$T_{\text{Total}} = 0.11 \text{ hrs} + 0.21 \text{ hrs} + 0.23 \text{ hrs} = 0.55 \text{ hrs}$$

$$\text{Basin Lag} = T_c \times 0.6 = 0.33 \text{ hrs} = 19.8 \text{ minutes}$$

Using this value in HEC-HMS

Summary Results for Subbasin "Subbasin-1"

Project: Iterative\_method Simulation Run: Run 1  
Subbasin: Subbasin-1

Start of Run: 01Jul2017, 00:00 Basin Model: Basin 1  
End of Run: 02Jul2017, 12:00 Meteorologic Model: 100 yr  
Compute Time: 15Mar2017, 15:09:47 Control Specifications: 24hr

Volume Units: ☒ IN ☐ AC-FT

Computed Results

Peak Discharge:	1526.7 (CFS)	Date/Time of Peak Discharge:	01Jul2017, 06:30
Precipitation Volume:	3.49 (IN)	Direct Runoff Volume:	1.85 (IN)
Loss Volume:	1.64 (IN)	Baseflow Volume:	0.00 (IN)
Excess Volume:	1.85 (IN)	Discharge Volume:	1.85 (IN)

HEC-HMS results = 1526 cfs

## 2. Calculate the 2<sup>nd</sup> Iteration using the new peak discharge estimate:

$$1526 \text{ cfs} \times 2/3 = 1017 \text{ cfs}$$

ManningSolver - Untitled

File Edit Options Channel Type Output Help

New Open Save Print Graph Table Section

Trapezoidal Channel

Input Data		Results	
Flow	1017 cfs	Depth	1.935 ft
Slope	0.077 ft/ft	Flow Area	114 sf
Manning's n	0.055	Velocity	8.91 fps
Base Width	30 ft	Velocity Head	1.23 ft
Side Slope Rt. (H:1)	15	Top Width	88.0 ft
Side Slope Lt. (H:1)	15	Froude No.	1.38
		Critical Depth	2.285 ft
		Critical Slope	0.0387 ft/ft

Flow Velocity = 8.91 ft/sec, Froude no. = 1.38

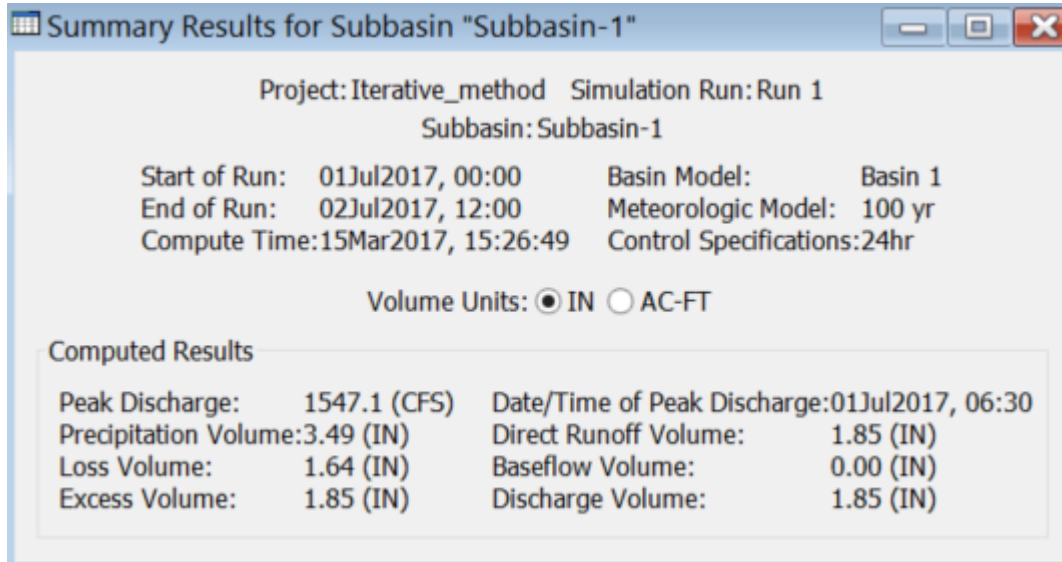
$$T_{\text{channel}} = 6700 \text{ ft} / 8.91 \text{ ft/sec} = 752 \text{ sec} = 0.21 \text{ hrs}$$

$$T_{\text{Total}} = 0.11 \text{ hrs} + 0.21 \text{ hrs} + 0.21 \text{ hrs} = 0.53 \text{ hrs}$$

$$\text{Basin Lag} = T_c \times 0.6 = 0.318 \text{ hrs} = 19.1 \text{ minutes}$$

Using this value in HEC-HMS





Project: Iterative\_method Simulation Run: Run 1  
Subbasin: Subbasin-1

Start of Run: 01Jul2017, 00:00 Basin Model: Basin 1  
End of Run: 02Jul2017, 12:00 Meteorologic Model: 100 yr  
Compute Time: 15Mar2017, 15:26:49 Control Specifications: 24hr

Volume Units: ☒ IN ☐ AC-FT

Computed Results

Peak Discharge:	1547.1 (CFS)	Date/Time of Peak Discharge:	01Jul2017, 06:30
Precipitation Volume:	3.49 (IN)	Direct Runoff Volume:	1.85 (IN)
Loss Volume:	1.64 (IN)	Baseflow Volume:	0.00 (IN)
Excess Volume:	1.85 (IN)	Discharge Volume:	1.85 (IN)

Results in 1547 cfs

Estimated Q = 1526 cfs

Calculated Q = 1547 cfs

$1547 \text{ cfs} - 1526 \text{ cfs} = 21 \text{ cfs} / 1547 \text{ cfs} = 1.3\% \text{ difference.}$

Calculated 100-yr 24-hr storm discharge = 1547 cfs

**Example Problem 6-6 (HEC-HMS Model – NRCS Unit Hydrograph)**

**Note:** The equation, figure, and table numbers referenced with 3 digits (ex. Table 403-1) refer to the same item numbers identified within the Drainage Design Manual.

**Project Information**

Location: I-25 Arroyo crossing South of Lemitar, New Mexico (MP 154.6)

Elevation: 5,073 ft

Watershed: Arid mountain range and desert escarpment

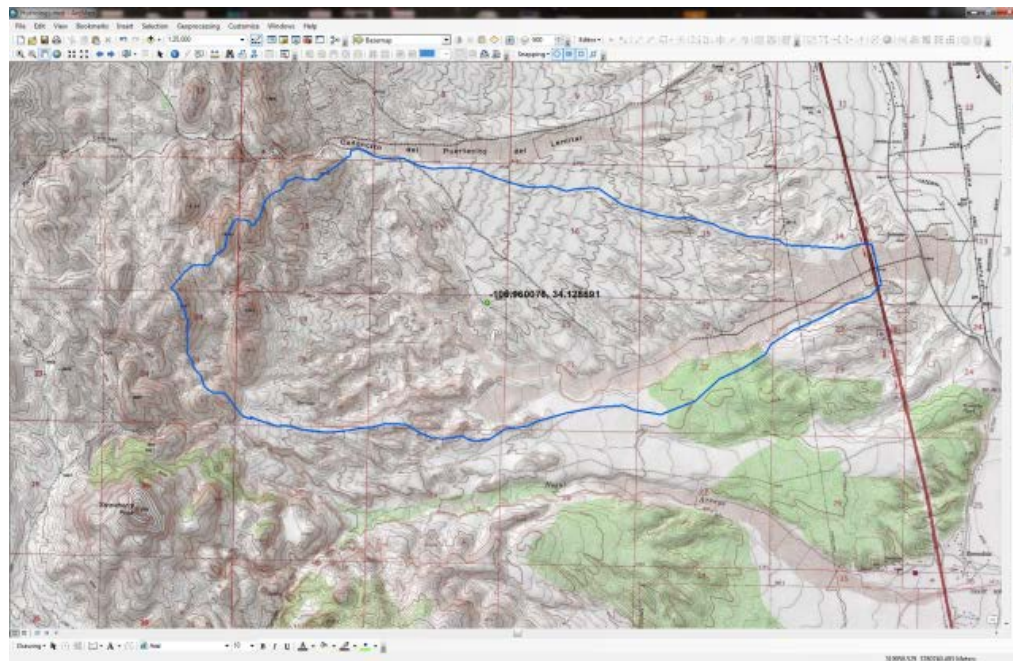
Design Frequency Flood: 50-yr

Check Flood: 100-yr

Watershed area: 4,856 acres (7.6 mi<sup>2</sup>)

**1. Delineate the watershed boundary & Locate the Centroid**

Develop the contributing area to the project site using USGS 1:24,000 quad sheets, see **Figure 1** below.



**Figure 1 Basin Delineation**

2. Determine the centroid of the basin (Figure 2) using a GIS or Drafting tool.

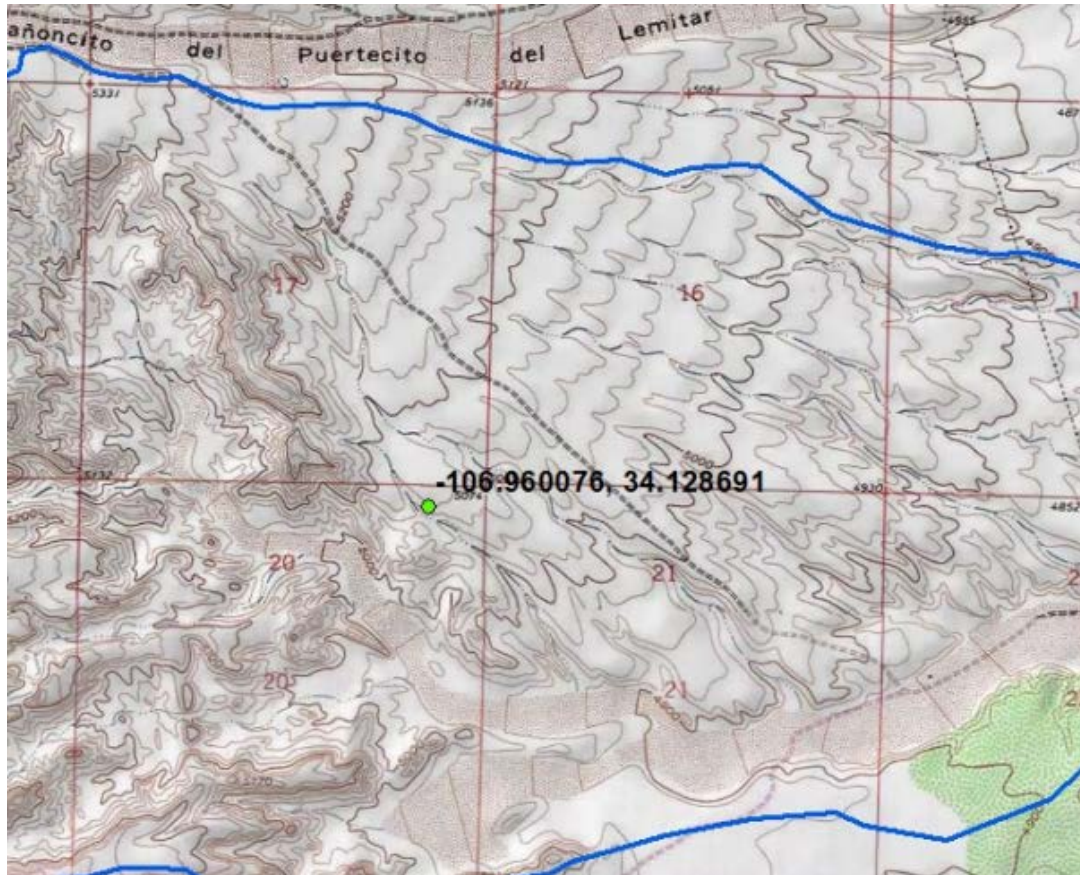


Figure 2 Centroid Location

### 3. Determine the rainfall depth and temporal distribution for the watershed

Using the centroid found for the watershed, determine rainfall depths for the 50-yr and 100-yr 24-hr storm events using the NOAA PFDS website (**Figure 3**).

**NOAA ATLAS 14 POINT PRECIPITATION FREQUENCY ESTIMATES: NM**

**DATA DESCRIPTION**

Data type: precipitation depth Units: english Time series type: partial duration

**SELECT LOCATION**

1. Manually:

a) Enter location (decimal degrees, use "N" for S and W): latitude: 34.128691 longitude: -106.960070 submit

b) Select station (click here for a list of stations used in frequency analysis for NM): select station

2. Use map:

a) Select location (move crosshair or double click)

b) Click on station icon (show stations on map)

**LOCATION INFORMATION:**

Name: Laramie, New Mexico, US  
Latitude: 34.1287  
Longitude: -106.9601  
Elevation: 5073 ft

**Figure 3 NOAA Website**

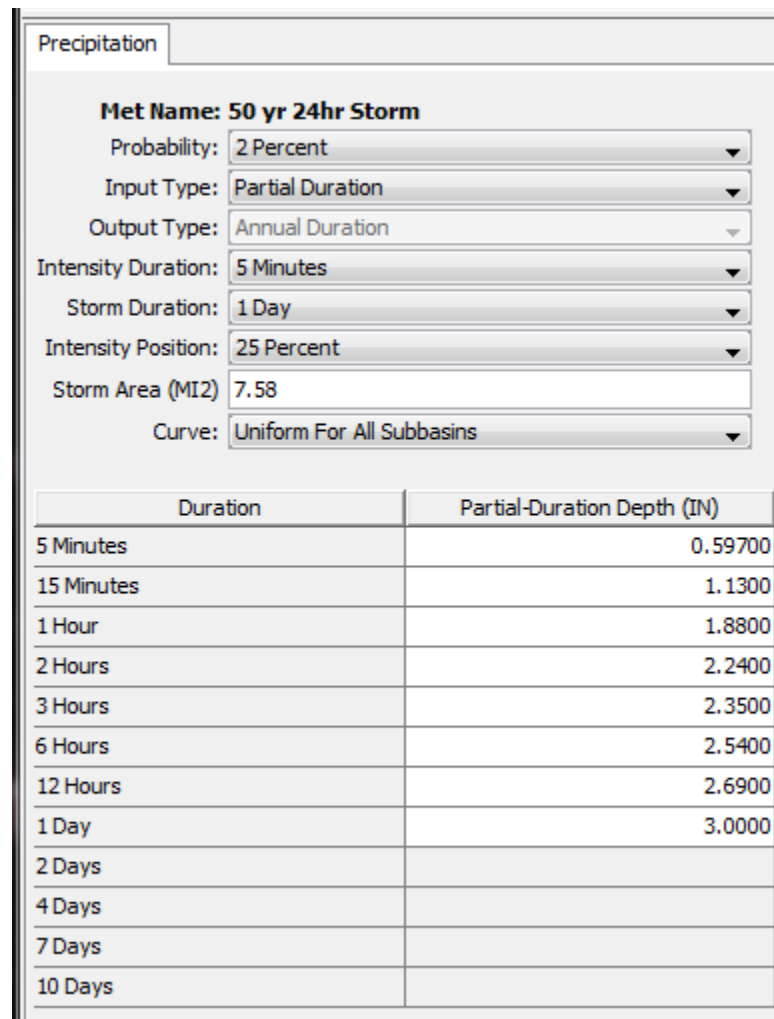
From the NOAA Data Server, create a hard copy record of the output (preferably a .pdf document) and record the 50-yr and 100-yr 24-hr rainfall depths.

#### 4. Create HEC-HMS Project

Open the HEC-HMS program and start a project with U.S. Customary units.

#### 5. Create a Meteorological model.

Create a Meteorological model is illustrated in **Figure 4** for the two project storms and input the data from the NOAA Precipitation Frequency Data Server.



Duration	Partial-Duration Depth (IN)
5 Minutes	0.59700
15 Minutes	1.1300
1 Hour	1.8800
2 Hours	2.2400
3 Hours	2.3500
6 Hours	2.5400
12 Hours	2.6900
1 Day	3.0000
2 Days	
4 Days	
7 Days	
10 Days	

**Figure 4 HEC-HMS Precipitation Input**



## 6. Delineate subbasins based on soils and topography

Define an AOI (Area of Interest) at the Web Soil Survey (**Figure 5**) using the basin delineated (this can be input manually or a shapefile can be uploaded).

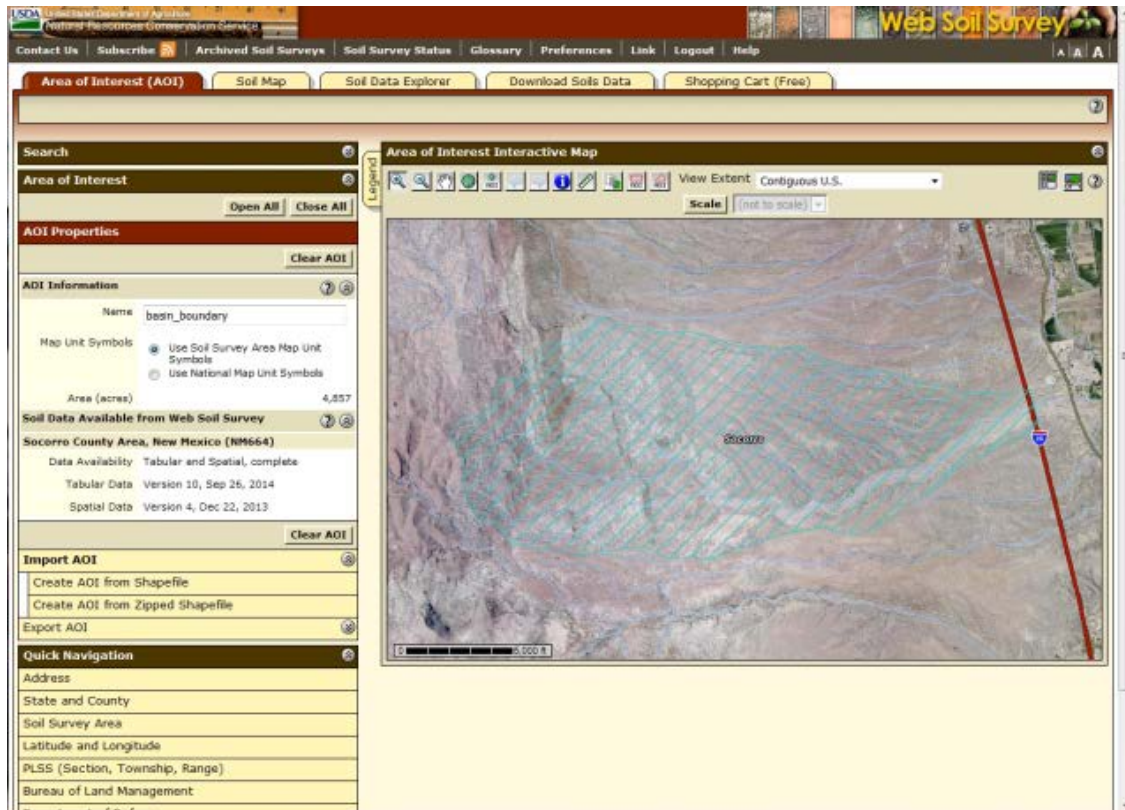
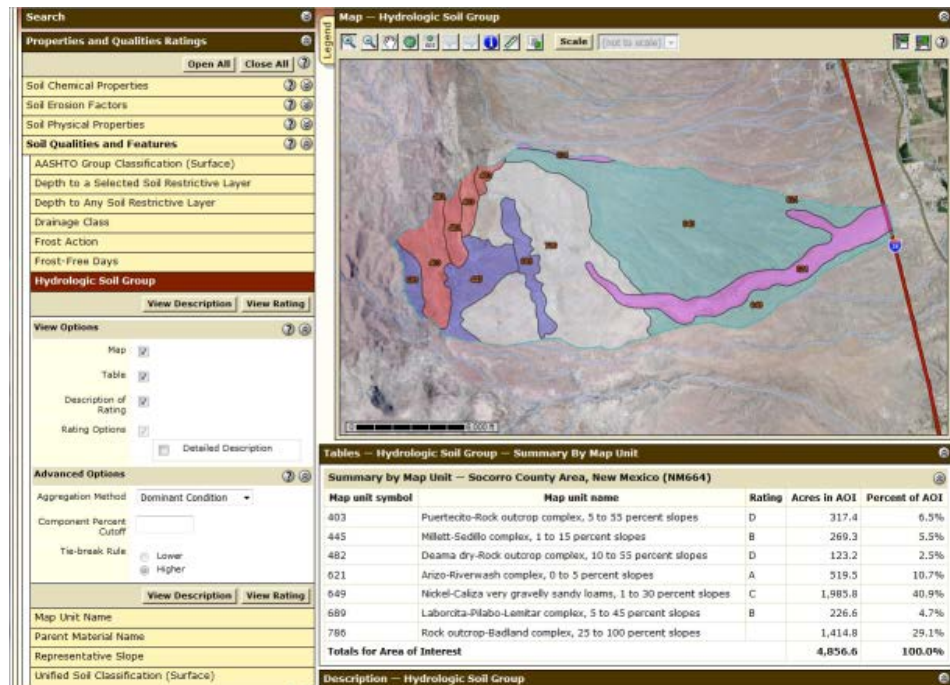


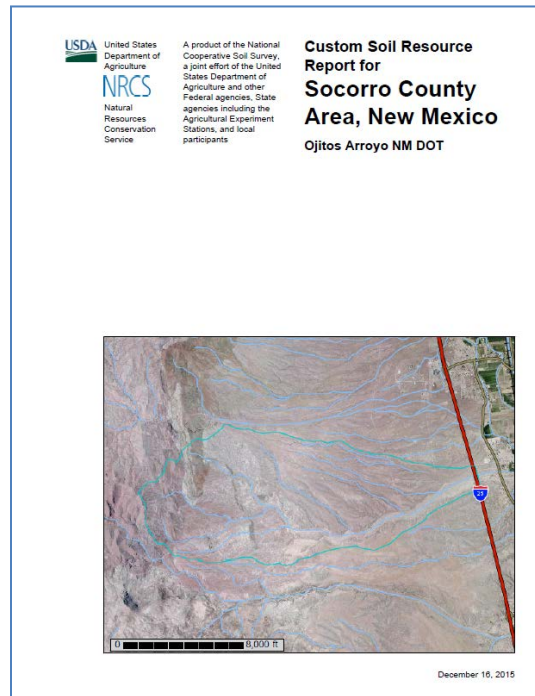
Figure 5 Defining AOI

## 7. Generate a report for the soils Group using the AOI is defined.

Click on the Soil Data Explorer tab and select under “Soil Qualities and Features” select Hydrologic Soil Group, click on “View Ratings” (**Figure 6**) and produce a custom pdf document (**Figure 7**), this can be part of the supplemental information provided in the reports.



**Figure 6 Hydrologic Soils Group Ratings**



**Figure 7 Custom Soils Report**

8. **Assign a Curve Number based on the land use for each specific soil group and Table 402-5 “Runoff Curve Numbers for Arid and Semiarid Rangelands”.**

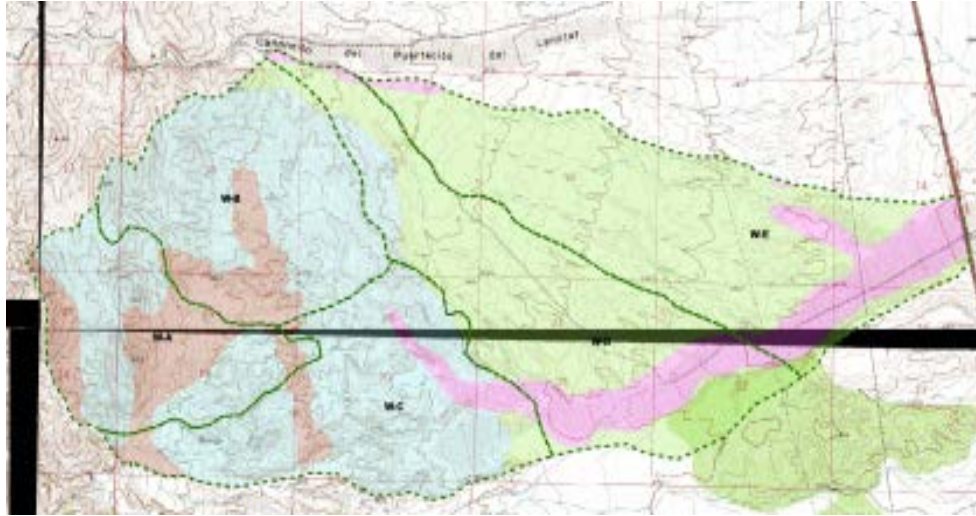
Take the table of Hydrologic Soil Group percentages from the Soils report. Based on the vegetative cover observed from aerial photography in the watershed, use the values for “Desert shrub” in “Fair” condition.

**Tabular Soils Data**

<b>Soil Group Unit Symbol</b>	<b>Area (acres)</b>	<b>Percent</b>	<b>Hyd Group</b>	<b>CN used</b>
403	317.4	6.5%	D	86
445	269.3	5.5%	B	72
482	123.2	2.5%	D	86
621	519.5	10.7%	A	55
649	1985.8	40.9%	C	81
689	226.6	4.7%	B	72
786	1414.8	29.1%	D	86

Subdivide the watershed based on topography and major soil groups as shown in **Figure 8**.





**Figure 8 Subdividing Watershed**

Weighted CN Values

Weighted subbasin CN values were computed and range from 72 to 70. The subbasin CN weighting computations are not presented here.

**9. Determine the longest flow path using the methods described in Section 402.8**

Calculate the time of concentration for each of the sub basins as well as the basin lag for each. See **Figure 9** and **Figure 10** below.



**Figure 9 Longest Flow Path for Each Basin**

Watershed Name	W-A	W-B	W-C	W-D	W-E
<b>Sheet Flow Characteristics</b>					
Manning's Roughness Coefficient	0.13	0.13	0.13	0.15	0.15
Flow Length (ft)	300	300	300	300	300
Two-Year 24-hour Rainfall (in)	1.5	1.5	1.5	1.5	1.5
Land Slope (ft/ft)	0.321	0.52	0.197	0.0338	0.045
<b>Sheet Flow Tt (hr)</b>	<b>0.17</b>	<b>0.14</b>	<b>0.21</b>	<b>0.47</b>	<b>0.42</b>
<b>Shallow Concentrated Flow Characteristics</b>					
Surface Description (1 - unpaved, 2 - paved)	1	1	1	1	1
Flow Length (ft)	870	2270	590	3510	3470
Watercourse Slope (ft/ft)	0.14	0.14	0.24	0.03	0.04
Average Velocity - computed (ft/s)	5.99	6.06	7.89	2.84	3.10
<b>Shallow Concentrated Flow Tt (hr)</b>	<b>0.04</b>	<b>0.10</b>	<b>0.02</b>	<b>0.34</b>	<b>0.31</b>
<b>Channel Flow Characteristics</b>					
Cross-sectional Flow Area (ft <sup>2</sup> )	10	10	10	10	10
Wetted Perimeter (ft)	9	15	10	10	10
Hydraulic Radius - computed (ft)	1.11	0.67	1.00	1.00	1.00
Channel Slope (ft/ft)	0.040	0.088	0.066	0.034	0.027
Manning's Roughness Coefficient	0.035	0.035	0.035	0.035	0.035
Average Velocity - computed (ft/s)	9.15	9.61	10.90	7.88	6.97
Flow Length (ft)	12950	8680	10060	15750	16420
<b>Channel Flow Tt (hr)</b>	<b>0.39</b>	<b>0.25</b>	<b>0.26</b>	<b>0.55</b>	<b>0.65</b>
<b>Watershed Time of travel (hr)</b>	<b>0.60</b>	<b>0.49</b>	<b>0.48</b>	<b>1.36</b>	<b>1.38</b>
Basin Lag (min) .6 * Tc	<b>22</b>	<b>18</b>	<b>17</b>	<b>49</b>	<b>50</b>

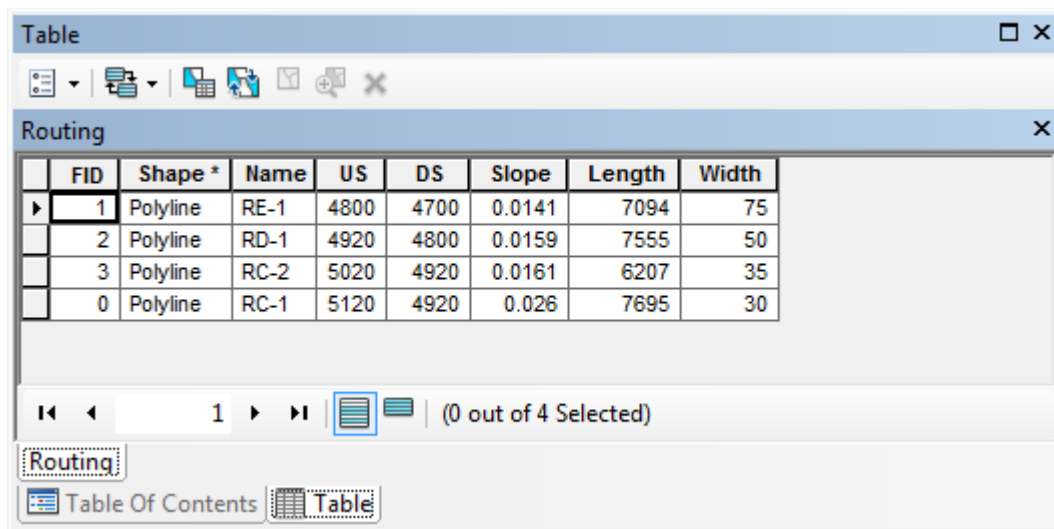
Figure 10 Table Calculating Tc and Basin Lag

## 10. Determine Routing Lengths

Measure and determine slopes of each routing length used in the watershed (see Figure 11).



Figure 11 Routed Reaches Lengths



FID	Shape *	Name	US	DS	Slope	Length	Width
1	Polyline	RE-1	4800	4700	0.0141	7094	75
2	Polyline	RD-1	4920	4800	0.0159	7555	50
3	Polyline	RC-2	5020	4920	0.0161	6207	35
0	Polyline	RC-1	5120	4920	0.026	7695	30

Figure 12 Routed Reach Measurements

## 11. Create HEC-HMS Basin Model

Using the information found above, build a HEC-HMS basin model, see **Figure 13** below.

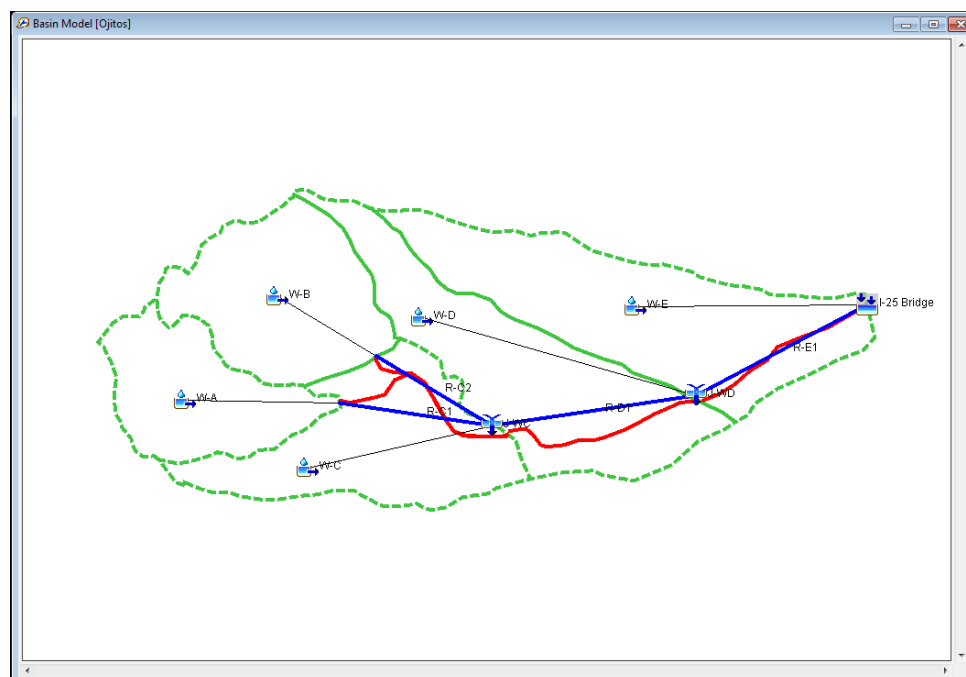


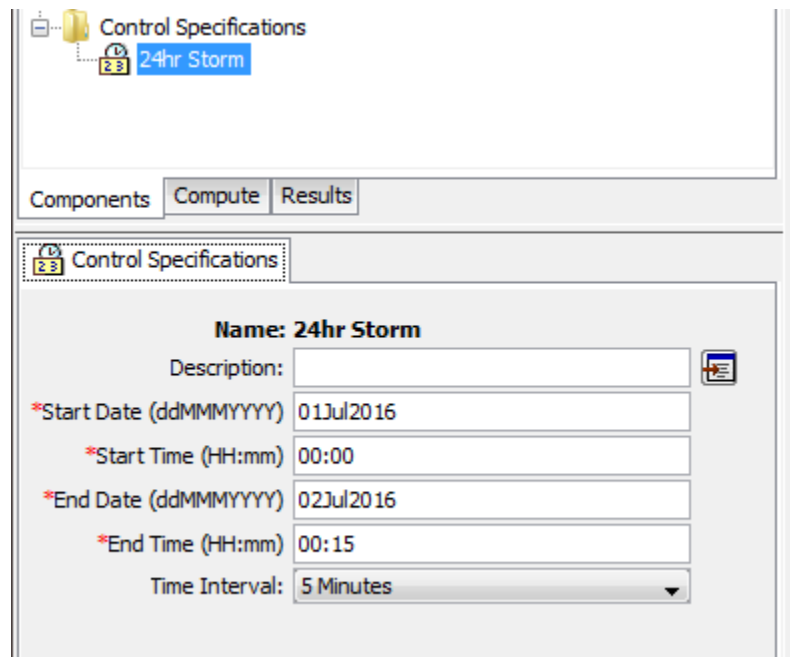
Figure 13 HEC-HMS Basin Model

## 12. Setup Control Specifications for the HEC-HMS model

Using the formula from **Section 405.9** calculate the time step for the model

$$\Delta t = 0.29 \times 0.60 T_c = 0.17 T_c.$$

$$\Delta t = 0.17 (0.48 \text{ hrs}) \times 60 \text{ min/hr} = 4.89 \text{ minutes} = 5 \text{ minutes, see Figure 14.}$$

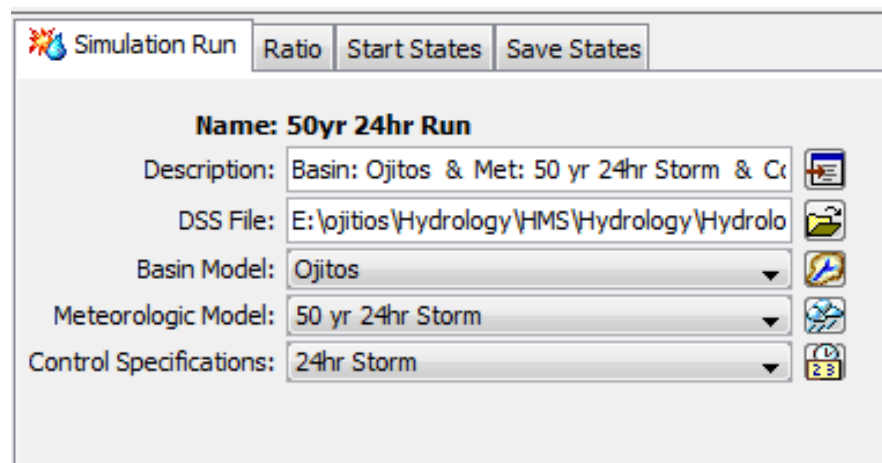


The screenshot shows the 'Control Specifications' dialog box in HEC-HMS. The 'Name' field is set to '24hr Storm'. The 'Description' field is empty. The 'Start Date' is '01Jul2016', 'Start Time' is '00:00', 'End Date' is '02Jul2016', and 'End Time' is '00:15'. The 'Time Interval' is set to '5 Minutes'.

Figure 14 Control Specifications Inputs

## 13. Create a Hydrologic Simulation

Create a compute run using the 3 parts of the model input, see **Figure 15**.



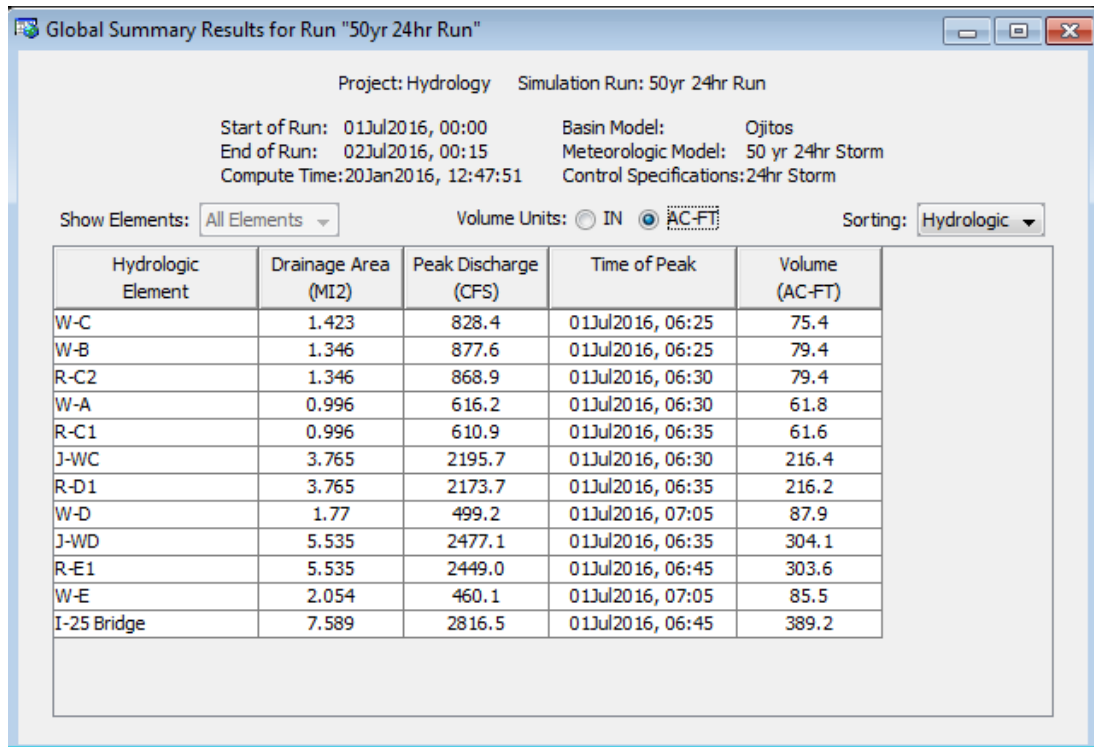
The screenshot shows the 'Simulation Run' dialog box in HEC-HMS. The 'Name' field is set to '50yr 24hr Run'. The 'Description' field is 'Basin: Ojitos & Met: 50 yr 24hr Storm & C'. The 'DSS File' is 'E:\ojitos\Hydrology\HMS\Hydrology\Hydrolo'. The 'Basin Model' is 'Ojitos', the 'Meteorologic Model' is '50 yr 24hr Storm', and the 'Control Specifications' is '24hr Storm'.

Figure 15 Simulation Inputs



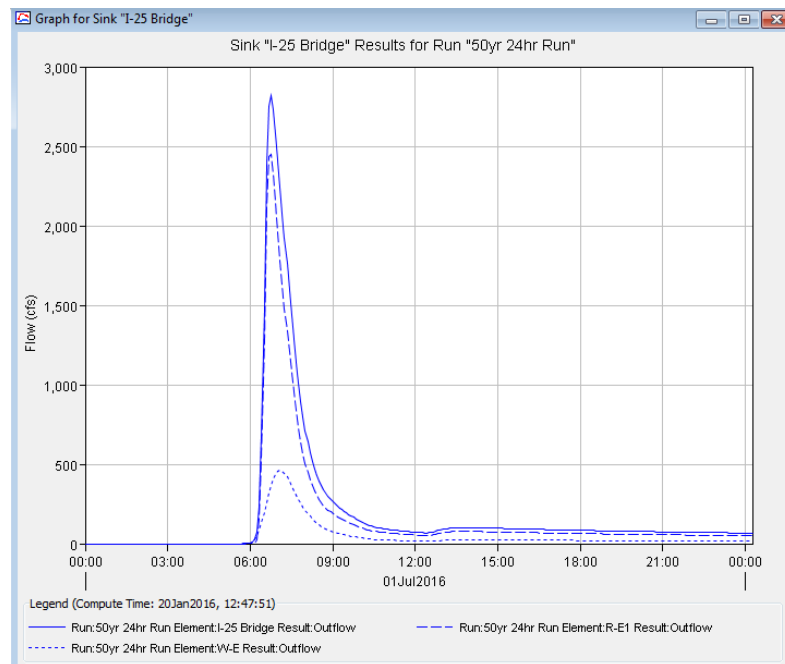
## 14. Model Summary and Output

Within the Results tab of the model, View the results of the Global Summary (being sure to use the “AC-FT” radio button on the results), see **Figure 16**.



**Figure 16 HEC-HMS Global Output**

Open the graph output at the study location to determine the outflow hydrograph shape, see **Figure 17**.



**Figure 17 Outflow Hydrograph**

**15. Solve for discharge per acre:**

$$50\text{-yr} = 2816 \text{ cfs} / 4,856 \text{ acres} = \mathbf{0.579 \text{ cfs/ac}}$$

$$100\text{-yr} = 3700 \text{ cfs} / 4,856 \text{ acres} = \mathbf{0.761 \text{ cfs/ac}}$$

**16. Complete the worksheet below for HEC-HMS.**

**17. Check the HEC-HMS results compared to another peak discharge estimation method, and/or compare the results based on experience based guideline values for reasonableness.**

Note that **Example Problem 6-3** applied the NRCS Simplified Peak Discharge Method for the same basin as for this HEC-HMS Method (**Example Problem 6-6**). The following Table contains the results from each method, for this basin, the results from both methods are reasonably close.

Method	50-year Peak Discharge	100-year Peak Discharge
HEC-HMS	2,816	3,700
NRCS Simplified Peak Discharge	2,859	3,507

**HEC-HMS Method Worksheet**

Structure Location: MP: 154.6 County: Socorro  
 District: Deming  
 Structure Description: I-25 North and Southbound Lanes  
 Drainage Area: **A** = 4,856 acres 7.6 mi<sup>2</sup>

**Meteorological Model Summary**

Elevation at Centroid of Watershed: **Elev** = 5,073 ft \*  
 Location of Centroid: **Lat:** 34.1287° **Long:** -106.9601°

<b>Design Frequency Flood</b>	<u>50</u> - year	<u>100</u> - year
24-hour Rainfall Depth (NOAA PFDS):	<b>P<sub>24</sub></b> <u>3.0</u> in.	<b>P<sub>24</sub></b> = <u>3.4</u> in.

**Basin Model Summary**

Number of Sub-Basins 5  
 Curve Number Range Used for modeling Low: 72 High: 79  
 Basin Lag Range Used for modeling Low: 17 min High: 50 min

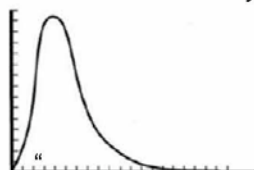
**Control Specifications Summary**

Total Model Duration 24:05 Hrs:Min Time Interval 5 min (min\hrs)

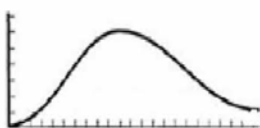
**Summary Output** (at Structure Location)

<b>Design Frequency Flood</b>	<u>50</u> - year	<u>100</u> - year
Peak Discharge (cfs)	<b>Q</b> = <u>2816</u> cfs	<b>Q</b> = <u>3700</u> cfs
Discharge per acre	<u>0.579</u> cfs/ac	<u>0.761</u> cfs/ac
Total Volume (ac-ft)	<b>V</b> = <u>389</u> ac-ft	<b>V</b> = <u>498</u> ac-ft
Total Runoff (in)	<b>V</b> = <u>0.96</u> in	<b>V</b> = <u>1.23</u> in

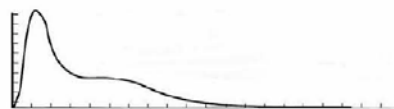
Approximate Outflow Hydrograph Shape:



☒ "Peaky"



☐ "Broad"



☐ "Mixed"

Project Location: I-25, MP 154.6  
 CN#: 79  
 Date: 1/20/16  
 Computed By: KCE  
 Checked By: CME

\* If elevation is greater than 7500 ft, use NRCS Unit Hydrograph method

**Example Problem 6-7 (Hatch Site 6 Hydrology – Runoff Determination Discussion)**

This problem is based on a technical paper titled “Hatch Site 6 Runoff Methods Revisited”, by Charles M. Easterling, PE, May 21, 2004.

**Runoff Determination Discussion:**Background

Hatch Site 6 is located in southern New Mexico, near the community of Garfield and was constructed in 1956 under Public Law 566 as a floodwater detention and sediment control structure. The dam and appurtenances are being studied for possible rehabilitation as the structure is nearing its 50 year design life.

Problem Statement

Upon review of the initial evaluation hydrology it was determined that the values of runoff volume and peak rate were unrealistically high. The hydrology was based on initial estimates of CN and channel transmission loss values as presented in **Table 1**.

**TABLE 1 CURVE NUMBER SUMMARY**

Sub-basin	Area (mi. <sup>2</sup> )	CN		COMPOSITE WEIGHT (AxCN)		COMPOSITE CN	
		Initial	Adjusted	Initial	Adjusted	Initial	Adjusted
<b>1</b>	2.41	85	81	205.04	195.39		
<b>2</b>	1.18	88	85	103.49	99.96		
<b>3</b>	0.45	77	77	34.82	34.82	85.0	81.7
<b>4</b>	0.12	63	63	7.48	7.48		
<b>Total at Dam</b>	4.16			350.82	337.65	<b>84.4</b>	<b>81.2</b>
<b>5</b>	0.21	77	77	16.48	16.48	84.0	81.0

When discussing possible alternatives (using the initial curve number and transmission losses) the sponsors expressed their alarm in the strongest terms and stated that based on 50 years of experience at Hatch Site 6, the results didn't make sense to them. A re-review of the data and methods was conducted to identify any potential sources of error in the methods used and to provide a course of action if the review indicated that the calculation of curve numbers and channel transmission losses should be revisited. The auxiliary spillway has not operated since the dam was constructed in 1956. The dam was initially designed to contain a 50 year storm preceded by a 25 year storm. (The probability of a storm equaling or exceeding the 50 year event since the structure was constructed is roughly  $(J = 1 - (1 - P)^N)$  or 65%). While there are no rain gages in the watershed, the gage at nearby Hatch measured a rainfall event of 3.46 in 1987, which is greater than the 100 year 24 hr storm volume (3.27 in). Anecdotally, long time



local residents have reported several rainfalls events in the range of the 50 to 100 year 24 hour storm in and nearby to the watershed.

### Runoff Curve Number

Hydrologic Group designations found in the published soils reports and field observation of the cover conditions in the watershed were initially used to determine the Runoff Curve Numbers for hydrologic evaluation and to generate alternatives for the rehabilitation of Hatch Site 6.

A review of the Sierra County Area and Doña Ana County Area Soil Survey Reports offer a somewhat different interpretation of the expected response of the soils in the Hatch Site 6 watershed from that found by using Group D. Both reports classify the soils in the upper 2/3 of the watershed as “Rock Outcrop”. The Sierra County Area report offers little in the way of interpretation or associations with other soils. The Doña Ana County Area report however provides soil associations, specifically “Courthouse and Torriorthents” which have interpretations as well as a listing of their engineering properties.

A review of this additional information and field examination of the soils suggests a different interpretation from that which assumes a universal classification of these soils into Hydrologic Group D. While the report accurately describes the soils as rock and cobble covering a high percentage of the ground surface, it appears from review of the more detailed association descriptions and from field and historic observation, these soils will not behave like rock outcrop, “D” soils, but rather as they are described as “gravely fine sandy loam” in the case of Courthouse and “gravelly loam and gravelly sandy clay loam” in the case of Torriorthents. Based on these engineering characteristics, these soils should exhibit a slower runoff response than would be otherwise expected from “D” soils. The soils are described and behave more similarly to other Hydrologic Group C soils in the area than they do Group D soils. Using Group C reduces the initial composite Runoff Curve Number of 85 to 81. (See attached weighted curve number calculation spreadsheet.) It should be noted that the Runoff Curve Number method of determining direct runoff assumes average conditions in the watershed.

At the time of the field examination of the cover conditions, the watershed had been severely impacted by several years of drought and thus the observed poor cover conditions may not have been representative of an average condition. While a fair condition may be more representative of the long term average range condition, based on recent research by the ARS in Arizona, changes in cover in the Sonoran Desert shrub environment make little difference in runoff. Therefore, there was little justification to alter the initial assessment and it was left at poor. This may add a small degree of conservatism to the final results.

### Transmission Losses

Channel transmission losses in the watershed were determined to be another potential source of over-prediction in the methodology used in the initial evaluation hydrology. It was initially believed that a detailed, reach by reach assessment of the channel losses (as found in NEH 4, Chapter 19) would result in the most accurate estimate of channel loss. Upon review and examination however, it was found that insufficient data is available to perform a credible evaluation using that approach. As a result, the methods outlined in NEH-4, Chapter 21, was deemed to be more appropriate resulting in a channel-loss factor of 0.755.

Application of the two changes described above results in the 50 year 24 hr storm event being stored at an elevation very near the existing auxiliary crest elevation which to some degree

corroborates the revised approach. The 100-year, 10-day storm and the PMF are still the governing storms for design of the auxiliary spillway and dam.

#### Summary

It is recommended that an “81” Curve Number and a transmission loss factor of 0.755 be used in the planning of the project. Use of these values results in a closer match to the history of existing structure and to the values used in the planning of the original site.

# APPENDIX 7 EXAMPLE HYDRAULIC CALCULATIONS

## **CONTENTS:**

Example Problem 7-1	HY-8 Program - Culvert Hydraulic Analysis
Example Problem 7-2	FlowMaster Program - Hydraulic Analysis for: Street and Curb Drop Inlet Flow Capacity Computations
Example Problem 7-3	Detention Pond Example Tables

**Example Problem 7-1 (HY-8 Program - Culvert Hydraulic Analysis)**

**Note:** The equation, figure, and table numbers referenced with 3 digits (ex. Table 403-1) refer to the same item numbers identified within the Drainage Design Manual.

Project Information

Road Type: Four lane principal arterial

50-year peak discharge (unbulked) = 83.3 cfs

100-year peak discharge (unbulked) = 112.5 cfs

Culvert Hydraulic Analysis Assumptions

Inlet Configuration: Assume a square edge with headwall

Material: Assume RCP Manning's "n" value = 0.013 (urban area upstream, very little sediment)

Initially propose 1 new 36" RCP, or 1 new 48" RCP, size depending on the culvert analysis results compared to the drainage criteria

Tailwater Assumption: The culvert outlet invert elevation equals the channel bed elevation at outlet, bed slope is 1 %, trapezoidal channel, 10 ft bottom width, 1V:2H side slopes, Manning's "n" estimated at 0.026 for bed and banks

Roadway Assumptions: Refer to **Figure 1** for the basic culvert and roadway assumptions

Drainage Criteria: Summarized in **Table 1**, new culvert installation

**Table 1 - Criteria for Storm Frequency and Hydraulics**

	Design Flood	Check Flood	Drainage Criteria Table (a)
Storm Frequency	50-yr	100-yr	203-2
Hydraulic Criterion	Ratio of headwater (HW) depth to culvert rise shall not exceed 1.5 and limit headwater to edge of shoulder	Limit headwater (HW) spread to 1 driving lane	204-1, 204-2, 205-1
Clogging Factor	Urban and Rural - Assume a 20% bulking and debris factor		
a - Tables are located in Section 200 of the NMDOT Drainage Design Manual			

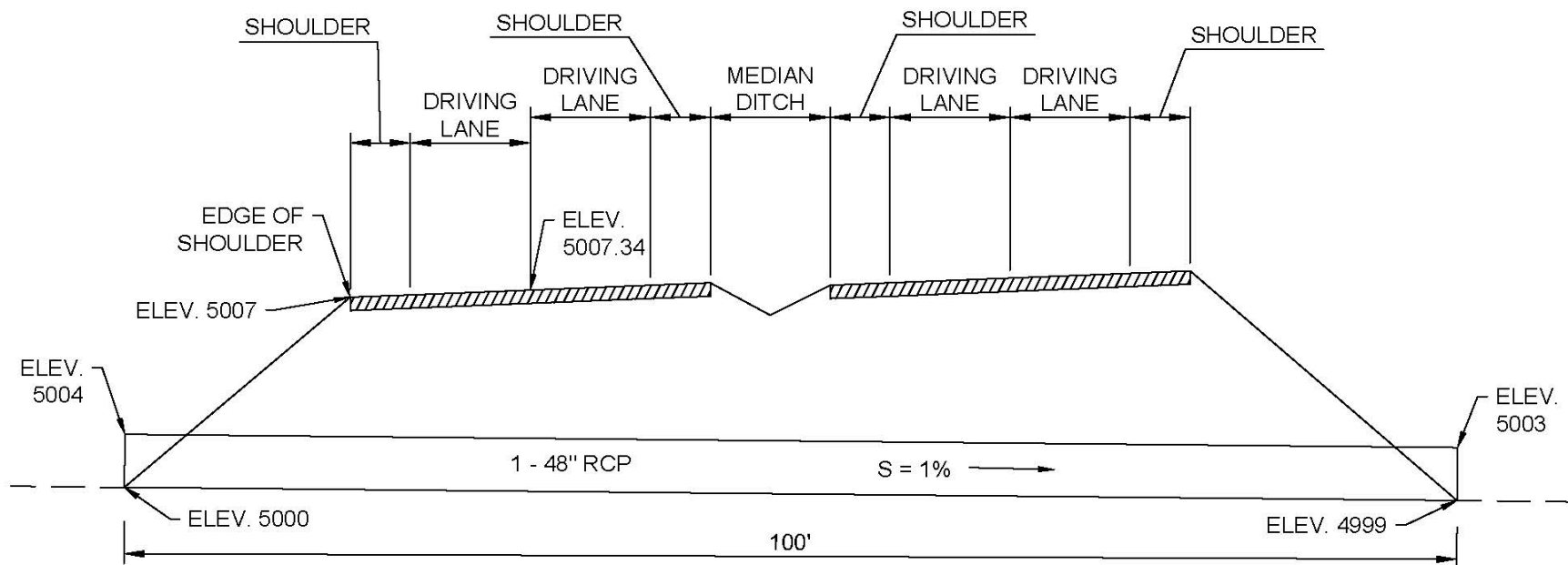
HY-8 Culvert Size Trials -

Trial 1 – Try 1-36" RCP for the Design Flood, if this fails to meet design criteria, then

Trial 2 – Try 1-48" RCP for the Design Flood, if this meets design criteria, then

Trial 3 – Try 1-48" RCP for the Check Flood, if this meets design criteria, then culvert hydraulic sizing is complete.

See **Figure 1** for the proposed culvert and roadway geometry and elevations.



**Figure 1 Culvert and Roadway Profile Schematic**

NOT TO SCALE

a - Apply a 20% sediment and clogging factor to the unbulked peak discharge per NMDOT Drainage Design Manual Criteria Table 205-1

**Conclusion**

1 – 48" RCP is recommended because it meets both Design Flood and Check Flood criteria.

**Sediment Bulking and Clogging Factor Modified Computation**

If a sediment bulking factor was applied within a hydrologic model of say 12%, then an additional 8% factor is required to attain a 20% bulking factor. The following procedure may be applied to compute or bulk, the total design discharge required to attain a 20% bulking/clogging factor.

Assume the 50-year peak discharge = 100 cfs, and that value includes a 12% sediment bulking factor (computed in a HEC-HMS model). Compute the un-bulked peak discharge, then multiply that value by 1.20 as follows:

The un-bulked peak discharge computation =  $100 \text{ cfs} / 1.12 = 89.3 \text{ cfs}$ .

The 20% bulked peak discharge computation =  $89.3 \text{ cfs} * 1.2 = 107 \text{ cfs}$ .

Crossing Data - Ex. Prob 7-1 Des. Flood-Trial 1, 36" RCP

Crossing Properties

Name: 1 Des. Flood-Trial 1, 36" RCP

Parameter	Value	Units
<b>DISCHARGE DATA</b>		
Discharge Method	Minimum, Design, and Maximum	
Minimum Flow	10.000	cfs
Design Flow	83.300	cfs
Maximum Flow	100.000	cfs
<b>TAILWATER DATA</b>		
Channel Type	Trapezoidal Channel	
Bottom Width	10.000	ft
Side Slope (H:V)	2.000	:1
Channel Slope	0.0100	ft/ft
Manning's n (channel)	0.026	
Channel Invert Elevation	4999.000	ft
Rating Curve	View...	
<b>ROADWAY DATA</b>		
Roadway Profile Shape	Constant Roadway Elevation	
First Roadway Station	0.000	ft
Crest Length	38.000	ft
Crest Elevation	5009.960	ft
Roadway Surface	Paved	
Top Width	38.000	ft

Culvert Properties

36" RCP

Add Culvert  
Duplicate Culvert  
Delete Culvert

Parameter	Value	Units
<b>CULVERT DATA</b>		
Name	36" RCP	
Shape	Circular	
Material	Concrete	
Diameter	3.000	ft
Embedment Depth	0.000	in
Manning's n	0.013	
Culvert Type	Straight	
Inlet Configuration	Square Edge with Headwall	
Inlet Depression?	No	
<b>SITE DATA</b>		
Site Data Input Option	Culvert Invert Data	
Inlet Station	0.000	ft
Inlet Elevation	5000.000	ft
Outlet Station	100.000	ft
Outlet Elevation	4999.000	ft
Number of Barrels	1	

Help Click on any icon for help on a specific topic

Low Flow AOP Energy Dissipation Analyze Crossing OK Cancel

See Table 3 for output

**Figure 2 Ex. Problem 7-1, HY-8 Input Design Flood-Trial 1, 36" RCP**

Table 3 Ex. Problem 7-1 Design Flood-Trial 1, 36" RCP HY-8 Analysis Results																	
Customized Table																	
Culvert Crossing: Ex. Problem 7-1 Design Flood-Trial 1, 36" RCP																	
Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)	Length Full (ft)	Length Free (ft)	Crest Control Elev (ft)	Face Control Elev (ft)	Throat Control Elev (ft)	Tailwater Elevation (ft)
10	10	5001.37	1.37	0.0*	1-S2n	0.76	1.00	0.79	0.35	6.48	2.69	0.00	100	0.00	0.00	0.00	4999.35
19	19	5002.02	2.02	0.65	1-S2n	1.07	1.40	1.11	0.51	7.73	3.39	0.00	100	0.00	0.00	0.00	4999.51
28	28	5002.57	2.57	1.25	1-S2n	1.32	1.71	1.38	0.64	8.55	3.88	0.00	100	0.00	0.00	0.00	4999.64
37	37	5003.1	3.10	1.92	5-S2n	1.55	1.97	1.63	0.75	9.15	4.28	0.00	100	0.00	0.00	0.00	4999.75
46	46	5003.69	3.69	2.67	5-S2n	1.78	2.21	1.86	0.85	9.66	4.60	0.00	100	0.00	0.00	0.00	4999.85
55	55	5004.39	4.39	3.79	5-S2n	2.02	2.40	2.11	0.95	10.09	4.89	0.00	100	0.00	0.00	0.00	4999.95
64	64	5005.23	5.23	4.61	5-S2n	2.29	2.57	2.37	1.03	10.42	5.14	0.00	100	0.00	0.00	0.00	5000.03
73	73	5006.21	6.21	5.43	7-M2c	3.00	2.70	2.70	1.11	10.91	5.36	0.00	100	0.00	0.00	0.00	5000.11
82	82	5007.33	7.33	6.54	7-M2c	3.00	2.79	2.79	1.19	11.97	5.57	80.27	19.73	0.00	0.00	0.00	5000.19
83	83	5007.5	7.50	6.69	7-M2c	3.00	2.80	2.80	1.20	12.13	5.60	83.51	16.49	0.00	0.00	0.00	5000.2
100	100	5009.97	9.97	0.0*	7-M2c	3.00	2.66	2.66	1.33	15.04	5.93	0.00	100	0.00	0.00	0.00	5000.33



Crossing Data - Ex. Prob 7-1 Des. Flood-Trial 2, 48" RCP

Crossing Properties

Name: 1 Des. Flood-Trial 2, 48" RCP

Parameter	Value	Units
<b>DISCHARGE DATA</b>		
Discharge Method	Minimum, Design, and Maximum	
Minimum Flow	10.000	cfs
Design Flow	83.300	cfs
Maximum Flow	100.000	cfs
<b>TAILWATER DATA</b>		
Channel Type	Trapezoidal Channel	
Bottom Width	10.000	ft
Side Slope (H:V)	2.000	:1
Channel Slope	0.0100	ft/ft
Manning's n (channel)	0.026	
Channel Invert Elevation	4999.000	ft
Rating Curve	View...	
<b>ROADWAY DATA</b>		
Roadway Profile Shape	Constant Roadway Elevation	
First Roadway Station	0.000	ft
Crest Length	38.000	ft
Crest Elevation	5007.000	ft
Roadway Surface	Paved	
Top Width	38.000	ft

Culvert Properties

48" RCP

Add Culvert  
Duplicate Culvert  
Delete Culvert

Parameter	Value	Units
<b>CULVERT DATA</b>		
Name	48" RCP	
Shape	Circular	
Material	Concrete	
Diameter	4.000	ft
Embedment Depth	0.000	in
Manning's n	0.013	
Culvert Type	Straight	
Inlet Configuration	Square Edge with Headwall	
Inlet Depression?	No	
<b>SITE DATA</b>		
Site Data Input Option	Culvert Invert Data	
Inlet Station	0.000	ft
Inlet Elevation	5000.000	ft
Outlet Station	100.000	ft
Outlet Elevation	4999.000	ft
Number of Barrels	1	

Help Click on any icon for help on a specific topic Low Flow AOP Energy Dissipation Analyze Crossing OK Cancel

See Table 4 for output

**Figure 3 Ex. Problem 7-1, HY-8 Input Design Flood-Trial 2, 48" RCP**

Table 4 Ex. Problem 7-1 Design Flood-Trial 2, 48" RCP HY-8 Analysis Results																	
Customized Table																	
Culvert Crossing: Ex. Problem 7-1 Design Flood-Trial 2, 48" RCP																	
Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)	Length Full (ft)	Length Free (ft)	Crest Control Elev (ft)	Face Control Elev (ft)	Throat Control Elev (ft)	Tailwater Elevation (ft)
10	10	5001.25	1.25	0.0*	1-S2n	0.69	0.92	0.72	0.35	6.26	2.69	0.00	100	0.00	0.00	0.00	4999.35
19	19	5001.75	1.75	0.35	1-S2n	0.96	1.28	1.00	0.51	7.50	3.39	0.00	100	0.00	0.00	0.00	4999.51
28	28	5002.19	2.19	0.72	1-S2n	1.16	1.56	1.22	0.64	8.31	3.88	0.00	100	0.00	0.00	0.00	4999.64
37	37	5002.61	2.61	1.08	1-S2n	1.35	1.81	1.42	0.75	8.92	4.28	0.00	100	0.00	0.00	0.00	4999.75
46	46	5002.98	2.98	1.44	1-S2n	1.51	2.03	1.61	0.85	9.38	4.60	0.00	100	0.00	0.00	0.00	4999.85
55	55	5003.33	3.33	1.82	1-S2n	1.67	2.23	1.78	0.95	9.86	4.89	0.00	100	0.00	0.00	0.00	4999.95
64	64	5003.67	3.67	2.21	1-S2n	1.82	2.41	1.95	1.03	10.2	5.14	0.00	100	0.00	0.00	0.00	5000.03
73	73	5004.02	4.02	2.62	5-S2n	1.97	2.58	2.10	1.11	10.55	5.36	0.00	100	0.00	0.00	0.00	5000.11
82	82	5004.38	4.38	3.06	5-S2n	2.11	2.74	2.26	1.19	10.88	5.57	0.00	100	0.00	0.00	0.00	5000.19
83	83	5004.43	4.43	3.12	5-S2n	2.13	2.76	2.28	1.20	10.92	5.60	0.00	100	0.00	0.00	0.00	5000.2
100	100	5005.18	5.18	4.47	5-S2n	2.39	3.03	2.56	1.33	11.45	5.93	0.00	100	0.00	0.00	0.00	5000.33

Crossing Data - Ex. Prob 7-1 Ck. Flood-Trial 3, 48" RCP

**Crossing Properties**

Name: -1 Ck. Flood-Trial 3, 48" RCP

Parameter	Value	Units
<b>DISCHARGE DATA</b>		
Discharge Method	Minimum, Design, and Maximum	
Minimum Flow	10.000	cfs
Design Flow	83.300	cfs
Maximum Flow	135.000	cfs
<b>TAILWATER DATA</b>		
Channel Type	Trapezoidal Channel	
Bottom Width	10.000	ft
Side Slope (H:V)	2.000	_:1
Channel Slope	0.0100	ft/ft
Manning's n (channel)	0.026	
Channel Invert Elevation	4999.000	ft
Rating Curve	View...	
<b>ROADWAY DATA</b>		
Roadway Profile Shape	Constant Roadway Elevation	
First Roadway Station	0.000	ft
Crest Length	38.000	ft
Crest Elevation	5007.340	ft
Roadway Surface	Paved	
Top Width	38.000	ft

**Culvert Properties**

48" RCP

Add Culvert  
Duplicate Culvert  
Delete Culvert

Parameter	Value	Units
<b>CULVERT DATA</b>		
Name	48" RCP	
Shape	Circular	
Material	Concrete	
Diameter	4.000	ft
Embedment Depth	0.000	in
Manning's n	0.013	
Culvert Type	Straight	
Inlet Configuration	Square Edge with Headwall	
Inlet Depression?	No	
<b>SITE DATA</b>		
Site Data Input Option	Culvert Invert Data	
Inlet Station	0.000	ft
Inlet Elevation	5000.000	ft
Outlet Station	100.000	ft
Outlet Elevation	4999.000	ft
Number of Barrels	1	

Help Click on any icon for help on a specific topic Low Flow AOP Energy Dissipation Analyze Crossing OK Cancel

See Table 5 for output

**Figure 4 Ex. Problem 7-1, HY-8 Input Check Flood-Trial 3, 48" RCP**

**Table 5 Ex. Problem 7-1 Check Flood-Trial 3, 48" RCP  
HY-8 Analysis Results**

Customized Table

Culvert Crossing: Ex. Problem 7-1 Check Flood-Trial 3, 48" RCP

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)	Length Full (ft)	Length Free (ft)	Crest Control Elev (ft)	Face Control Elev (ft)	Throat Control Elev (ft)	Tailwater Elevation (ft)
10.0	10.0	5001.25	1.25	0.0*	1-S2n	0.69	0.92	0.72	0.35	6.26	2.69	0.00	100	0.00	0.00	0.00	4999.35
22.5	22.5	5001.92	1.92	0.49	1-S2n	1.04	1.40	1.09	0.56	7.85	3.60	0.00	100	0.00	0.00	0.00	4999.56
35.0	35.0	5002.52	2.52	1.00	1-S2n	1.31	1.76	1.38	0.73	8.80	4.20	0.00	100	0.00	0.00	0.00	4999.73
47.5	47.5	5003.04	3.04	1.51	1-S2n	1.54	2.06	1.64	0.87	9.46	4.65	0.00	100	0.00	0.00	0.00	4999.87
60.0	60.0	5003.52	3.52	2.03	1-S2n	1.75	2.33	1.87	0.99	10.05	5.03	0.00	100	0.00	0.00	0.00	4999.99
72.5	72.5	5004	4.00	2.60	1-S2n	1.96	2.57	2.09	1.11	10.53	5.35	0.00	100	0.00	0.00	0.00	5000.11
83.3	83.3	5004.43	4.43	3.12	5-S2n	2.13	2.76	2.28	1.20	10.92	5.60	0.00	100	0.00	0.00	0.00	5000.2
97.5	97.5	5005.06	5.06	3.85	5-S2n	2.35	2.99	2.51	1.31	11.38	5.88	0.00	100	0.00	0.00	0.00	5000.31
110.0	110.0	5005.7	5.70	4.95	5-S2n	2.55	3.17	2.72	1.41	11.76	6.11	0.00	100	0.00	0.00	0.00	5000.41
122.5	122.5	5006.41	6.41	5.60	5-S2n	2.76	3.33	2.92	1.49	12.12	6.32	0.00	100	0.00	0.00	0.00	5000.49
135.0	135.0	5007.21	7.21	6.30	5-S2n	3.00	3.46	3.14	1.58	12.43	6.51	0.00	100	0.00	0.00	0.00	5000.58

### Example Problem 7-2 (FlowMaster Program - Hydraulic Analysis for: Street and Curb Drop Inlet Flow Capacity Computations)

**Note:** The equation, figure, and table numbers referenced with 3 digits (ex. Table 403-1) refer to the same item numbers identified within the Drainage Design Manual.

#### Project Information

Road Type: Four lane principal arterial

Analysis Location: Location on a constant longitudinal slope of 1.5% (not in a sag)

Drainage Structure: Curb (6-inch) and Gutter, proposed NMDOT Drop Inlet for Type "B" Curb required

50-year peak discharge = 4.3 cfs

100-year peak discharge = 8.6 cfs

Drainage Criteria: Summarized **Table 1**

**Table 1 - Criteria for Storm Frequency and Hydraulics**

	Design Flood	Check Flood	Drainage Criteria Table (a)
Storm Frequency	50-yr	100-yr	203-2
Hydraulic Criterion	Limit spread to one driving lane	Limit water depth to top of curb	204-1, 204-2, 205-1
Clogging Factor	Inlet Grates on Grade – assume a 25% grate clogging factor. Inlet Grates in Sag – assume a 50% grate clogging factor. Inlet grates in sag will require a minimum of one flanking inlet (an inlet near to and upstream of the sag inlet). Median Inlet Grates – assume a 50% grate clogging factor.		
a - Tables are located in Section 200 of the NMDOT Drainage Design Manual			

#### NMDOT standard drawings for Drop Inlet Type(s)/Curb and Gutter Assumed for the Analysis:

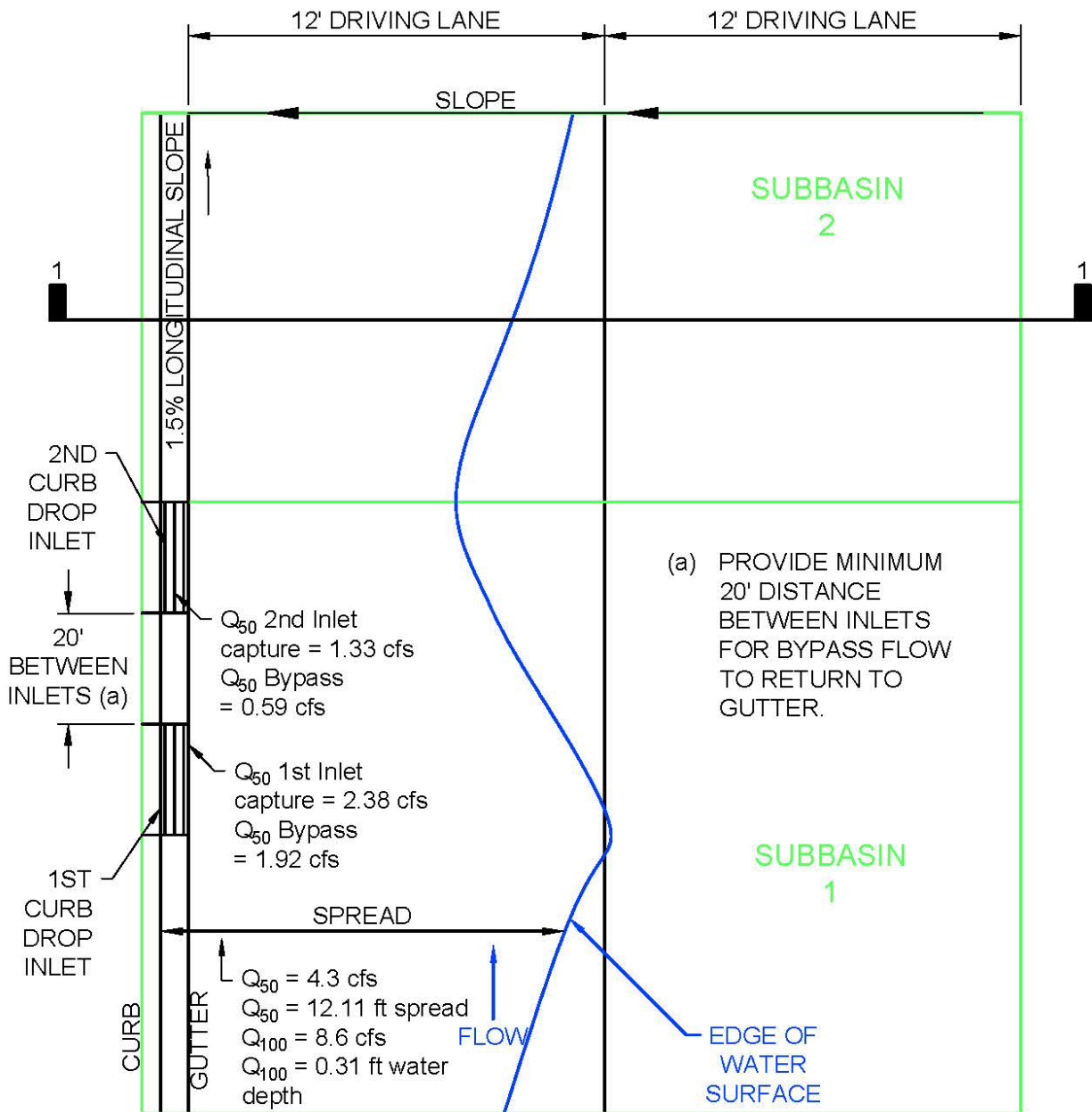
623-14-1/3 Drop Inlet for Type "B" Curb

623-14-2/3 Drop Inlet for Type "B" Curbs, Grates, Frames, Nose Plates, and Center Support Beams

623-14-3/3 Drop Inlet for Type "B" Curbs, Grates, Frames, Nose Plates, and Center Support Beams

Curb per 609-01-1/1 Sidewalk Curb and Gutter.

See **Figure 1** for the roadway schematic, subbasin locations and the proposed inlets.



Roadway Plan View Schematic Not to scale



Section 1-1 Not to scale

Figure 1 Roadway Schematic

**Design Flood - Street Spread**

The FlowMaster program was adopted to solve for the Subbasin 1 (see Figure 1) flow/spread at which the Design Flood spread criterion is violated. Curb drop inlets on grade will be required to reduce the spread to remain within the spread criterion.

The spread at the 50-year Design Flood is 12.11 ft which exceeds the design criterion to limit the spread to one 12 ft driving lane. Therefore, an inlet will be required at this location to remove flow and attain the criterion.

Refer to the FlowMaster Design Flood Street Flow Summary Output.

**Check Flood - Street Flow Depth**

The FlowMaster program was adopted to solve for the Subbasin 1 flow/depth prior to the first drop inlet. The flow depth at the 100-year Check Flood is 0.31 ft which is far below the 0.5 ft depth criterion. Therefore, drop inlet capacity computations are not required for the Check Flood.

Refer to the FlowMaster Check Flood Street Flow Summary Output.

### Prob 7-2 Street Flow Design Flood

#### Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

#### Input Data

Channel Slope	0.01500	ft/ft
Discharge	4.30	ft <sup>3</sup> /s
Section Definitions		

Station (ft)	Elevation (ft)
0+05	100.50
0+05	100.00
0+29	100.48

#### Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+05, 100.50)	(0+29, 100.48)	0.015

#### Options

Current Roughness Weighted Method	Pavlovskii's Method
Open Channel Weighting Method	Pavlovskii's Method
Closed Channel Weighting Method	Pavlovskii's Method

#### Results

Normal Depth	0.24	ft
Elevation Range	100.00 to 100.50 ft	
Flow Area	1.47	ft <sup>2</sup>
Wetted Perimeter	12.36	ft
Hydraulic Radius	0.12	ft
Top Width	12.11	ft
Normal Depth	0.24	ft
Critical Depth	0.28	ft
Critical Slope	0.00645	ft/ft
Velocity	2.93	ft/s

### Prob 7-2 Street Flow Check Flood

## Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

### Input Data

Channel Slope	0.01500	ft/ft
---------------	---------	-------

Discharge	8.60	ft <sup>3</sup> /s
-----------	------	--------------------

## Section Definitions

Station (ft)	Elevation (ft)
0+05	100.50
0+05	100.00
0+29	100.48

### Roughness Segment Definitions

Start Station	Ending Station	Roughness Coefficient
(0+05, 100.50)	(0+29, 100.48)	0.015

## Options

Current Roughness Weighted Method	Pavlovskii's Method
Open Channel Weighting Method	Pavlovskii's Method
Closed Channel Weighting Method	Pavlovskii's Method

## Results

Normal Depth	0.31	ft
Elevation Range	100.00 to 100.50	ft
Flow Area	2.47	ft²
Wetted Perimeter	16.02	ft
Hydraulic Radius	0.15	ft
Top Width	15.71	ft
Normal Depth	0.31	ft
Critical Depth	0.37	ft
Critical Slope	0.00589	ft/ft
Velocity	3.49	ft/s

**Design Flood – Curb Drop Inlet Grate Capacity and Bypass Flow Analysis**

The FlowMaster program was adopted to solve for the Drop Inlet Type “B” Curb (on grade) capacity, bypass flow, and the bypass flow spread. The input data follows:

$$Q_{50} = 4.3 \text{ cfs}$$

longitudinal roadway slope = 1.5 % or 0.015 ft/ft

roadway cross slope = 2% or 0.02 ft/ft

gutter cross slope = 2% 0.02 ft/ft

curb height = 6 inches or 0.5 ft

use Manning’s “n” based on appropriate site conditions. In this case assume “n” = 0.015

**Results Summary for First Drop Inlet Type “B” Curb**

Intercepted Flow = 2.38 cfs

Bypass Flow = 1.92 cfs

Spread = 11.27 ft which is less than one 12 ft lane width. The spread just meets Design Flood criterion, therefore 1 inlet could be considered sufficient.

Refer to the FlowMaster First Inlet Design Flood - Summary Output.

**Need for a Second Inlet**

However, one additional inlet adjacent to the first inlet is recommended to reduce the spread to a smaller value considering that the next downstream Subbasin 2 (see **Figure 1**) will be contributing to the bypass flow. The bypass flow combined with the Subbasin 2 flow will soon exceed the spread criterion. Therefore, an additional second inlet adjacent to the first inlet is proposed.

The bypass discharge from Inlet 1 was applied as the inflow to Inlet 2.

**Results Summary for Second Drop Inlet Type “B” Curb**

Intercepted Flow = 1.33 cfs

Bypass Flow = 0.59 cfs

Spread = 8.33 ft which is less than one 12 ft lane width. Therefore, the second inlet reduces the spread to a more reasonable value than only the first inlet, and of course attains the Design Flood spread criterion.

Refer to the FlowMaster Second Inlet Design Flood - Summary Output.



**PROB 7-2 DESIGN FLOOD 1ST INLET****Project Description**

Solve For Efficiency

**Input Data**

Discharge	4.30	ft <sup>3</sup> /s
Slope	0.01500	ft/ft
Gutter Width	1.40	ft
Gutter Cross Slope	0.02	ft/ft
Road Cross Slope	0.02	ft/ft
Roughness Coefficient	0.015	
Local Depression	2.00	in
Local Depression Width	1.40	ft
Grate Width	1.38	ft
Grate Length	5.00	ft
Grate Type	P-50 mm (P-1-7/8")	
Clogging	25.00	%
Curb Opening Length	5.50	ft

**Options**

Calculation Option	Use Both
Grate Flow Option	Exclude None

**Results**

Efficiency	55.27	%
Intercepted Flow	2.38	ft <sup>3</sup> /s
Bypass Flow	1.92	ft <sup>3</sup> /s
Spread	11.27	ft
Depth	0.23	ft
Flow Area	1.27	ft <sup>2</sup>
Gutter Depression	0.00	ft
Total Depression	0.17	ft
Velocity	3.39	ft/s
Splash Over Velocity	11.14	ft/s
Frontal Flow Factor	1.00	
Side Flow Factor	0.24	
Grate Flow Ratio	0.29	
Equivalent Cross Slope	0.05547	ft/ft
Active Grate Length	3.75	ft
Length Factor	0.08	
Total Interception Length	22.13	ft

**PROB 7-2 DESIGN FLOOD 2ND INLET****Project Description**

Solve For Efficiency

**Input Data**

Discharge	1.92	ft <sup>3</sup> /s
Slope	0.01500	ft/ft
Gutter Width	1.40	ft
Gutter Cross Slope	0.02	ft/ft
Road Cross Slope	0.02	ft/ft
Roughness Coefficient	0.015	
Local Depression	2.00	in
Local Depression Width	1.40	ft
Grate Width	1.38	ft
Grate Length	5.00	ft
Grate Type	P-50 mm (P-1-7/8")	
Clogging	25.00	%
Curb Opening Length	5.50	ft

**Options**

Calculation Option	Use Both
Grate Flow Option	Exclude None

**Results**

Efficiency	69.04	%
Intercepted Flow	1.33	ft <sup>3</sup> /s
Bypass Flow	0.59	ft <sup>3</sup> /s
Spread	8.33	ft
Depth	0.17	ft
Flow Area	0.69	ft <sup>2</sup>
Gutter Depression	0.00	ft
Total Depression	0.17	ft
Velocity	2.77	ft/s
Splash Over Velocity	11.14	ft/s
Frontal Flow Factor	1.00	
Side Flow Factor	0.31	
Grate Flow Ratio	0.38	
Equivalent Cross Slope	0.06618	ft/ft
Active Grate Length	3.75	ft
Length Factor	0.12	
Total Interception Length	14.19	ft

**Conclusion**

The Design Flood spread criterion was the limiting criterion for this roadway/curb and gutter location. The Check Flood depth criterion was not the limiting criterion and was attained even without the first inlet. Therefore, design two Drop Inlet Type “B” Curb Inlets to meet the Design Flood spread criterion.

**Hydraulic Consideration for the Second Inlet**

To allow the bypass flow spread and velocity to somewhat normalize in the street/gutter section after partial interception by the first inlet, the second inlet should be located a minimum of 20 ft downstream from the first inlet as illustrated in **Figure 1**.

**Example Problem 7-3 (Detention Pond Example Tables)**

**Note:** The equation, figure, and table numbers referenced with 3 digits (ex. Table 403-1) refer to the same item numbers identified within the Drainage Design Manual.

Examples of the following useful tables will be presented

Detention Pond Elevation – Storage-Volume-Discharge Data

Principal Spillway Pipe Rating Curve (from CulvertMaster)

Detention Pond Routing Summary Table

Project Information

A 0.62 sq mi urban watershed was simulated with a HEC-HMS model. Evaluation of downstream capacity constraints revealed that only 90 cfs or less is allowed. The 100-yr 24-hr. inflow hydrograph peak discharge is 755 cfs. Therefore, a detention pond is required. Only the 100-yr 24-hr. flood is presented in this example.

Watershed and Inflow Hydrograph Data

Urban watershed that is fully developed

Drainage area = 0.62 sq mi

Check Flood = 100-yr 24-hr peak discharge of 755 cfs, and runoff volume of 41.3 ac-ft

Downstream capacity limitation = 90 cfs

Detention Pond Criteria

The NMDOT Detention Pond Drainage Criteria (Section 200) are summarized here:

Design Flood –

50-yr 24-hr. storm, provide 2 ft of freeboard to the top of the embankment

Peak water surface elevation - must remain at or below the emergency spillway

Check Flood –

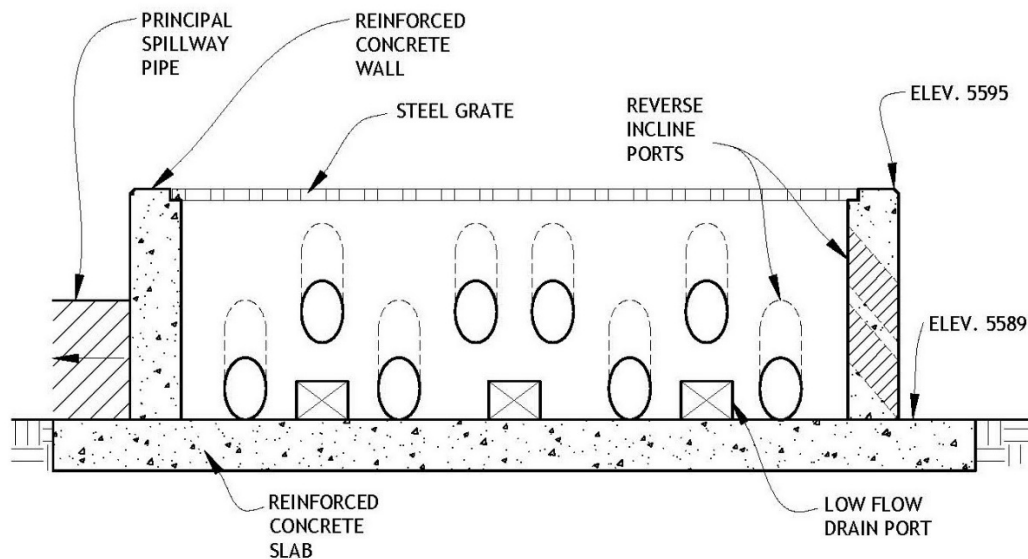
100-yr 24-hr storm, provide 1 ft of freeboard to the top of the embankment

Peak water surface elevation - may flow through the emergency spillway

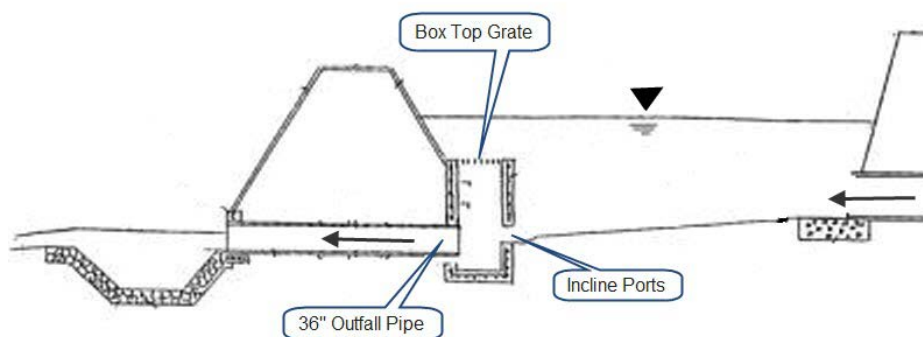
Dead Storage – Urban Condition, must provide 10% of the hydrograph inflow volume as dead storage volume.

Computation = 41.3 ac-ft (0.10) = 4.1 ac-ft of dead storage volume is required.

The detention pond was designed for the check flood. **Figure 1** illustrates a principal spillway reinforced concrete box structure. This structure contains drain ports, reverse incline ports, a top grate and a principal outlet pipe.



**Figure 1 Reinforced Concrete Spillway With Reverse Incline Ports**



**Figure 2 Pond and Embankment Profile Including the Reinforced Concrete Spillway With Reverse Incline Ports**

### **Description of the Hydraulic Function of the Vertical Wall Principal Spillway Structure with Reverse Incline Ports**

1. Low flow drain ports are located at the invert elevation of the principal spillway box walls. These are included for pond bottom drainage to minimize standing or stagnant water. The small discharge capacity may be ignored considering that sediment and debris are likely to plug these to some degree during a storm runoff event.
2. The reverse incline ports are included as stormwater quality improvement features. This design ensures that floatable debris will not enter the principal spillway box. These will function as orifice flow until submerged within the principal spillway box, and then the ports will be non-functional.

3. The grate located on top of the vertical walls will also prevent most floatable debris from entering the principal spillway box depending on the grate bar spacing. The crest of the vertical walls will function as a weir for a brief time until submerged. Note that if the principal spillway pipe capacity is lower than the weir flow capacity, then the principal spillway pipe will govern the discharge. Then the box will become full of water and will submerge the weir, and the weir will become non-functional.
4. The principal spillway pipe is the terminal outfall from the principal spillway box.
5. Table 1 assumes that once the water surface elevation exceeds elevation 5595 (top of vertical wall), flow through the reverse incline ports is negligible (therefore shown as zero in the table starting at elevation 5596), and all flow to the outfall pipe is through the box top steel grate.

**Table 1** presents the required data that is necessary to simulate a reservoir or pond routing with the HEC-HMS program. In addition, **Table 1** provides documentation of the assumptions and formulas applied. The principal spillway pipe discharge rating curve was developed with the CulvertMaster Program, and the results of that analysis are presented in **Table 2**.

#### Reservoir Routing Results

The results of a reservoir routing should be summarized in a table such as **Table 3**. The table helps to understand how the reservoir functions with respect to many items, including freeboard to the: principal spillway elevation, emergency spillway elevation, and the top of embankment elevation.

The results indicate that the freeboard criteria were achieved. There is 2 ft of freeboard to the emergency spillway and 3 ft of freeboard to the top of the embankment. The peak discharge was reduced from 755 cfs to 74 cfs (the downstream constraint was 90 cfs or less).

#### **Conclusion**

The top of the pond embankment and emergency spillway elevations could be reduced if desired, to minimize embankment quantities and reduce the total project cost.

**Table 1 - Detention Pond Elevation – Storage Volume – Discharge Data**

TABLE 1

POND NAME Based on Final Design Plans

Elevation - Storage Volume - Discharge Data and Computations

grey box means must input elevation and area data

Contour Elevation NAVD 1988	Depth	Contour	Area	Incremental Volume	Incremental Volume	Cumulative Volume	1st Row of Reverse Incline Ports (principal spillway box - in vertical wall)	2nd Row of Reverse Incline Ports (principal spillway box - in vertical wall)	Box Top Grate Discharge	SUMMATION of reverse incline ports, drains and top grate	Principal Spillway 36-in. Outfall Pipe Discharge	Total Principal Spillway / Outfall Pipe Discharge	Emergency Spillway Discharge	Total Discharge Rating Curve	Comment	
		Principal Spillway Orifice Diameter (inches)					8.0	8.0			36.0					
		Number of Orifices					12.0	12.0			1.0					
ft d		sq ft d	cu ft	ac-ft	ac-ft	cfs a	cfs a	cfs a	cfs b	cfs f	cfs c	cfs e	cfs b	cfs		
5585.00	0.00	0	0	0.0000	0.0000											
5586.00	1.00	46,585	23,293	0.5347	0.5347	THIS VOLUME REPRESENTS THE DEAD STORAGE VOLUME located below the principal spillway elevation - Dead storage volume computed as 10% of the inflow hydrograph volume (Drainage Criteria for an urban watershed) . The 100-yr. 24-hr. inflow hydrograph volume = 41.3 ac-ft. Therefore, provide at least 4.3 ac-ft of dead stoage volume										
5587.00	2.00	105,063	75,824	1.7407	2.2754											
5588.00	3.00	113,329	109,196	2.5068	4.7822											
5589.00	0.00	118,000	0	0.0000	0.0000	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0	Pond bottom and principal spillway structure invert	
5590.00	1.00	122,000	120,000	2.7548	2.7548	0.0	0.0	0.0	0.0	0.0	4.0	0.0	0.0	0	Highest Invert 1st row of reverse incline ports	
5591.00	2.00	128,000	125,000	2.8696	5.6244	19.8	0.0	0.0	0.0	19.8	16.0	16.0	0.0	16		
5592.00	3.00	132,000	130,000	2.9844	8.6088	28.0	0.0	0.0	0.0	28.0	32.0	28.0	0.0	28	Highest Invert of 2nd row of reverse incline ports	
5593.00	4.00	136,000	134,000	3.0762	11.6850	34.4	19.8	0.0	0.0	54.2	50.0	50.0	0.0	50		
5594.00	5.00	141,000	138,500	3.1795	14.8646	39.7	28.0	0.0	0.0	67.7	61.0	61.0	0.0	61		
5595.00	6.00	145,000	143,000	3.2828	18.1474	44.3	34.4	0.0	0.0	78.7	64.0	64.0	0.0	64	Top of principal spillway grate	
5596.00	7.00	148,116	146,558	3.3645	21.5119			96.0	96.0	96.0	67.0	67.0	0.0	67		
5597.00	8.00	157,339	152,728	3.5061	25.0180			271.5	271.5	271.5	70.0	70.0	0.0	70		
5598.00	9.00	166,790	162,065	3.7205	28.7385			498.8	498.8	498.8	72.0	72.0	0.0	72		
5599.00	10.00	176,505	171,648	3.9405	32.6790			606.0	606.0	606.0	74.0	74.0	0.0	74		
5600.00	11.00	186,425	181,465	4.1659	36.8449			677.6	677.6	677.6	77.0	77.0	0.0	77		
5601.00	12.00	196,606	191,516	4.3966	41.2415			742.3	742.3	742.3	79.0	79.0	0.0	79	Emergency Spillway Elevation	
5601.20	12.20	198,606	39,521	0.9073	42.1487			754.5	754.5	754.5	81.0	81.0	23.3	104		
5601.40	12.40	200,606	39,321	0.9165	43.0652			766.6	766.6	766.6	82.0	82.0	65.8	148		
5601.60	12.60	202,606	40,321	0.9256	43.9909			778.5	778.5	778.5	83.0	83.0	120.8	204		
5601.80	12.80	204,606	40,721	0.9348	44.9257			790.2	790.2	790.2	84.0	84.0	186.0	270		
5602.00	13.00	206,606	41,121	0.9440	45.8697			801.7	801.7	801.7	85.0	85.0	260.0	345		

a - Orifice equation and coefficient were obtained from Equation 4-10 and Table 4-3 from "Handbook of Hydraulics" Sixth Edition, by Brater & King, 1982.

$$Q = C a \sqrt{2 g h}$$
$$C = \frac{\pi D^2}{4}$$

C = 0.590

g=32.2 ft/sec^2, a=area (sq ft) h=head (ft)

full area formula

Note-The reverse incline ports will be submerged after water exceeds the top of box/grate elevation, so 0 cfs

b - Principal spillway box and emergency spillway flows were computed based on the following data used in the weir equation

Q = CLH^1.5

C = discharge coefficient, L = spillway length perp. to flow (ft), H = head (ft)

Emergency Spillway C = 2.6

L, ft = 100

El. Emer. Spill. 5601.0

Principal Spillway box Grate / Weir C = 3.0

L, ft = 32

El. 8'x 8' grate 5595.0 princ. spill. pipe capacity governs

c - Rating curve computed with the CulvertMaster Program - see Table 2 for assumptions and rating curve developed with CulvertMaster

d - Data Source : 1 ft. accurate topographic design survey

e - Below the top of the 36" pipe elevation, the reverse incline ports govern the discharge. After the principal spillway pipe becomes submerged at elevation 5593, the principal spillway pipe governs the remaining discharge rating curve.

b - Weir equation and "C" coefficients were obtained from Equation 5-10 and Table 5-3 from "Handbook of Hydraulics" Sixth Edition, by Brater & King, 1982.

f - Assume 0 cfs flow at drains located at the bottom of the vertical wall as these have a tendency to plug with sediment/debris, but these are necessary to allow water at pond bottom to drain into principal spillway box

**Table 2 - Principal Spillway Pipe Discharge Rating Curve**  
**Principal Spillway Pipe Discharge Rating Curve**  
 (computed with the CulvertMaster Program)

<b>Basic Data</b>						
culvert material =	RCP	--				
culvert shape =	round	--				
culvert diameter =	36	inches				
Manning's n =	0.025	--				
culvert length =	225	ft				
culvert slope =	0.01	ft / ft				

Pond Elevation	Depth	Upstream Invert Elevation	Headwater Elevation	Downstream Invert Elevation	Tailwater Elevation	Discharge from Culvert Master
ft	ft	ft a	ft b	ft a	ft c	
5589	0	5589	5589	5587	5587.0	0
5590	1		5590		5587.7	4
5591	2		5591		5588.3	16
5592	3		5592		5589.0	32
5593	4		5593		5589.7	50
5594	5		5594		5590.4	61
5595	6		5595		5591.0	64
5596	7		5596		5591.7	67
5597	8		5597		5592.4	70
5598	9		5598		5593.0	72
5599	10		5599		5593.7	74
5600	11		5600		5594.4	77
5601	12		5601		5595.0	79
5601.2	12.2		5601.2		5595.2	81
5601.4	12.4		5601.4		5595.3	82
5601.6	12.6		5601.6		5595.4	83
5601.8	12.8		5601.8		5595.6	84
5602	13		5602		5595.7	85

a - Based on the final design grading plan with 1 ft accurate contours

b = headwater elevation = depth + upstream invert elevation

c - Tailwater elevation assumption, =  $(2/3 * \text{depth}) + \text{downstream invert elevation}$



### Table 3 - Detention Pond Routing Summary

Detention Pond Name	Existing or Proposed Pond	Development / Model Condition	Storm Return Period / Duration	Peak Inflow	Peak Outflow	Inflow Runoff Volume	Outflow Runoff Volume	Maximum Storage Volume (top of embankment)	Peak Storage Volume	Peak Water Surface Elevation	Top of Principal Spillway Elevation	Emergency Spillway Elevation	Pond Invert Elevation	Maximum Pond Depth	Peak Water Depth	Top of Pond Embankment Elevation	Freeboard to Principal Spillway Elevation	Freeboard to Emergency Spillway Elevation	Freeboard to top of Pond Embankment
		inches	yr / hr	cfs	cfs	ac-ft	ac-ft	ac-ft	ac-ft	ft	ft	ft	ft	ft	ft	ft	ft	ft	ft
a			b	b	b	b	b	c	b	b	c	c	c	c		c	d	d	d
	Proposed	Full Watershed Development	100 / 24	755	74	41.3	41.2	45.9	27.8	5599.0	5595.0	5601.0	5589.00	13	10.0	5602.00	-4.0	2.0	3.0

a - Refer to Detention Pond Final Grading Plan

b - Refer to the HEC-HMS model output for the pond routing results

c - See Table 1 for the pond Elevation - Storage Volume - Discharge Data. Pond invert elev. of 5589.00 here represents top of dead storage elevation. The pond invert elev. = 5585.0 at bottom of dead storage.

d- Negative number indicates the flow depth exceeds referenced elevation - no freeboard available

# **APPENDIX 8 EXAMPLE SEDIMENT TRANSPORT AND SCOUR CALCULATIONS**

## **CONTENTS:**

**Example Problem 8-1 Sediment Transport Analysis with the Zeller-Fullerton Equation**

**Example Problem 8-2 NM 456 Bridge # 4920 Bridge Replacement Scour Analysis**

**Example Problem 8-1 Sediment Transport Analysis with the Zeller-Fullerton Equation**

**Note:** The equation, figure, and table numbers referenced with 3 digits (ex. Table 403-1) refer to the same item numbers identified within the Drainage Design Manual.

**Project Information**

The purpose of this example is to demonstrate how to delineate an assumed alluvial sand and gravel bed watercourse into reaches and demonstrate how to compute the aggradation/degradation depths for a given hydrograph and other data, by application of the Zeller-Fullerton bed load transport capacity equation. The assumed data fall within the range of parameters applicable to the Zeller-Fullerton Equation (refer to **Table 606-2** in **Section 606.6.1**).

**Table 606-2 Range of Parameter Values for the Zeller-Fullerton Equation**

Source: AMAFCA, November 1994, Sediment and Erosion Design Guide, Table 3.5, p. 3-35  
[http://www.bernco.gov/uploads/FileLinks/cbb8af72471e4746ba9e92e9a67318bb/Sediment\\_and\\_Erosion\\_Design\\_Guide\\_AMAFCA\\_.pdf](http://www.bernco.gov/uploads/FileLinks/cbb8af72471e4746ba9e92e9a67318bb/Sediment_and_Erosion_Design_Guide_AMAFCA_.pdf)

Parameter	Range
Depth	1 – 20 ft
Velocity	3 – 30 ft/s
Manning's $n$	0.018 – 0.035
Bed slope	0.001 – 0.040 ft/ft
Unit discharge	10 – 200 cfs/ft
Particle size	$0.5 \text{ mm} \leq D_{50} \leq 10 \text{ mm}$
Gradation coefficient	$2 \leq G \leq 5$

The wash load component of the total sediment load is not considered in this example. The wash load is comprised of silts and clays that wash through the system, with only minor deposition near the tail end of a hydrograph.

**Step 1**

Delineate the watercourse into two or more reaches. Refer to **Figure 1** for the example watercourse and bridge plan view. The watercourse has been delineated into four reaches. Reach 1 is not evaluated, Reaches 2, 3 and 4 were delineated to have similar lengths and assumed characteristics (Reaches 2 and 4 are the same length and have the same characteristics for this example).

Similar reach lengths are important to avoid erroneous results when computing the aggradation/degradation volumes and depths that result from the upstream reach sediment supply. Reach length is particularly important for manual sediment transport computations as presented here. A reach beginning and ending point should bound the limits within which, the characteristics are similar. Characteristics include: cross-section shape, bed slope, roughness coefficients, bed and bank sediment gradation range, and bed armoring. Reach 2 is assumed

to be the supply reach that is required to begin the computations, and to compute the downstream reach aggradation / degradation quantities.

### Step 2

Obtain or simulate a hydrograph. Refer to **Figure 2** that presents an example hydrograph with a 24-hour duration and a peak discharge of 8,000 cfs. The hydrograph was discretized into eight flow segments (**Figure 2**). The duration (time) and discharge of each segment approximate the actual hydrograph, and those segment values are summarized on the figure.

### Step 3

Plot the bed sediment gradation data as shown on **Figure 3**. From the figure determine  $D_{84}$ ,  $D_{50}$  and  $D_{16}$  that are required to compute the gradation coefficient  $G$ .

### Step 4

Prepare a spreadsheet as shown on **Table 1**. The top of the spreadsheet presents the Zeller-Fullerton bed material load sediment transport equation, the reach data, the sediment gradation data and the computed sediment gradation coefficient  $G$ .

The table presents two rows that have letters such as C, etc., and these are provided to indicate the columns required to compute some of the results (example, H is computed as column F\*E).

For each reach and each hydrograph segment, the spreadsheet formulas compute the unit sediment transport capacity  $q_s$  (cfs/ft) based on the velocity, top width, area, hydraulic depth and sediment gradation data. Then it computes the sediment transport capacity  $Q_s$  cfs ( $q_s$  (cfs/ft)  $\times T$  (ft),  $T$  = top width for that hydrograph flow segment). Then the  $Q_s$  (cu ft) is computed as  $Q_s$  (cfs)  $\times$  segment duration (seconds).

Note –  $Q_s$  (cu ft) at this point (column M) was computed for an average cross-section in the stream reach, or as  $Q_s$  (cu ft) / 1 ft reach length in the flow direction and does not define the total  $Q_s$  for the reach,  $Q_s$  total must incorporate reach length. Therefore,  $Q_s$  (cu ft) (column U) for the reach is computed as  $Q_s$  (cu ft) / 1 ft reach length  $\times$  reach length (ft), (columns M\*O).

Aggradation / degradation volume and depth are computed in the last column. Aggradation occurs when the supply sediment load is greater than the transport capacity of the current reach. Degradation occurs when the supply sediment load is less than the transport capacity of the current reach.

The average aggradation / degradation depth (ft) in the reach is computed as the difference in these sediment transport capacities (volumes (cu ft)), divide by the reach bed area (sq ft).

### Aggradation Depth Adjustment for Sediment Porosity

If the supply sediment load from Reach 2 was assumed to come from compacted material, then when that sediment deposits as a loose volume, that volume will be greater due to porosity ( $n$ ). A typical sediment porosity ( $n$ ) value is assumed as  $n=0.4$ .

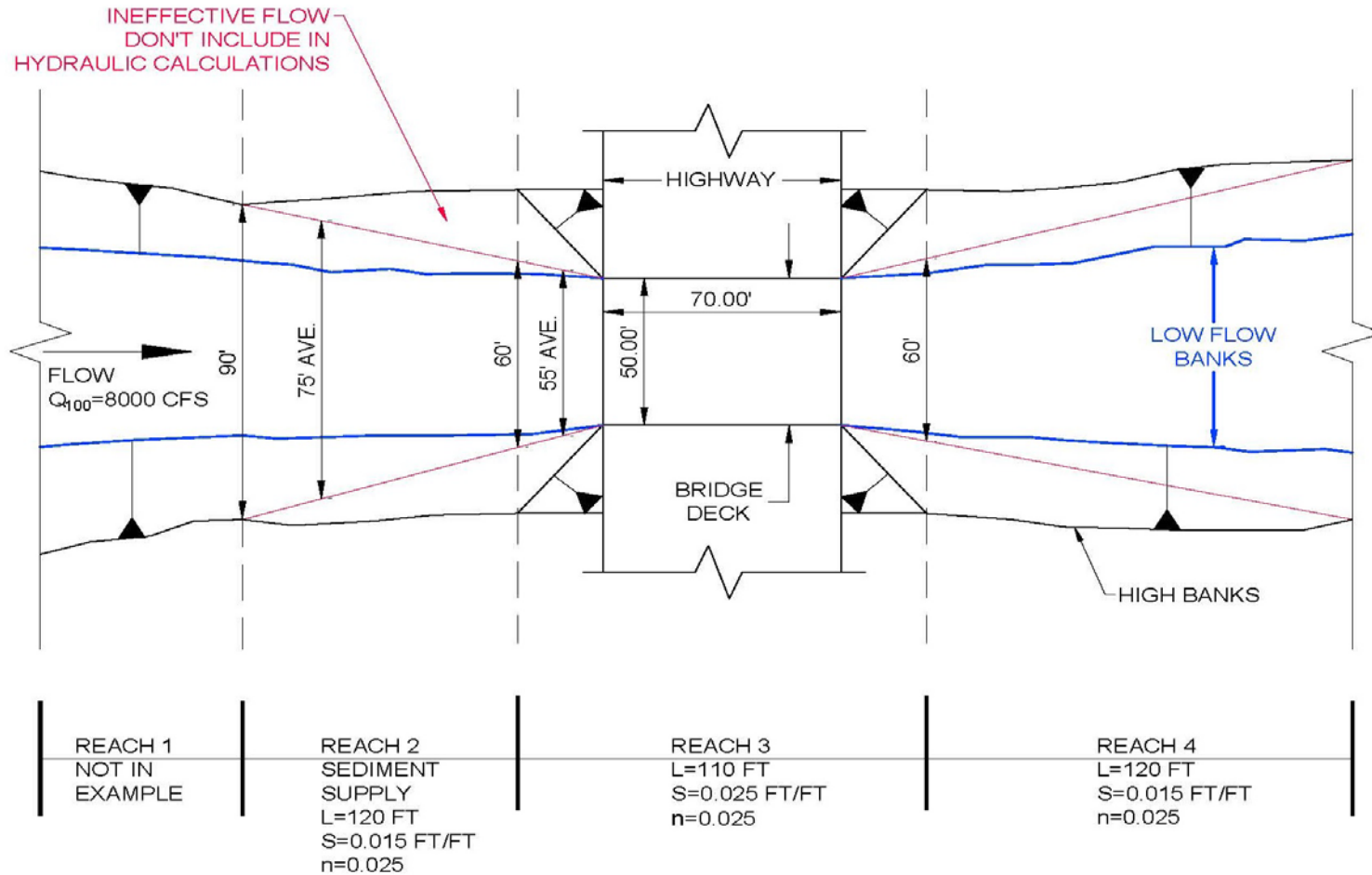
The aggradation depth computed for Reach 3 could be adjusted to account for the porosity (0.4) as follows:

$$Z_{\text{adjusted, ft (aggradation depth)}} = Z_{\text{ft (unadjusted aggradation depth)}} / (1-n)$$

$$\text{Therefore, } Z_{\text{adjusted}} = 0.4 \text{ ft} / (1-0.4) = 0.7 \text{ ft}$$

**CONCLUSION -**

This example indicates that Reach 3 under the bridge will aggrade about 0.7 ft and Reach 4 downstream from the bridge will degrade about 0.2 feet. However, due to the uncertainty involved in sediment transport analyses, an additional factor of safety could be included to the computed values, such as a 50% factor applied to either an aggradation or a degradation value

**FIGURE 1 WATERCOURSE AND BRIDGE PLAN VIEW**

NOT TO SCALE

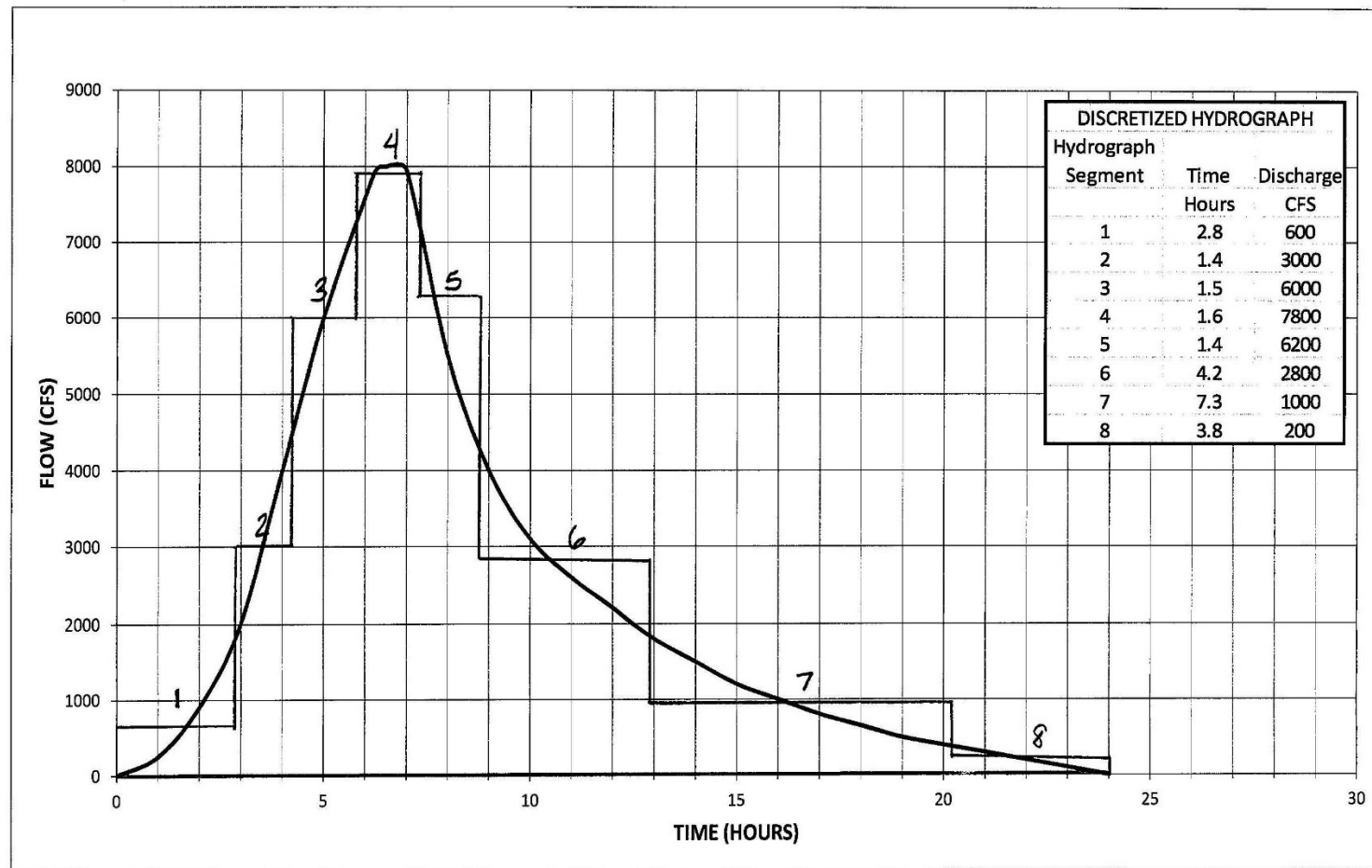
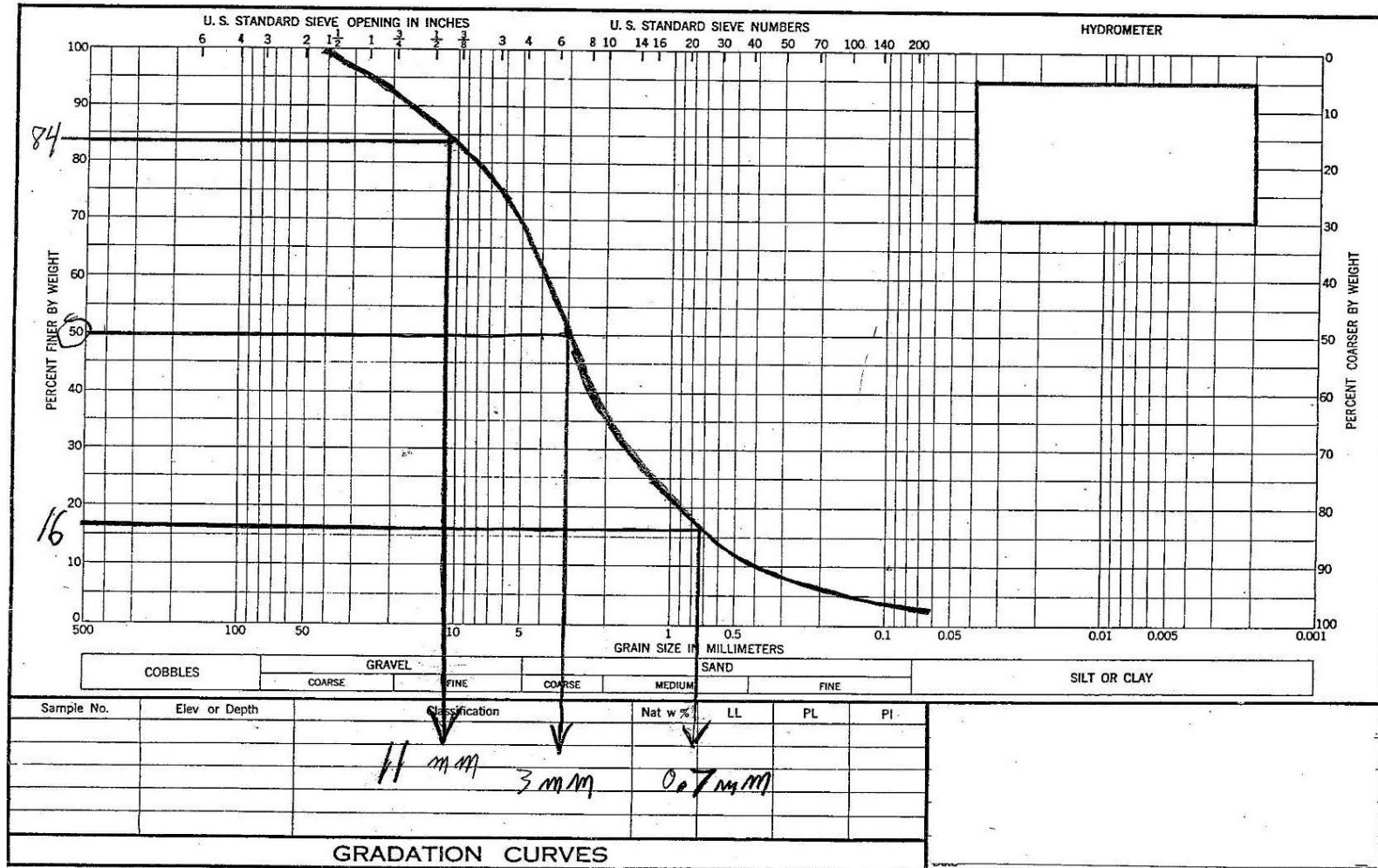


FIGURE 2 EXAMPLE HYDROGRAPH



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FIGURE 3 GRADATION DATA CURVE





**Example Problem 8-2 Bridge Scour Analysis**

**Note:** The equation, figure, and table numbers referenced with 3 digits (ex. Table 403-1) refer to the same item numbers identified within the Drainage Design Manual.

**Project Information**

The purpose of this example is to demonstrate how to compute total scour at a proposed bridge. The drainage analysis and report were prepared in 2014 by Smith Engineering Company (Smith) to develop drainage (hydrologic/hydraulic) analyses, scour analyses and scour counter measure recommendations for NM 456, proposed replacement for existing Bridge # 4920 located on the Dry Cimarron River. The existing bridge has two piers and was identified as scour critical by the NMDOT. The existing bridge will be replaced with a new clear span bridge. The following photographs illustrate the existing bridge and the Dry Cimarron River near the bridge.



Upstream of the existing bridge looking downstream at the bridge and the Dry Cimarron River.



About 100 feet upstream of the bridge looking upstream at Dry Cimarron River.





Upstream of the existing bridge looking downstream at the bridge, Dry Cimarron River and the south overbank.



Upstream of the existing bridge about 300 feet looking upstream at the Dry Cimarron River.



Standing on the existing bridge looking upstream at the Dry Cimarron River.





Downstream of the existing bridge near first meander looking upstream at the bridge.

## **Roadway Classification**

NM 456 is a two-lane highway and is considered a rural, two lane Collector with an existing Average Daily Traffic (ADT) of 70, and the future ADT was predicted to be 88 in year 2033.

## **Drainage Criteria**

NMDOT Drainage Criteria applicable at the time of this project analysis follow.

### Freeboard

Design Flood = 25-year – minimum freeboard of 2 ft

Check Flood = 50-year – below the low chord

### Bridge Scour

Design Flood = 100-year – determine scour depth as needed

Check Flood = 500-year – determine scour depth as needed

## **Watercourse and Bridge Scour Analysis Overview**

The prediction of the total bed elevation drop at a bridge is comprised of two components described here.

### Component 1 – Long Term Aggradation/Degradation

First consider the potential for long term bed aggradation/degradation of the watercourse reach upstream and downstream of the bridge. A reach is defined as a length of the watercourse selected for use in an analysis.

### Component 2 - Total Scour at the Bridge Location.

Total scour is the summation of three possible scour components that are: contraction scour (general scour), abutment scour and pier scour.

## **Long Term Aggradation/Degradation Analysis**

An initial evaluation of long term bed stability may be computed with simple equations to predict the maximum stable slope, and that computed stable slope compared to the existing bed slope may provide an initial indication of the potential for bed aggradation or degradation. Note however that engineering judgement may take precedence over computational results, if those results appear unreasonable and/or, if field observations of existing scour evidence and other items that must be considered, appear to be significant factors that would lead to the decision to negate the computational results.

The following items must be observed and if applicable, collected, measured and documented to assist in the prediction of the total bed elevation drop.

- reach/bed stability
- bed armoring (location and extent)
- sediment characteristics (clay, sand, gravel, cobbles, boulders)

- bank stability
- evidence of existing scour locations and depths
- vegetation type(s), location(s) and densities in the bed, banks and overbanks

If the initial bed stability assessment indicates that bed degradation is likely, then a sediment transport analysis may be required.

#### Total Scour Analysis at the Bridge Location

Total scour is the summation of three possible scour components that are:

- contraction scour (also known a general scour)
- abutment scour
- pier scour

Contraction scour may be a more explicit term than “general scour”, because this component of the total scour defines scour due to flow contraction at the bridge section. This is because many bridges are designed with smaller cross section widths than the upstream floodplain widths. The flow and corresponding water surface top width approaching the bridge must be contracted to some degree at the narrowed bridge cross section.

Abutment scour may be computed for each abutment, however, the larger of the two prediction scour depths may be adopted for both abutments.

Pier scour is computed if piers are, or will be present.

#### Total Bed Elevation Drop

The predicted total bed elevation drop is the summation of:

Long term bed degradation, contraction scour, abutment scour and pier scour.

If aggradation is predicted at the bridge location, this value is not applied to the total bed elevation drop computation as this positive value would reduce the predicted total scour depth.

#### Hydrologic Analysis with the HEC-HMS Program

The HEC-HMS program Version 4.0 was adopted to simulate 25-year, 50-year, 100-year and 500-year, 24-hour duration rainfall events. The results for the 295 square mile basin are listed here.

25-year 24-hour peak discharge = 10,726 cfs  
50-year 24-hour peak discharge = 15,372 cfs  
100-year 24-hour peak discharge = 21,417 cfs  
500-year 24-hour peak discharge = 30,000 cfs

## Hydraulic Analysis with the HEC-RAS Water Surface Profile Program

### Steady Flow Model

The HEC-RAS Program Version 4.1.0 was adopted to simulate the steady flow analysis that was the basis for recommending proposed bridge low chord elevations and for computing hydraulic results needed for the scour calculations. The hydraulic and scour analyses were based on the 60% complete design plans provided by Bohannon-Huston Inc. (BHI). The proposed bridge will have one pier. Wire enclosed riprap was proposed by BHI for protection of the abutment slopes. The flow regime is subcritical for this reach of the Dry Cimarron River.

Note that the HEC-RAS program, at the time of this Drainage Design Manual preparation, does not contain the latest FHWA HEC-18 recommended scour equations. Therefore, scour computations were developed based on the HEC-18, April 2012 scour equations within Excel spreadsheets, using the preliminary bridge plans, sediment gradation data, reach topographic data, and the HEC-RAS model hydraulic results.

FHWA, April 2012, "HEC-18, Evaluating Scour at Bridges, Fifth Edition".

<http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hif12003.pdf>

### Quasi-Steady Flow Model

Sediment transport analyses to compute long term aggradation/degradation may be computed using the HEC-RAS program assuming the Quasi-Steady Flow Regime. However, that model was not developed for this analysis, because the initial bed stability assessment determined that long term bed degradation was unlikely as described here.

### Sediment Transport Analysis - Long Term Aggradation or Degradation

A maximum stable slope computation was completed for the watercourse reach based on the data and results developed for the analysis, and the methodology presented in the Sediment and Erosion Design Guide (AMAFCA, November 1994).

Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA), November 1994, "Sediment and Erosion Design Guide".

[https://www.bernco.gov/uploads/FileLinks/cbb8af72471e4746ba9e92e9a67318bb/Sediment\\_and\\_Erosion\\_Design\\_Guide\\_AMAFCA\\_.pdf](https://www.bernco.gov/uploads/FileLinks/cbb8af72471e4746ba9e92e9a67318bb/Sediment_and_Erosion_Design_Guide_AMAFCA_.pdf)

The existing river reach bed slope is 0.45 % and the computed stable slope is less at 0.17%. This would imply that the existing steeper watercourse slope may degrade to reach a 0.17% mild slope. **Table 1** contains the data, computations and the documentation of the maximum stable slope equation.

However, the equation is based on the assumption of a sand/gravel bed channel and is not applicable because much of the existing river bed and overbanks are heavily vegetated with grass (see photographs). In addition, the bed was assumed to be stable and unlikely to degrade based on several factors that include:

- heavily vegetated channel bed, stable heavily vegetated overbanks
- mild channel slope
- low range of velocities (5 to 12 ft/s downstream of the bridge and 7 to 18 ft/s upstream of the bridge)
- engineering judgement

Therefore, the aggradation/degradation depth was assumed to be zero, meaning that a sediment transport analysis to determine the long term aggradation/degradation depth of the bed profile is not necessary.



<b>TABLE 1</b> <b>Stable Bed Slope Calculations</b> Dry Cimarron River, near Bridge # 4920, NM 456		
Item - Term	Value or Result	Definition of Terms
<b>Existing Arroyo Bed Slope Computations (a)</b>		
most downstream elevation, ft	4,202.31	--
most upstream elevation, ft	4,218.04	--
length of river reach, ft	3,513.50	--
existing reach slope, ft/ft	0.0045	--
existing reach slope, %	<b>0.45</b>	--
<b>Stable Bed Slope Computations</b>		
$Q_{100}$ (cfs) = HEC-HMS 100-yr. - 24-hr. peak discharge	21,417	--
$Q_d$ (cfs) = 0.2 ( $Q_{100}$ )	4,283.4	Dominant discharge per Eqn. 3.77 AMAFCA Sediment and Erosion Design Guide, pg. 3-72, 3-73
n	0.035	Manning's "n" value used in HEC-RAS model
F	40.00	Typ. Width-Depth ratio of flowing water
Fr	0.40	Average Froude number from HEC-RAS model
$S_s$ (ft/ft) = $Q_d^{-0.133} (18.28) (n^2) (F^{0.133}) (Fr^{2.133})$	0.0017	Maximum Stable Slope per AMAFCA Sediment and Erosion Design Guide, Eqn. 3.59, pgs. 3-49, 3-50
$S_s$ (%)	<b>0.17</b>	Maximum Stable Slope, %
a - From a 1 ft accurate contour map.		
<p>The existing river slope is steeper than the computed slope. However, the equation is based on the assumption of non-cohesive sand/gravel channels. However, much of the existing river bed and overbanks are heavily vegetated with grass, therefore this computation is not valid. Velocities downstream of the bridge range from about 5 to 12 ft/s and this bed/watercourse appears to be very stable (see photographs). From just downstream of the bridge to far upstream of the bridge, velocities range from about 7 to 18 ft/s. Based on engineering judgement - the river appears to be relatively stable and is assumed to be at approximately the equilibrium slope or a stable slope.</p>		
a - From a 1ft accurate contour map.		
Source: Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA), November 1994, "Sediment and Erosion Design Guide". <a href="http://www.berncogov/uploads/FileLinks/cbb8af72471e4746ba9e92e9a67318bb/Sediment_and_Erosion_Design_Guide_AMAFCA.pdf">http://www.berncogov/uploads/FileLinks/cbb8af72471e4746ba9e92e9a67318bb/Sediment and Erosion Design Guide AMAFCA .pdf</a>		

**Contraction Scour – Live Bed or Clear Water**

Contraction scour may be based on live bed or clear water conditions based on the computed critical velocity of the  $D_{50}$  particles (50% of the sediment sample is finer by weight) within the simulation. Live bed conditions were assumed for scour calculations as the channel velocity was higher than the critical velocity. **Table 2** documents the computations and determination that live bed equations should be applied.

The 100-year scour depth calculations are based on Laursen's Equations as outlined in the Hydraulic Engineering Circular No. 18 (HEC 18) while the 500-year contraction scour calculations were based on the Live Bed Pressure Flow Scour Method outlined in HEC-18.

**Table 3** summarizes the 100-year Design Flood live bed calculations and **Table 4** summarizes the 500-year Check Flood live bed pressure flow calculations as the bridge will experience pressure flow during the 500-year flood.

Note that the 100-year storm water surface is at the bridge low chord elevation. The proposed concrete bridge barrier railing top elevation is several feet above the road surface elevations on each side of the bridge. The 500-year flood water surface elevation will exceed the bridge deck elevation and cause pressure flow, and excess flow will spill around the concrete wall barrier prior to overtopping the barrier. The 100-year contraction scour depth is 4.2 ft and the 500-year contraction scour depth is 19.0 ft as presented in **Tables 3** and **4**, respectively.

**Existing Scour Hole**

The existing scour hole depth from existing bridge deck was measured at about 6.5 feet by Smith Engineering Company. The computed 100-year contraction scour depth of 4.2 ft below the existing bed elevation seems reasonable based on depth of the existing scour hole.

**TABLE 2****Determination of Either Clear Water or Live Bed Contraction Scour****NM 456, Proposed Bridge # 4920 (Dry Cimarron River)**

Use HEC-18 Equation 6.1 - page 6.2 (1)

Conclusion based on : If  $V_c < \text{Velocity of channel}$ , then "Live Bed", else "Clear Water"

Design Storm	River Sta	Y	D <sub>50</sub>	K <sub>u</sub>	Velocity of Channel	V <sub>c</sub>	CONCLUSION
	River Sta	(ft)	(ft)		(ft/s)	(ft/s)	
		a	b	c	a	d	
100-hr, 24-hr	1800	24.2	0.0164	11.17	13.95	4.83	Use Live Bed Scour Equation
500-yr, 24-hr	1800	41.6	0.0164	11.17	10.58	5.28	Use Live Bed Scour Equation

a - Obtained from HEC-RAS summary results

b - Obtained from bed material gradation data, D<sub>50</sub> = 5 mm, = 0.0164 ft

c - Coefficient from HEC-18, pg. 6.2

d - HEC-18 equation 6.1, pg. 6.2

$$V_c = K_u y^{1/6} D^{1/3}$$

**Definition of Variables**V<sub>c</sub> = critical velocity above which bed material for size D and smaller will be transported, ft/s

y = average depth of flow upstream from the bridge, ft

D = particle size for V<sub>c</sub>, ftD<sub>50</sub> particle size in a mixture of which 50% are smaller, ftK<sub>u</sub> = 11.17 English units

1 - Evaluating Scour at Bridges (Fifth Edition), Hydraulic Engineering Circular No. 18. U.S. Department of Transportation Federal Highway Administration, National Highway Institute. Publication No. FHWA-HIF-12-003, April 2012.

**TABLE 3**

**Live Bed Contraction Scour Computation for NM 456,  
Proposed Bridge # 4920 - Dry Cimarron River, 100-year Design Flood**

**NOTE - Ys negative means no scour**

Use HEC-18 Equation 6.2 - Modified Laursen Equation (1)

Bridge low chord elev. = 5146.00, high chord elev. = 5153.88

									A			B	= A/B		
Y <sub>0</sub>	Y <sub>1</sub>	Y <sub>2</sub>	Q <sub>1</sub>	Q <sub>2</sub>	W <sub>1</sub>	W <sub>2</sub>	k <sub>1</sub>	S <sub>1</sub>	g	V*	ω	ω	V*/ω	Y <sub>s</sub>	
at RS 1779.63	at RS 1800														
ft	ft	ft	ft <sup>3</sup> /s	ft <sup>3</sup> /s	ft	ft		ft/ft	ft/s <sup>2</sup>	ft/s	m/s	ft/s		ft	
a	a	g	a	a	b	b	c	a		d	e	e		f	

RS - HEC-RAS River Section. RS 1779.63 is 10 ft upstream of upstream bridge deck, deck is 33.7 ft wide

**100-year storm - Design Flood - Water Surface Summary - The 100-yr. flood water surface will pass under the low chord, just barely**

23.0	24.2	27.2	21,417	21,417	30.0	25.0	0.64	0.002	32.20	1.3	0.3500	1.1	1.1	4.2	
------	------	------	--------	--------	------	------	------	-------	-------	-----	--------	-----	-----	-----	--

**500-year storm - Check Flood. Contraction Scour computed with Pressure Scour Method due to overtopping**

#### FOOTNOTES

1 - Evaluating Scour at Bridges (Fifth Edition), Hydraulic Engineering Circular No. 18. U.S. Department of Transportation Federal Highway Administration, National Highway Institute. Publication No. FHWA-HIF-12-003, April 2012.

a- HEC-RAS Output, Sect. 1779.63 is 10 ft upstream from bridge deck

b- From a 1 ft accurate contour map

c - Assume water temperature = 20 degrees celsius

#### c -- Determining Exponent k<sub>1</sub>

V*/ω	k <sub>1</sub>	Mode of Bed Material Transport
< 0.5	0.59	mostly contact bed material discharge
0.5 - 2.0	0.64	some suspended bed material discharge
>2.0	0.69	mostly suspended bed material discharge

d-  $V^* = (g \cdot Y_1 \cdot S_1)^{0.5}$

e - HEC-18, Figure 6.8, Page 6.11. Multiply m/s by 3.28 to obtain ft/s

e - From bed material gradation curve plot (D<sub>50</sub>=5 mm)

f - HEC-18 Equation 6.3, page 6.10 :  $Y_s = Y_2 - Y_0$

g -LIVE BED CONTRACTION SCOUR HEC-18 Equ. 6.2 (Modified Laursen Eq.)

$$(Y_2 / Y_1) = [(Q_2 / Q_1)]^{6/7} * [(W_1 / W_2)]^{k_1}$$

Note - HEC-18 pg. 6.10 has an error, fall velocity "ω" is incorrectly labeled as "T"

#### Definition of Variables (see Footnote Ref. 1 page 6.10)

Y<sub>s</sub> = Y<sub>2</sub> - Y<sub>0</sub> = average contraction scour depth, ft

Y<sub>0</sub> - existing depth in the contracted section before scour, ft

Y<sub>1</sub> - average depth in the upstream channel, ft

Y<sub>2</sub> - average depth in the contracted section, ft

Q<sub>1</sub> - flow in the upstream channel transporting sediment, ft<sup>3</sup>/s

Q<sub>2</sub> - flow in the contracted channel, ft<sup>3</sup>/s

W<sub>1</sub> - bottom width of the upstream channel transporting sediment, ft

W<sub>2</sub> - bottom width of main channel in contracted section less pier widths, ft

k<sub>1</sub> - exponent

V\* - shear velocity in the upstream section, ft/s

ω - fall velocity of bed material based on D<sub>50</sub> from HEC-18, Figure 6.8

g - gravitational acceleration, ft/s<sup>2</sup>

S<sub>1</sub> - slope of the energy grade line in main channel, ft/ft

**Cell requires data input**

**Cell uses equations for computation**

TABLE 4																														
Live Bed Pressure Flow Contraction Scour Computation for NM 456, Proposed Bridge # 4920 - Dry Cimarron River, 500-year Check Flood												NOTE - Ys negative means no scour					Pressure Scour depth calculations for Live Bed with bridge overtopping in the 500-year storm													
Use HEC-18 Equation 6.2 - Modified Laursen Equation (1) to Compute Y <sub>2</sub>																														
Scour computed using HEC-18 Pressure Scour Method for Bridge Overtopping, Section 6.10 pg. 6.24																	Bridge low chord elev. = 5146.00, high chord elev. = 5153.88													
Solve for Y <sub>2</sub> HEC-18 Eq. 6.2 now using Q <sub>ue</sub> , instead of Q <sub>1</sub>																		Pressure Scour Depth = Y <sub>2</sub> + t - h <sub>b</sub>		Y <sub>s</sub>										
h <sub>ue</sub> at RS 1779.63	Y <sub>1</sub> = h <sub>u</sub> at RS 1800	Y <sub>2</sub>	Q <sub>1</sub>	Q <sub>ue</sub>	Q <sub>2</sub>	W <sub>1</sub>	W <sub>2</sub>	k <sub>1</sub>	S <sub>1</sub>	g	V*	ω	ω	V*/ω	h <sub>b</sub>	h <sub>t</sub>	h <sub>u</sub>	h <sub>w</sub>	t	Y <sub>s</sub>										
ft	ft	ft	ft <sup>3</sup> /s	ft <sup>3</sup> /s		ft	ft		ft/ft	ft/s <sup>2</sup>	ft/s	m/s	ft/s		ft	ft	ft	ft	ft	ft										
a	a	g	a	a	a	a	a	c	a		d	e			a	a	a	a												
RS = HEC-RAS River Section																														
500-year storm - Check Flood. Contraction Scour computed with Pressure Scour Method due to overtopping.																														
27.5	40.70	30.4	30,000	19,212		30.0	26.0	0.64	0.001	32.20	1.2	0.3500	1.148	1.0	19.65	21.05	40.70	13.17	8.21	19.0										
FOOTNOTES																														
1 - Evaluating Scour at Bridges (Fifth Edition), Hydraulic Engineering Circular No. 18. U.S. Department of Transportation Federal Highway Administration, National Highway Institute. Publication No. FHWA-HIF-12-003, April 2012. a- HEC-RAS Output, Section 1779.63 is 10 ft upstream from bridge deck b- From a 1 ft accurate contour map																														
c -- Determining Exponent k <sub>1</sub>																	Definition of Variables (see Ref. 1 page 6.10)							Definition of Variables (see Reference 1, page 6.24-6.25)						
V*/ω      k <sub>1</sub> Mode of Bed Material Transport																	Y <sub>1</sub> - average depth in the upstream channel, ft							h <sub>b</sub> - vertical size of the bridge opening prior to scour, ft						
< 0.5      0.59      mostly contact bed material discharge																	Y <sub>2</sub> - average depth in the contracted section, ft							h <sub>t</sub> - distance from the water surface to the lower face of the bridge girders, equals h <sub>u</sub> - h <sub>b</sub> , ft						
0.5 - 2.0      0.64      some suspended bed material discharge																	Q <sub>1</sub> - flow in the upstream channel transporting sediment, ft <sup>3</sup> /s							h <sub>ue</sub> - effective upstream channel flow depth for live-bed conditions and bridge overtopping, ft						
> 2.0      0.69      mostly suspended bed material discharge																	Q <sub>2</sub> - flow in the contracted channel, ft <sup>3</sup> /s							h <sub>u</sub> - upstream channel flow depth as defined for Equation 6.2 (y <sub>1</sub> ), ft						
d- V* = (g*Y <sub>1</sub> *S <sub>1</sub> ) <sup>0.5</sup>																	W <sub>1</sub> - bottom width of the upstream channel transporting sediment, ft							h <sub>w</sub> - weir flow height = h <sub>t</sub> - T for h <sub>t</sub> > T, h <sub>w</sub> = 0 for h <sub>t</sub> ≤ T, ft						
e - HEC-18, Figure 6.8, Page 6.11. Multiply m/s by 3.28 to obtain ft/s																	W <sub>2</sub> - bottom width of main channel in contracted section less pier widths, ft							T - distance from the low chord to high chord, ft (not used here)						
e - From bed material gradation curve plot (D <sub>50</sub> =5 mm)																	k <sub>1</sub> - exponent							t - separation zone thickness, ft						
f - not used																	V* - shear velocity in the upstream section, ft/s							t = 0.5 ((h <sub>b</sub> *h <sub>u</sub> /(h <sub>u</sub> <sup>2</sup> )) <sup>0.2</sup> * (1 - h <sub>w</sub> /h <sub>u</sub> ) <sup>0.1</sup> * h <sub>b</sub>						
g-LIVE BED CONTRACTION SCOUR HEC-18 Equ. 6.2 (Modified Laursen Eq.)																	ω - fall velocity of bed material based on D <sub>50</sub> from HEC-18, Figure 6.8							Y <sub>s</sub> = Y <sub>2</sub> + t - h <sub>b</sub> , ft						
(Y <sub>2</sub> /Y <sub>1</sub> ) = [(Q <sub>2</sub> /Q <sub>1</sub> )] <sup>1/6</sup> * [(W <sub>1</sub> /W <sub>2</sub> )] <sup>1/6</sup>																	g - gravitational acceleration, ft/s <sup>2</sup>							Q <sub>ue</sub> - effective channel discharge for live bed conditions and bridge overtopping, ft <sup>3</sup> /s						
Note - HEC-18 pg. 6.10 has an error, fall velocity "ω" is incorrectly labeled as "T"																	S <sub>1</sub> - slope of the energy grade line in main channel, ft/ft							Q <sub>ue</sub> = Q <sub>1</sub> *(h <sub>ue</sub> /h <sub>u</sub> )^1.14, ft <sup>3</sup> /s						
																	Cell requires data input							h <sub>u</sub> = Y <sub>1</sub> - average depth in the upstream channel, ft						
																	Cell uses equations for computation													

**Pier Scour**

Pier scour was computed with a HEC-18 equation based on the Colorado State University pier scour equation. Calculations were necessary as the proposed bridge will have one set of piers. The 100-year and 500-year pier scour depths are predicted to be 4.8 ft and 4.8 ft as presented in **Tables 5 and 6**. Note that these calculations use the allowed rule-of-thumb, HEC-18 equation 7.2, due to the use of a round pier nose. Use of other pier nose shapes would require a different K1 value, and yield different results for the design and check floods.

TABLE 5									
Pier Scour by the HEC-18 Equation 7.1									
CN:		NM 456, Bridge 4920			By:		Dry Cimarron River		
Subject:									
Design Flood 100-year									
HEC-18 simple pier scour analysis							Pier Description		
<div><math display="block">\frac{y_s}{y_1} = 2.0 K_1 K_2 K_3 \left( \frac{a}{y_1} \right)^{0.65} Fr_1^{0.43}</math></div> <div>Pier Scour, eq. 7.1</div>							round nose (K1 coeff.)		
							angle of attack = 0 deg (K2 coeff.)		
							Assume small dunes (K3 coeff.)		
							Pier is 26 ft long		
24.20	ft	y1	Flow depth directly upstream of the pier						
		Flow depth taken from Contraction Scour, or enter depth from HEC-RAS							
13.0	ft/sec	V1	Mean velocity of flow directly upstream of the pier						
1.0	--	K1	Correction factor, pier nose shape, Figure 7.3 and Table 7.1						
2	ft	a <sub>pier</sub>	Pier width						
0 degrees		angle of attack on pier (set K1=1.0 if angle > 5)							
26.00	ft	L	Length of pier (only needed if "angle of attack on pier" is > 0)						
13.0	ft/ft	L/a <sub>pier</sub>							
1.0	--	K2	Correction factor, angle of attack of flow, Table 7.2 or Equation 7.4						
1.1	--	K3	Correction factor for bed condition from Table 7.3						
32.2	ft/s <sup>2</sup>	g	Acceleration of gravity						
0.466	--	Fr	Froude Number directly upstream of the pier = V1/[(gy1)^(1/2)]						
12.1	ft/ft	y1/a <sub>pier</sub>	ratio to determine shallow flow if (y/a)<0.8						
0.31	ft/ft	y <sub>s</sub> /y1	(y <sub>s</sub> pier)/y1; intermediate ratio in calculations						
7.58	ft	y <sub>s</sub> pier	Scour component for the pier stem in the flow=(y <sub>s</sub> /y1)*y1						
USE RULE OF THUMB MAXIMUM if round nose pier									
4.8	ft	y <sub>s</sub> Max	max scour based on rule of thumb below						
Angle of Attack must equal zero, and pier has <b>round nose</b>									
As a Rule of Thumb, the maximum scour depth for round nose piers aligned with the flow is:									
y <sub>s</sub> ≤ 2.4 times the pier width (a) for Fr ≤ 0.8 (7.2)									
y <sub>s</sub> ≤ 3.0 times the pier width (a) for Fr > 0.8									

CN:	NM 456, Bridge 4920	By:	Dry Cimarron River
Subject:			
<b>Design Flood 500-year</b>			
HEC-18 simple pier scour analysis			<b>Pier Description</b>
<div style="border: 1px solid black; padding: 10px; display: inline-block;"> <math display="block">\frac{y_s}{y_1} = 2.0 K_1 K_2 K_3 \left( \frac{a}{y_1} \right)^{0.65} Fr_1^{0.43}</math> </div>			round nose (K1 coeff.)
			angle of attack = 0 deg (K2 coeff.)
			Assume small dunes (K3 coeff.)
			Pier is 26 ft long
40.70 ft	y1	Flow depth directly upstream of the pier	
		Flow depth taken from Contraction Scour, or enter depth from HEC-RAS	
16.0 ft/sec	V1	Mean velocity of flow directly upstream of the pier	
1.0 --	K1	Correction factor, pier nose shape, Figure 7.3 and Table 7.1	
2 ft	a <sub>pier</sub>	Pier width	
0 degrees		angle of attack on pier (set K1=1.0 if angle > 5)	
26.00 ft	L	Length of pier (only needed if "angle of attack on pier" is > 0)	
13.0 ft/ft	L/a <sub>pier</sub>		
1.0 --	K2	Correction factor, angle of attack of flow, Table 7.2 or Equation 7.4	
1.1 --	K3	Correction factor for bed condition from Table 7.3	
32.2 ft/s <sup>2</sup>	g	Acceleration of gravity	
0.442 --	Fr	Froude Number directly upstream of the pier = V1/[(gy1) <sup>(1/2)</sup> ]	
20.4 ft/ft	y1/a <sub>pier</sub>	ratio to determine shallow flow if (y/a)<0.8	
0.22 ft/ft	y <sub>s</sub> /y1	(y <sub>s</sub> pier)/y1; intermediate ratio in calculations	
8.89 ft	y <sub>s</sub> pier	Scour component for the pier stem in the flow=(y <sub>s</sub> /y1)*y1	
<b>USE RULE OF THUMB MAXIMUM if round nose pier</b>			
4.8 ft	y <sub>s</sub> Max	max scour based on rule of thumb below	
Angle of Attack must equal zero, and pier has <b>round nose</b>			
As a Rule of Thumb, the maximum scour depth for round nose piers aligned with the flow is:			
$y_s \leq 2.4 \text{ times the pier width (a) for } Fr \leq 0.8$			
$y_s \leq 3.0 \text{ times the pier width (a) for } Fr > 0.8$			

## **Abutment Scour**

Abutment Live Bed scour calculations may be computed using several methods. The NCHRP 24-20 Method (HEC-18, 2012) was selected.

### **NCHRP 24-20 Abutment Scour Method**

The NCHRP 24-20 Method was applied for the 100-year Design Flood and 500-year Check Flood. **Table 7** presents the 100-year Design Flood scour data and results and **Table 8** contains the 500-year Check Flood data and results. The predicted 100-year and 500-year abutment scour depths are -14.8 ft and -16.4 ft, respectively. However, HEC-18, page D.14 describes that negative values are to be assumed as zero scour.

Note that this Method includes contraction scour as part of the predicted abutment scour depth. The contraction scour previously computed may be reviewed for comparison to the scour computed with this Method. However only the result from this Method should be combined with the long term degradation to compute the total scour at a bridge abutment.

### **Excluded Abutment Scour Depth**

The NCHRP 24-20 method values for this proposed bridge were determined to be negative and are therefore assume to be zero as described previously. Also note the following regarding the total scour depth computation.

- Field observations of the existing bridge abutments revealed that only minor scour has occurred adjacent to the east abutment, no scour was observed adjacent to the west abutment.
- There was evidence of a contraction scour hole in the main channel. The depth of the existing scour hole was measured by Smith Engineering Company relative to the bridge deck and the channel invert depth (contraction/pier scour hole depth) relative to the low flow banks at the water's edge was measured at 6.5 ft. This depth fit well with contraction scour calculations performed by Smith that computed a 100-year contraction scour depth of 4.2 ft.
- The fifth edition of HEC-18, April 2012, describes on page 8.4 that "Contraction Scour should be viewed as the reference scour depth for abutment scour". However, as noted previously, the NCHRP 24-20 Method does include contraction scour.



### TABLE 7

### Abutment Scour by the NCHRP 24-20 Method - 100-Year Design Flood

CN:	NM 456, Bridge: 4920	By:	Dry Cimarron River
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Design Flood: 100-year

See HEC-18, April 2010, Section 8.6.3, NCHRP 24-20 Abutment Scour Approach pg. 8.8

Live/Clear calculation rows show/hide based on result of Row 9, based on L/Bf value

LEFT	RIGHT						
80	80 ft	L	projected length of the embankment				
80	80 ft	Bf	width of the floodplain				
1.00	1.00 ft/ft	L/Bf	ratio				
LIVE	LIVE						
<b>LIVE-BED SCOUR EQUATION (<math>0.75 \leq L/BF \leq 1.0</math>)</b>							
21,417	21,417 cfs	Q2	Flow under bridge				
89.2	89.2 ft	W2	Bridge opening width				
240.10	240.10 cfs/ft	q <sub>2c</sub>	Unit discharge in the constricted opening accounting for non-uniform flow distribution				
21,417	21,417 cfs	Q1	Upstream flow in main channel (excludes Q <sub>f</sub> , floodplain flow)				
24	24 ft	W1	Upstream width of main channel (exclude floodplains)				
892.38	892.38 cfs/ft	q <sub>1</sub>	Upstream main channel unit discharge				
21.06	21.06 ft	y <sub>1</sub>	Average Upstream flow depth in main channel				
0.27	0.27 --	q <sub>2</sub> /q <sub>1</sub>	ratio to read graph				
6.84	6.84 ft	y <sub>c</sub>	Flow depth including live-bed contraction scour (eq. 8.5)				
s	s --	W or S	Wingwall or spillthrough?				
1.20	1.20 --	α alpha	<b>Read from Fig (8.9, spill) or (8.10, wing)</b>				
8.20	8.20 ft	y <sub>max</sub>	Maximum flow depth resulting from abutment scour				
23.03	23.03 ft	y <sub>o</sub>	Flow depth prior to scour				
-14.8	-14.8 ft	y <sub>s</sub>	Abutment scour depth				
<b>NOTE - Negative Y<sub>s</sub> values are interpreted as zero scour, See HEC-18, pf.D.14</b>							

### TABLE 8

### Abutment Scour by the NCHRP 24-20 Method - 500-Year Check Flood

CN:	NM 456, Bridge: 4920	By:	Dry Cimarron River
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Check Flood: 500-year	
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See HEC-18, April 2010, Section 8.6.3, NCHRP 24-20 Abutment Scour Approach pg. 8.8

Live/Clear calculation rows show/hide based on result of Row 9, based on L/Bf value

LEFT	RIGHT								
80	80 ft	L	projected length of the embankment						
80	80 ft	Bf	width of the floodplain						
1.00	1.00 ft/ft	L/Bf	ratio						
LIVE	LIVE								
<b>LIVE-BED SCOUR EQUATION (<math>0.75 \leq L/BF \leq 1.0</math>)</b>									
21,417	21,417 cfs	Q2	Flow under bridge						
89.2	89.2 ft	W2	Bridge opening width						
240.10	240.10 cfs/ft	q2c	Unit discharge in the constricted opening accounting for non-uniform flow distribution						
30,000	30,000 cfs	Q1	Upstream flow in main channel (excludes Qf, floodplain flow)						
24	24 ft	W1	Upstream width of main channel (exclude floodplains)						
1250.00	1250.00 cfs/ft	q1	Upstream main channel unit discharge						
38.00	38.00 ft	y1	Average Upstream flow depth in main channel						
0.19	0.19 --	q2/q1	ratio to read graph						
9.24	9.24 ft	yc	Flow depth including live-bed contraction scour (eq. 8.5)						
s	s --	W or S	Wingwall or spillthrough?						
1.20	1.20 --	$\alpha$ alpha	<b>Read from Fig (8.9, spill) or (8.10, wing)</b>						
11.09	11.09 ft	y <sub>max</sub>	Maximum flow depth resulting from abutment scour						
27.50	27.50 ft	y <sub>o</sub>	Flow depth prior to scour						
-16.4	-16.4 ft	y <sub>s</sub>	Abutment scour depth						
<b>NOTE - Negative Ys values are interpreted as zero scour, See HEC-18, pf.D.14</b>									

**TOTAL SCOUR RESULTS SUMMARY and PROPOSED SCOUR COUNTERMEASURES****SUMMARY**

Assume that the existing scour hole depth of 6.5 ft is the result of contraction and/or pier scour. The computed contraction scour and pier scour depths are summarized in **Table 9**. The proposed scour counter measure is wire enclosed riprap located on each abutment slope, that will be tied to the existing pier foundations that will remain (the two existing piers will be removed). The wire enclosed riprap and existing bridge pier foundations will be sufficient to prevent abutment scour and will also prevent contraction scour from undermining the abutments. The proposed bridge abutments will be set on deep driven piles.

TABLE 9

Dry Cimarron River/Bridge Data and Scour Results Summary - NM 456, Proposed Bridge # 4920

Dry Cimarron River, Drainage Area = 295 sq mi				Storm Return Period (24-hour storms)			
ITEM (at upstream side of bridge)	foot notes		Units	25-year	50-year	100-year	500-year
Design Discharge	--	--	cfs	10,726	15,372	21,417	30,000
Bridge High Chord Elevaiton	--	--	ft	5,153.88	5,153.88	5,153.88	5,153.88
Bridge Low Chord Elevation	--	--	ft	5,146.00	5,146.00	5,146.00	5,146.00
Upstream Water Surface Elevation	--	--	ft	5,141.78	5,144.83	5,146.02	5,153.88
Freeboard	--	--	ft	4.22	1.17	NR	NR
Average Flow Depth	--	--	ft	NR	NR	23.03	38
Average Velocity	--	--	ft/s	NR	NR	13	16
Long Term Aggradation/Degradation Depth	a	A	ft	NR	NR	0	0
Contraction Scour Depth	e	B	ft	NR	NR	4.2	19.0
Pier Scour Depth	c	C	ft	NR	NR	4.8	4.8
Abutment Scour Depth (Left) NCHRP 24-20 Method (includes contraction scour)	e	D	ft	NR	NR	0.0	0.0
Abutment Scour Depth (Right) NCHRP 24-20 Method (includes contraction scour)	e	D	ft	NR	NR	0.0	0.0
COMPUTED TOTAL SCOUR DEPTH below existing contraction scour elevation - <b>See Explanation</b>	e	E = (A+B+C+D)	ft	NR	NR	9.0	23.8

EXPLANATION - The existing scour hole depth from existing bridge deck was measured to be 6.5 ft by Smith Engineering Company. The computed 100-yr. contraction scour depth of 4.2 ft below the existing bed elevation seems reasonable based on depth of the existing scour hole. Note that the 100-yr storm water surface is at the bridge low chord elevation. The proposed concrete bridge barrier railing top elevation is several feet above the road surface elevations on each side of the bridge. Therefore the 500-yr flood water surface elevation will spill around the concrete wall barrier prior to overtopping the barrier. The 500-yr water surface will cause pressure flow.

NR - NMDOT drainage criteria does not require scour calculations for the 25-yr or 50-yr floods, freeboard not required for the 100-yr and 500-yr floods.

a - Watercourse is at approximately equilibrium slope therefore no aggradation or degradation (if aggradation, do not add to other scour depths).

c - One pier at the proposed bridge.

e - Abutment scour values are excluded from total scour depths for several reasons: 1. Because the values were computed to be negative and these are interpreted as zero scour values per HEC-18. 2. Observations of existing abutments - existing bridge and abutments have been in place for many years, and the unlined slopes adjacent to the abutments and the east abutment, only show evidence of minor abutment scour. 3. HEC-18, April 2012 pg. 8.4 states that contraction scour depths should act as the reference scour depths for abutments.

# **APPENDIX 9 EXAMPLE CORROSION RESISTANCE NUMBER AND ABRASION CALCULATIONS**

## **CONTENTS:**

**Example Problem 9-1 Pipe Corrosivity and Abrasion**

**Example Problem 9-1 (Pipe Corrosivity and Abrasion)**

**Note:** The equation, figure, and table numbers referenced with 3 digits (ex. Equation 801-2) refer to the same item numbers identified within the Drainage Design Manual.

Problem Information

A 36 inch culvert is needed at a new road site. Tests done at the site show that resistivity is 1000  $\Omega$ -cm and pH is 7.9. Initial hydraulic assessment calculates the 5-year frequency velocity through the culvert as 12 ft/s. There is gravel with some rock evident in the channel.

An 18 gage 2-oz. galvanized metal culvert is initially selected for this culvert. Since this is a galvanized culvert and pH is greater than 7.3 use **Equation 801-2**.

$$\text{EMSL} = 3.82 R^{0.41}$$

**801-2**

$$\text{EMSL} = 3.82 (1000)^{0.41}$$

$$\text{EMSL} = 65 \text{ years}$$

From **Table 801-2**, the factor under an 18 gage galvanized culvert is 0.7, so the adjusted

$$\text{EMSL} = 65 (0.7) = 45 \text{ years}$$

Note that 45 years is less than 50-year minimum service life limit placed on an NMDOT culvert, so therefore it is unacceptable.

The next thicker gage is 16. **Table 801-2** has a factor of 1, so EMSL for an 18 gage 2-oz. galvanized metal culvert is 65 years. This meets the service life requirement.

However, **Table 801-1** shows that resistivity of less than or equal to 1000  $\Omega$ -cm is good only for culverts in the CR4 through CR7 range. That leaves aluminum alloy, polymetric galvanized steel, Aramid bonded galvanized steel, concrete pipe and HDPE as viable choices.

Next, check on potential abrasion. Since calculated velocities are 12 ft/s and channel has gravels with some rock select 'Level 3' (Moderate Abrasion) from **Table 801-3**. All of the items mentioned above are still viable with the exception of aluminum alloy where it is recommended thickening the wall of the culvert by one gage. A literature search reveals that the smallest gage available for aluminum alloy is 16 gage. The next size up is a 14 gage.

Therefore, selection of a 36 inch culvert can be from an aluminum alloy (14 gage), polymetric galvanized steel (16 gage), Aramid bonded galvanized steel (16 gage), concrete pipe or HDPE pipe.

# **APPENDIX 10 SUMMARY OF RESEARCH**

**This Appendix contains tables that summarizes research conducted for other selected state DOTs and selected drainage/flood control organizations.**

**This effort was conducted as the starting point for development of this Drainage Design Manual. Other DOT drainage manuals were reviewed for drainage design criteria and methods and compared to the NMDOT criteria and methods prior to this current Drainage Design Manual update.**

**NMDOT  
DRAINAGE DESIGN MANUAL  
SUMMARY OF  
RESEARCH AND IDENTIFICATION**

Prepared for

NMDOT

Prepared by

Smith Engineering Company

And

OCCAM-EC Consulting Engineers

**September 1, 2015**



## APPENDIX 10.1

TABLE 5 Survey of State and Drainage  
Organizations - Hydrologic Methods  
Outside New Mexico

TABLE 6 Survey of State and Drainage  
Organizations - Hydraulic Methods Outside  
New Mexico

**TABLE 5**  
**Survey of State and Drainage Organizations - Hydrologic Methods Outside New Mexico**

HYDROLOGY		Selected Western DOT's and Flood Control Organizations											
Topic	Details	Current NMDOT	Proposed NMDOT	Arizona DOT	CalTrans	Colorado DOT	Nevada DOT	Texas DOT	Utah DOT	Wyoming DOT	Maricopa County Flood Control	Denver Urban Drainage District	Clark County Regional Flood Control District
Manual Version/Date		Drainage Manual Volume I, Hydrology 1995		Second Edition January 2014 (Draft)	Highway Design Manual, Chapter 810, Hydrology July 1, 2015	Drainage Design Manual Chapter 7 Hydrology 2004	Drainage Manual 2nd Addition December 2006	Hydraulic Design Manual May 2014	UDOT Drainage Manual January 2004 (US Customary Edition) Hydrology Chapter Updated Sept 28, 2005	Chapter 3, Section 3-04 Culvert Design August 2011	Drainage Design Manual for Maricopa County Hydrology August 2013	Urban Drainage and Flood Control District Drainage Criteria Manual 2008	Hydrologic Criteria and Drainage Design Manual August 12, 1999
Time of Concentration	Kirpich	Yes		No	No	No	No	Yes	No	No	No	No	No
	Stream Hydraulic Method	Yes		No	No	No	No	No	No	No	No	No	No
	Kinematic Wave Method	No		No	Yes	No	No	No	No	No	No	No	No
	NRCS Upland Method	Yes		No	Yes	Yes	Yes	Yes	Yes	Yes, sort of?	No	No	No
	Other	As and If Specified in Urban Areas		Papadakis and Kazan, 1987	No	No	No	Kerby-Kirpich Method, Kerby	NRCS Curve Number Method	Method embedded in CDS	Papadakis and Kazan, 1987	Denver Urban Method (similar to NRCS)	Clark County Urban Method,
Rainfall/Runoff	Rational Formula C	from ADOT Tables based on NRCS HSG		ADOT Tables based on NRCS HSG	CalTrans Specified Values	Urban Storm Drainage Criteria Manual 2001 (Denver)	NA	Table in Tx DOT Manual	UDOT Table in Manual	Table in Wyoming DOT Manual	Table in Manual	Table in Manual (Uses NRCS Soil Hydrologic Groups)	Modified Rational Formula with Clark County C Factors
	Greene Ampt	No		Yes	No	No	No	Yes	No	No	Yes	No	No
	Initial and Constant Loss	No		Yes	No	No	No	Yes - Texas Method	No	Method embedded in	Yes	No	Ok, but not
	NRCS Curve Number	Yes		No	Yes	Yes	Yes	Yes - Texas Adjustment	Yes	Yes	Yes	No	Yes, Values Specified in
	Rainfall Distribution	NOAA Atlas 2, NRCS Type IIa 150 Ac		HEC-HMS 24 Hr Frequency Storm at 160 Ac	NOAA HDRO40 Tool for Desert Areas 320 Ac	NOAA Atlas 2?	NOAA Atlas 14 except Clark County 200 Ac	Texas Storm Hyetographs (Asquith et	NRCS Type II 300 Ac	None listed	NRCS Type II 160 Ac	NOAA Atlas 160 Ac	NRCS Type II plus three special distributions for 150 Ac
Hydrologic Method	Rational Formula (area limits)	Yes		No	No	160 Ac	No	200 Ac	300 Ac	Yes, No Limit Listed	No	No	No
	NRCS NM Chapter 2	Yes		No	No	No	No	No	No	No	No	No	No
	Parametric and Synthetic UH	No		Yes	Yes	No	Yes	Yes	Yes	No	No	No	No
	NRCS Unit Graph	Yes		No	Yes	Yes	Yes	?	Yes	No	No	No	Yes
	Snyder's Unit Graph	No		No	Yes	Yes	No	?	No	No	No	No	No
	Clark Unit Hydrograph	No		Yes	Yes	Yes	No	?	No	No	Yes	No	No
	ModClark Model	No		No	No	No	No	?	No	No	No	No	No
	Kinematic-wave Model	No		No	Yes	No	No	?	No	No	No	No	No
	Improved Highway Design Method for Desert Storms	No		No	Yes	No	No	No	No	No	No	No	Yes
	S-Graph Method	No		Yes	Yes	No	No	No	No	No	Yes	No	No
	CUHP	No		No	No	No	No	No	No	No	No	Yes	No
	Flood Frequency Analyses	Yes		Yes	Yes	Yes	No	Yes	Yes	No	Yes	No	No
Routing Routines	USGS Regression	Yes		Yes	Yes	?	?	Yes	Yes	Yes	Yes	No	No
	Muskingum	No		No	Yes	No	No	?	No	No	Yes	No	Yes
	Muskingum-Cunge	Yes		Yes	Yes	No	No	?	?	No	Yes	No	Yes
	Normal Depth Routing	No		No	No	No	No	No	No	No	Yes	No	No
	Kinematic Wave	No		Yes	Yes	No	No	?	?	No	Yes	Yes (within SWMM)	Yes
Computer Models and Tools	Modified Puls	No		Yes	Yes	No	No	?	?	No	No	No	No
	HEC-HMS	HEC-1		Yes	Yes	?	Yes	HEC-1	HEC-1	No	HEC-1	No	HEC-1
	Win TR-55	No		No	Yes	Yes	Yes	Yes	TR-55	No	No	No	TR-55
	Win TR-20	TR-20		No	Yes	?	?	?	TR-20	No	No	No	TR-20
	FLO-2D	No		Yes	No	No	No	?	?	No	No	No	No
	SWMM	No		No	No	Yes	No	?	?	No	No	Yes	No
	CDS	No		No	No	No	No	No	No	Yes	No	No	No
	CUHP	No		No	No	No	No	No	No	No	Yes	No	No
	HYMO 1993	Yes		No	No	No	No	No	No	No	No	No	No
	SCSHYDRO 1993	Yes		No	No	No	No	No	No	No	No	No	No
	WMS	No		No	Yes	No	No	?	Yes	No	No	No	No
Miscellaneous Other	HEC-GeoHMS	No		Yes	Yes	Yes	?	?	?	No	No	No	HEC-1
	Transmission Losses	No		Yes	No	No	No	No	?	No	No	No	No
	NRCS Graphical Peak Discharge Method	No		No	No	No	No	No	Yes	No	Yes	No	Yes
	Unit Hydrograph Methods	No		No	No	No	No	No	Special UDOT Method	No	No	No	No
	Log-Pearson Type III Distr.	No		No	Yes	Yes	No	Yes	Yes	No	No	No	No
	Log-normal Distr.	No		No	Yes	No	No	No	No	No	No	No	No
	Omega EM Regression Eq.	No		No	No	No	No	Yes	No	No	No	No	No
	NFF Regression Equations	No		No	No	No	No	No	Yes	No	No	No	No
	UDOT Regression Equations	No		No	No	No	No	No	Yes	No	No	No	No
	Gumbell Extreme Value Distr.	No		No	Yes	No	No	No	No	No	No	No	No
	Channel Geometry	No		No	No	Yes	No	No	No	No	No	No	No
	Colorado Urban Hydrograph	No		No	No	No	No	No	No	No	No	Yes	No
	Paisechydrology	No		No	No	Yes	No	No	No	No	Yes	No	No
	Level Pool Storage Routing	No		Yes	No	No	No	No	No	No	No	Yes	No
Weblink to Document	Rainfall Adjustments	NRCS NM Type IIa (65,70,75 & 80)		ADOT Rainfall Averaging Tool	Desert Specific NOAA Method	No	No	Yes - Texas Method	No	No	No	Areal reduction	Yes, Clark County Specified
		<a href="http://dot.state.nm.us/transportation/nm-hydrology/Manual.pdf">http://dot.state.nm.us/transportation/nm-hydrology/Manual.pdf</a>		<a href="http://www.adot.gov/docs/default-source/roadway-engineering-library/2014_adot_hydrology_manual.pdf?sfvrsn=14">http://www.adot.gov/docs/default-source/roadway-engineering-library/2014_adot_hydrology_manual.pdf?sfvrsn=14</a>	<a href="http://www.dot.ca.gov/hq/opsd/hdm/pdf/english/v-bp0810.pdf">http://www.dot.ca.gov/hq/opsd/hdm/pdf/english/v-bp0810.pdf</a>	<a href="https://www.cdott.gov/programs/environmental/evaluation/documents/drainage-design-manual-chapter07_hydrology.pdf/view">https://www.cdott.gov/programs/environmental/evaluation/documents/drainage-design-manual-chapter07_hydrology.pdf/view</a>	<a href="https://nevadadot.com/About_NDOT/NDOT_Divisions/Engineering/Hydrology/Drainage_Manual.aspx">https://nevadadot.com/About_NDOT/NDOT_Divisions/Engineering/Hydrology/Drainage_Manual.aspx</a>	<a href="http://onlinemanuals.bddot.gov/bddotmanuals/hydro/manual_notice.htm">http://onlinemanuals.bddot.gov/bddotmanuals/hydro/manual_notice.htm</a>	<a href="http://www.udot.utah.gov/main?n=100.pq.0:1:1/V.826_">http://www.udot.utah.gov/main?n=100.pq.0:1:1/V.826_</a>	<a href="http://www.dot.state.wy.us/files/5w/sites/vydot/files/shared/Project%20Development/Road%20Design%20Manual_1-3-06%20%20Aug-2011.pdf">http://www.dot.state.wy.us/files/5w/sites/vydot/files/shared/Project%20Development/Road%20Design%20Manual_1-3-06%20%20Aug-2011.pdf</a>	<a href="http://fd.maricopa.gov/downloads/manuals/hydrology-manual.pdf">http://fd.maricopa.gov/downloads/manuals/hydrology-manual.pdf</a>	<a href="http://www.udfcd.org/downloads/critmanual/usdcm%20%20%201%20%20Dec%202011%20.pdf">http://www.udfcd.org/downloads/critmanual/usdcm%20%20%201%20%20Dec%202011%20.pdf</a>	<a href="http://questfont.ccrfcd.org/pdf_arch/hdddm/Current%20Manual%20Complete/hdddm.pdf">http://questfont.ccrfcd.org/pdf_arch/hdddm/Current%20Manual%20Complete/hdddm.pdf</a>
									Follows AASHTO Model Drainage Manual modified for UDOT				

**Table 6**  
**Survey of State and Drainage Organizations - Hydraulic Methods Outside New Mexico**

HYDRAULICS		Selected DOTs and Flood Control Organizations										
Topic	Details	Current NMDOT	Proposed NMDOT (Preliminary)	Arizona DOT	CalTrans	Colorado DOT	Nevada DOT	Texas DOT	Utah DOT	Wyoming DOT	Maricopa County Flood Control	Denver Urban Drainage Flood Control District
Manual Version/Date		Drainage Manual Volume 1, Hydrology 1995		Second Edition January 2014 (Draft)	Highway Design Manual, Chapter 810, Hydrology 1, 2015	Drainage Design Manual Chapter 7 Hydrology 2004	Drainage Manual 2nd Addition December 2006	Hydraulic Design Manual May 2014	UDOT Drainage Manual January 2004 (US Customary Edition) Hydrology Chapter Updated Sept 28, 2005	Chapter 3, Section 3-04 Culvert Design August 2011	Drainage Design Manual for Maricopa County Hydrology August 2013	Urban Drainage and Flood Control District Drainage Criteria Manual 2008
Open Channel Hydraulics	WSPRO (USGS)	yes		---	---	yes	---	yes	yes	---	---	---
	HEC-RAS (USACE) (New Version has 2D)	yes	YES	yes	yes	yes	yes	yes	yes	---	yes	yes
	HEC-15 (FHWA)	---		---	---	---	---	yes	---	---	---	---
	UNET (USACE)	---		---	---	---	---	---	---	---	---	---
	E431 (USGS)	---		---	---	---	---	---	yes	---	---	---
	WSP-2 (NRCS)	---		---	---	---	---	---	yes	---	---	---
	Hydraulic Tool Box (FHWA)	---		---	yes	---	---	---	---	---	---	---
	WSPG	---		---	---	---	yes	---	---	---	---	---
	UD - Channels Spreadsheet (Denver)	---		---	---	---	---	---	---	---	---	yes
Street Hydraulics	FESWMS (USGS for FHWA)	yes		---	yes	---	yes	---	yes	---	---	---
	HY-4 (FHWA)	---		---	---	---	---	---	---	---	---	yes
		---		---	---	---	---	---	---	---	---	---
Culvert Hydraulics	Gutter - Nomographs	yes	YES	---	---	---	---	---	yes	---	yes	---
	Gutter / Street Equations	---	YES	---	---	---	---	yes	---	---	---	yes
	Inlet Equations	---		---	---	---	---	---	---	---	---	yes
Storm Drain Inlet and Pipe Hydraulics	HY8 (FHWA)	---		yes	yes	---	yes	---	yes	---	---	yes
	CDS (Wyoming Culvert Design System)	---		---	---	---	---	---	yes	yes	---	---
	Nomographs - HDS 5 (FHWA)	yes		yes	yes	---	---	---	yes	yes	yes	yes
	UD-Culvert Spreadsheet (Denver)	---		---	---	---	---	---	---	---	---	yes
	Grate Inlets - Nomographs	---	YES	---	---	yes	---	---	yes	---	yes	---
Sedimentation & Erosion	Full Flow Storm Drains - Nomographs	---		---	---	---	---	---	yes	---	---	---
	HGL Calculations - Manual Table	yes		yes	---	yes	---	---	yes	---	yes	---
	Select CADD (Bently Software)	---		yes	---	---	---	---	yes	---	---	---
	StormCADD (Bently Software)	---	YES	yes	---	---	---	---	yes	---	---	---
	Flow Master (Bently - Haestad Software)	---	YES	yes	---	---	---	---	yes	---	---	---
	Equations	---		---	---	---	---	yes	---	---	---	---
	Hydra Flow (AutoDesk Civil 3D)	---		---	yes	---	---	---	---	---	---	---
Bridge Hydraulics	UD-Inlet Software (Denver)	---		---	---	---	---	---	---	---	---	yes
		---		---	---	---	---	---	---	---	---	---
Web link to Document	BRI-STARS	---		---	---	---	---	---	yes	---	---	---
	HEC-18, HEC-20 (FHWA)	yes	YES	---	---	---	---	---	yes	---	---	---
Weblink to Document		<a href="http://dot.state.nm.us/content/dam/nmdot/infrastructure/NMHydrologyManual.pdf">http://dot.state.nm.us/content/dam/nmdot/infrastructure/NMHydrologyManual.pdf</a>		<a href="http://www.azdot.gov/docs/default-source/roadway-engineering-library/2014_azdot_hydrology_manual.pdf?sfvrsn=14">http://www.azdot.gov/docs/default-source/roadway-engineering-library/2014_azdot_hydrology_manual.pdf?sfvrsn=14</a>	<a href="http://www.dot.ca.gov/hq/oppd/hdm/pdf/english/chp0810.pdf">http://www.dot.ca.gov/hq/oppd/hdm/pdf/english/chp0810.pdf</a>	<a href="https://www.codot.gov/programs/environmental-quality/documents/drainage-design-manual/drainagedesignmanual_chapter17_hydrology.pdf#view">https://www.codot.gov/programs/environmental-quality/documents/drainage-design-manual/drainagedesignmanual_chapter17_hydrology.pdf#view</a>	<a href="https://nevadadot.com/About_NDOT/NDOT_Divisions/Engineering/Hydraulics/Drainage_Manual.aspx">https://nevadadot.com/About_NDOT/NDOT_Divisions/Engineering/Hydraulics/Drainage_Manual.aspx</a>	<a href="http://online manuals.bdot.gov/txdotmanuals/hydraulicmanual_notice.htm">http://online manuals.bdot.gov/txdotmanuals/hydraulicmanual_notice.htm</a>	<a href="http://www.udot.utah.gov/main?pe100.pq.0::1:T.V.826">http://www.udot.utah.gov/main?pe100.pq.0::1:T.V.826</a>	<a href="http://www.dot.state.vy.us/files/live/sites/vydot/files/shared/Project%20Development/Road%20Design%20Manual_1/3-06%20%20Aug-2011.pdf">http://www.dot.state.vy.us/files/live/sites/vydot/files/shared/Project%20Development/Road%20Design%20Manual_1/3-06%20%20Aug-2011.pdf</a>	<a href="http://fcd.maricopa.gov/downloads/manuals/hydrology-manual.pdf">http://fcd.maricopa.gov/downloads/manuals/hydrology-manual.pdf</a>	<a href="http://www.uctd.org/downloads/pdf/critmanual/USDCM%20vols%201%202%20Dec%202011%20.pdf">http://www.uctd.org/downloads/pdf/critmanual/USDCM%20vols%201%202%20Dec%202011%20.pdf</a>
									Follows AASHTO Model Drainage Manual modified for UDOT			

## APPENDIX 10.2

### DOT DRAINAGE CRITERIA

### SUMMARY TABLES

### From other DOTs and Organizations

Table 1A	Bridge Freeboard
Table 2A	Bridge Scour
Table 3A	Existing Culverts
Table 4A	Proposed Culverts
Table 5A	Sidewalk Culverts
Table 6A	Bridge Deck Drains
Table 7A	Roadside Ditches and Inlets
Table 8A	Median Ditches and Inlets
Table 9A	Trunk Lines
Table 10A	Curb Drop Inlets, Allowable Spread, Roads

TABLE 1A BRIDGE FREEBOARD  
DRAINAGE CRITERIA SUMMARY COMPARISON

DOT NAME	Design Item	Roadway Type	Highway Level and Condition	ADT Range			Flood Name/ Return Period /No. of Lanes /Criteria			COMMENTS / OTHER CRITERIA
				All	>	<	Design Flood Return Period / Duration and Criteria	Check Flood Return Period / Duration and Criteria	Check Flood Return Period / Duration and Criteria	
							Two, Four and Six Lane Roads	Two Lane Roads	Two, Four and Six Lane Roads	
							note - 50 = 50-year return period storm, etc.			
NMDOT (a)	Bridge Freeboard	Interstate Highways	----	Y	NA	NA	50 - Minimum of 2 ft	100 - Below the low chord	100- Below the low chord	
		Principal Arterials	----	Y	NA	NA	* *	* *	* *	
		Minor Arterials Collectors and Local Roads	----	NA	Rural >= 400 ADT and All Urban		* *	* *	* *	
		Minor Arterials Collectors and Local Roads	----	NA	Rural < 400		25 - Minimum of 2 ft	50 - Below the low chord	50- Below the low chord	
CDOT (Colorado) (c)	Bridge Freeboard	Multi-lane Roads including Interstate	----	----	----	----	100	----	----	High debris streams need 4 feet
		In Urban Areas	----	----	----	----	100	----	----	Low to moderate debris streams: freeboard = $0.1 Q^{0.3} + 0.008V^2$ ; where Q is the design discharge and V is the mean velocity of the design flow through the bridge. If V > 16 fps, widen the bridge
		In Rural Areas	----	----	----	----	50	----	----	Rounded or tapered girders can allow freeboard requirements to reduce by one foot for design flow.
		Two Lane Roads in Urban Areas	----	----	----	----	100	----	----	
		Two Lane Road in Rural Areas	----	----	----	----	50 if Q50=4000 cfs	----	----	
		Two Lane Road in Rural Areas	----	----	----	----	50 if Q50=4000 cfs	----	----	
TXDOT (Texas) (d)	Bridge Freeboard	Interstate Highways	----	----	----	----	50 year unspecified freeboard to low chord	100 year unspecified freeboard to low chord	----	
		Principal Arterials	----	----	----	----	50 - 25 - 10 year unspecified freeboard to low chord	100 year unspecified freeboard to low chord	----	
		Minor Arterials	----	----	----	----	50 - 25 - 10 year unspecified freeboard to low chord	100 year unspecified freeboard to low chord	----	
		Local Roads	----	----	----	----	50 - 25 - 10 year unspecified freeboard to low chord	100 year unspecified freeboard to low chord	----	
AZDOT (e,f)	Bridge Freeboard	Interstate Highways	Drainage Class 1	----	----	----	----	----	----	3 ft for bridges on Class 1 routes
		Principal Arterials	New Construction	----	----	----	50	----	----	1 ft for bridges on routes of Classes 2-4 but not less than the design freeboard of the approach channel
			Reconstruction	----	----	----	50	----	----	
		Minor Arterials Collectors and Local Roads	Structure affected by major project	----	----	----	50	----	----	Drainage class 1 include Interstates, freeways, and some state routes (per Joseph M. Phillips, AZDOT)
		Minor Arterials Collectors and Local Roads	Drainage Class 2	----	----	----	----	----	----	The remaining classes are state routes (per Joseph M. Phillips, AZDOT)
			New Construction	----	----	----	50	----	----	A business route is where you have the option of exiting the freeway to enter a town adjacent to the freeway and
			Reconstruction	----	----	----	50	----	----	then having the ability to reenter the freeway. Typically in rural areas, You utilize local criteria due to the
			Drainage Class 3	----	----	----	----	----	----	maintenance of the route, which is typically accomplished by the town it travels through. (per Joseph M. Phillips,
			New Construction	----	----	----	25	----	----	I could not find any data online for ADT and drainage class although they the later is determined from ADT.
			Reconstruction	----	----	----	25	----	----	
			Drainage Class 4	----	----	----	----	----	----	these are the suggested minimum design storm frequencies
			New Construction	----	----	----	10	----	----	
			Reconstruction	----	----	----	10	----	----	
		Business Routes		----	----	----	Use local criteria	----	----	
CALTRANS (California) (g)	Bridge Freeboard	Does not distinguish between Roadway Types	----	----	----	----	50 year - freeboard to the lowest structural member should include the effects of bedload and debris	100 year - freeboard can be zero to lowest structural member should include effects of bedload and debris	----	Two methods to selecting the design flood frequency : 1st - By policy - using a preselected recurrence interval. 2nd - By analysis - using the recurrence interval that is most cost effective and best satisfies the specific site considerations and associated risks. NOTE - two feet of freeboard is often assumed for preliminary bridge designs, no Roadway Type was Specified
UDOT (h)	Bridge Freeboard	Interstate Highways - Rural Principal Arterial and Rural Minor Arterial	----	----	----	----	50-year - minimum clearance of 2 feet to low chord	----	----	where this is not practicable, clearance should be established by hydraulics engineer
		Urban Principal Arterial and Rural Major Collector	----	----	----	----	25-year - minimum clearance of 2 feet to low chord	----	----	
		Urban Collect - Urban Local, Rural Minor Collector, Rural Local	----	----	----	----	25-year - minimum clearance of 2 feet to low chord	----	----	

NDOT (i)	Bridge Freeboard	Interstate Highways (includes Ramps)	----	----	----	50 year - where feasible, a min freeboard of 2 feet	100 year - ensure no adverse flooding impacts are created to adjacent properties	-----	where this is not feasible, freeboard should be based on the type of stream and level of protection desired as approved
		Principal Arterials (and Other Freeways and Expressways - includes ramps)	----	----	----	50 year - **	* *	-----	
		Principal Arterials (Other Principal Arterial)	----	----	----	25 year - **	* *	-----	
		Minor Arterials Collectors and Local Roads (Minor Arterials)	----	----	----	25 year - **	* *	-----	
		Rural Major Collector	----	----	----	25 year - **	* *	-----	
		Urban or Rural Minor Collector	----	----	----	10 year - **	* *	-----	
		Frontage Roads (if not classified)	----	----	----	10 year - **	* *	-----	
WYDOT (n)	Bridge Freeboard	Interstate Highways (includes Ramps)	----	----	----	50 year - no encroachment any lane	100 year - ASSUME following - depth over crown <= 6-in., depth over gutter flow line <=18 in. adjacent buildings shall not be flooded	-----	
		Principal Arterials (and Other Freeways and Expressways - includes ramps)	----	----	----	50 year - no encroachment any lane	* *	-----	
		Principal Arterials (Other Principal Arterial)	----	----	----	50 year - no encroachment any lane	* *	-----	
		Minor Arterials Collectors and Local Roads (Minor Arterials)	----	----	----	25 year - no encroachment any lane	* *	-----	
		Minor Arterials Collectors and Local Roads (Rural Major Collector)	----	----	----	25 year - no encroachment any lane	* *	-----	
		Minor Arterials Collectors and Local Roads (Urban or Rural Minor Collector)	----	----	----	25 year - no encroachment any lane	* *	-----	
		Minor Arterials Collectors and Local Roads (Frontage Roads - if not classified)	----	----	----	25 year - no encroachment any lane	* *	-----	
CLARK COUNTY (j)	Bridge Freeboard	Streets with ROW >=80 ft	----	----	----	100	----	----	1004.1 - all bridges shall pass the 100 year design flow, water surface elevation within the bridge shall be a minimum of 2 feet below the bridge low chord. Bridge shall not back up water greater than 1 ft above the natural water surface elevation without mitigation measures, and no adjacent properties adversely affected. NOTE - This does not agree with Roadway Type Design Flood criteria
		Streets with ROW <80 ft	----	----	----	100 - an overflow section is allowed if depth X velocity <=6 and overflow depth does not exceed 1 foot	----	----	
Maricopa County (k)	Bridge Freeboard	Arterial - All weather crossings	----	----	----	50 - Max water surface at lowest adjacent road subgrade	100-Max depth = 6-inches in travel lane	----	
		Collector Streets	----	----	----	25 - Max water surface at lowest adjacent road subgrade	100-Max depth = 6-inches in travel lane	----	
		Local Streets	----	----	----	10 - Max water surface at lowest adjacent road subgrade	100-Max depth = 8-inches in travel lane	----	
Denver Urban Flood Control District (m)	Bridge Freeboard	----	----	----	----	----	----	----	Freeboard may vary from several feet to minus several feet. There are no general rules, each case must be studied separately. In large watersheds, streams and on rivers where large floating debris is likely, at least a 3-foot freeboard during a 100-year flood should be considered.
		----	----	----	----	----	----	----	
		----	----	----	----	----	----	----	
		----	----	----	----	----	----	----	

---- No Data Found

a - NMDOT - Drainage Design Criteria for New Mexico Department of Transportation Projects Fourth Revision , June 2007

b - AASHTO - American Association of State Highway and Transportation Officials - AASHTO Drainage Manual, Volume 1 - Policy 2014

c - CDOT - Colorado Department of Transportation - Drainage Design Manual, 2004

d - TXDOT - Texas Department of Transportation - Hydraulic Design Manual, Revised May 2014

e - AZDOT - Arizona Department of Transportation, Highway Drainage Design Manual, Hydraulics, Final Report January, 2007

f - AZDOT - Arizona Department of Transportation, Roadway Engineering Group - Roadway Design Guidelines, May 2012 (RDG)

g - CALTRANS - California Department of Transportation Highway Design Manual, var. dates of revision

h - UDOT - Utah Department of Transportation Manual of Instruction, Roadway Drainage, January 2004

i - NDOT - Nevada Department of Transportation Drainage Manual, NDOT Hydraulics, 2nd Edition, December 2006

j - Clark County Regional Flood Control District, Hydrologic Criteria and Drainage Design Manual, Adopted August 12, 1999

k - Drainage Design Manual for Maricopa County, Arizona - Hydraulics, 3rd Edition, August 2013

m - Urban Drainage and Flood Control District, Urban Storm Drainage Criteria Manual Vol. 1 and 2, April 2008, Vol. 3 2010

n - WYDOT - Operating Policy 18-6 - Drainage Design for Highway Systems, Feb. 15, 1979. (not available on the internet)

\*Wyoming states for drainage in their ACCESS MANUAL - Drainage in highway side ditches shall not be altered or impeded unless approved by WYDOT when drainage structures are required. Size and type of pipe and other design features shall be as directed by the Engineer having jurisdiction in the area. These costs and the costs of a drainage study, if required, shall be borne by the applicant or grantee.

**TABLE 2A BRIDGE SCOUR**  
DRAINAGE CRITERIA SUMMARY COMPARISON

DOT NAME	Design Item	Roadway Type	ADT Range			Flood Name/ Return Period /No. of Lanes /Criteria			COMMENTS / OTHER CRITERIA
			All	>	<	Design Flood Return Period / Duration and Criteria	Check Flood Return Period / Duration and Criteria	Check Flood Return Period / Duration and Criteria	
						Two, Four and Six Lane Roads	Two Lane Roads	Two, Four and Six Lane Roads	
						note - 50 = 50-year return period storm, etc.			
NMDOT ( a )	Bridge Scour	Interstate Highways	Y	NA	NA	100 - Determine scour depth as needed	500 - Determine scour depth as needed	500- Determine scour depth as needed	
		Principal Arterials	Y	NA	NA	" "	" "	" "	
		Minor Arterials Collectors and Local Roads Use Overlapping flood if less than 100 years	NA	Rural >= 400 ADT and All Urban	----	" "	" "	" "	
		Minor Arterials Collectors and Local Roads Use Overlapping flood if less than 100 years	NA	----	Rural < 400 ADT	" "	" "	" "	
CDOT (Colorado) ( c )	Bridge Scour	No Roadway type Specified	----	----	----	100 - Roadway cannot be overtopped, should be used to design the bridge abutments	500 - scour should not cause failure of the bridge structure for the 500 year flood, should be used to design the bridge foundation	500 - scour should not cause failure of the bridge structure for the 500 year flood, should be used to design the bridge foundation	
TXDOT (Texas) ( d )	Bridge Scour	Interstate Highways	----	----	----	200 year	500 year floods	----	
		Principal Arterials	----	----	----	200 year	500 year floods	----	
		Minor Arterials	----	----	----	200 year	500 year floods	----	
		Local Roads	----	----	----	200 year	500 year floods	----	
AZDOT (e,f)	Bridge Scour	Interstate Highways	----	----	----	----	500 year - Determine scour depths	----	It is ADOT's Goal that the bridge does not fail during its lifetime due to scour for flow events up-to and including the 500-year event.
		Principal Arterials	----	----	----	----	----	----	
		Minor Arterials Collectors and Local Roads	----	----	----	----	----	----	
		Minor Arterials Collectors and Local Roads	----	----	----	----	----	----	
CALTRANS (California) ( g )	Bridge Scour	Interstate Highways	----	----	----	----	----	----	HEC 23, Bridge Scour and Stream Instability Countermeasures (2009)
		Principal Arterials	----	----	----	----	----	----	
		Minor Arterials Collectors and Local Roads	----	----	----	----	----	----	
		Minor Arterials Collectors and Local Roads	----	----	----	----	----	----	
UDOT ( h )	Bridge Scour	Interstate Highways - Rural Principal Arterial and Rural Minor Arterial	----	----	----	100 year- to determine scour max scour depth	500 year - determine the max scour depth	----	
		Urban Principal Arterial and Rural Major Collector	----	----	----	101 year- to determine scour max scour depth	500 year - determine the max scour depth	----	
		Urban Collect - Urban Local, Rural Minor Collector, Rural Local	----	----	----	104 year- to determine scour max scour depth	500 year - determine the max scour depth	----	

NDOT (n)	Bridge Scour	Interstate Highways (includes Ramps)	----	----	----	100 year foundation must not fail or be damaged, the overtopping flood is used as design event if less than the 100-year and is defined as the event at which flow just begins to overtop the approach embankment(s), the bridge or both.	500- year - the bridge foundation must also be checked using estimated total scour for the lesser of the 500-year (1.7 multiplied by the 100- year unless better data is avail) or the overtopping event	----	bridge foundations must be designed by an interdisciplinary team of hydraulic, geotechnical and structural engineers to withstand the effects of est. total scour including local scour at piers and abutments, contraction scour and long-term channel agg or deg.
		Principal Arterials (and Other Freeways and Expressways - includes ramps)	----	----	----	" "	" "	----	lesser flood events should be checked if there are indications they may produce deeper scour than the 100-year or overtopping flood.
		Principal Arterials (Other Principal Arterial)	----	----	----	" "	" "	----	
		Minor Arterials Collectors and Local Roads (Minor Arterials)	----	----	----	" "	" "	----	
		Rural Major Collector	----	----	----	" "	" "	----	
		Urban or Rural Minor Collector	----	----	----	" "	" "	----	
		Frontage Roads (if not classified)	----	----	----	" "	" "	----	
WYDOT (a)	Bridge Scour	Interstate Highways	----	----	----	----	----	----	
		Principal Arterials	----	----	----	----	----	----	
		Minor Arterials Collectors and Local Roads	----	----	----	----	----	----	no info
		Use Overtopping flood if less than 100 years	----	----	----	----	----	----	
		Minor Arterials Collectors and Local Roads	----	----	----	----	----	----	
CLARK COUNTY (j)	Bridge Scour	-----	----	----	----	-----	-----	-----	no info
		-----	----	----	----	-----	-----	-----	
		-----	----	----	----	-----	-----	-----	
Maricopa County (k)	Bridge Scour	-----	----	----	----	-----	-----	-----	no guidance other than to use a generated flow hydrograph from historical flow records if available, when no record use a synthetic long term hydrograph.
		-----	----	----	----	-----	-----	-----	
		-----	----	----	----	-----	-----	-----	
		-----	----	----	----	-----	-----	-----	
Denver Urban Flood Control District (m)	Bridge Scour	-----	----	----	----	-----	-----	-----	no info
		-----	----	----	----	-----	-----	-----	
		-----	----	----	----	-----	-----	-----	
		-----	----	----	----	-----	-----	-----	

---- No Data Found

a - NMDOT - Drainage Design Criteria for New Mexico Department of Transportation Projects Fourth Revision , June 2007

b - AASHTO - American Association of State Highway and Transportation Officials - AASHTO Drainage Manual, Volume 1 - Policy 2014

c - CDOT - Colorado Department of Transportation - Drainage Design Manual, 2004

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e - AZDOT - Arizona Department of Transportation, Highway Drainage Design Manual, Hydraulics, Final Report. January, 2007

f - AZDOT - Arizona Department of Transportation, Roadway Engineering Group - Roadway Design Guidelines, May 2012 (RDG)

g - CALTRANS - California Department of Transportation Highway Design Manual, var. dates of revision

h - UDOT - Utah Department of Transportation Manual of Instruction, Roadway Drainage, January 2004

i - NDOT - Nevada Department of Transportation Drainage Manual, NDOT Hydraulics, 2nd Edition, December 2006

j - Clark County Regional Flood Control District, Hydrologic Criteria and Drainage Design Manual, Adopted August 12, 1999

k - Drainage Design Manual for Maricopa County, Arizona - Hydraulics, 3rd Edition, August 2013

m - Urban Drainage and Flood Control District, Urban Storm Drainage Criteria Manual Vol. 1 and 2. April 2008, Vol. 3 2010

n - WYDOT - Operating Policy 18-6 - Drainage Design for Highway Systems, Feb. 15, 1979. (not available on the internet)

\*Wyoming states for drainage in their ACCESS MANUAL - Drainage in highway side ditches shall not be altered or impeded unless approved by WYDOT when drainage structures are required. Size and type of pipe and other design features shall be as directed by the Engineer having jurisdiction in the area. These costs and the costs of a drainage study, if required, shall be borne by the applicant or grantee.



TABLE 3A EXISTING CULVERTS  
DRAINAGE CRITERIA SUMMARY COMPARISON

DOT NAME	Design Item	Roadway Type	ADT Range			Flood Name/ Return Period /No. of Lanes /Criteria			COMMENTS / OTHER CRITERIA
			All	>	<	<u>Design</u> Flood Return Period / Duration and Criteria	<u>Check</u> Flood Return Period / Duration and Criteria	<u>Check</u> Flood Return Period / Duration and Criteria	
						Two, Four and Six Lane Roads	Two Lane Roads	Two, Four and Six Lane Roads	
						note - 50 = 50-year return period storm, etc.			
NMDOT ( a )	Existing Culverts	Interstate Highways	Y	NA	NA	50 - Limit headwater to edge of driving lane	100 - Limit headwater to one <u>half</u> of a driving lane	100- Limit headwater to one driving lane	
		Principal Arterials	Y	NA	NA	" "	" "	" "	
		Minor Arterials Collectors and Local Roads	NA	Rural >= 400 ADT and All Urban	----	" "	" "	" "	
		Minor Arterials Collectors and Local Roads	NA	----	Rural < 400 ADT	25 - Limit headwater to edge of driving lane	50 - Limit headwater to one <u>half</u> of a driving lane	50 - Limit headwater to one driving lane	
CDOT (Colorado) ( c )	Existing Culverts (no distinction between existing and new culverts)	Multi-lane Roads including Interstate	----	----	----	100	100 - does not exceed 1-ft increase over the existing 100-yr. FEMA floodplain or in the vicinity of an insurable building	----	detour culverts-monthly discharges for 2 to 5 years
		In Urban Areas	----	----	----	100	" "	----	AH - max HWD = 2 for < 36"
		In Rural Areas	----	----	----	50	" "	----	AH - max HWD = 1.7 for 36" - 60"
		Two Lane Roads in Urban Areas	----	----	----	100	" "	----	AH - max HWD = 1.5 for > 60" & <84"
		Two Lane Road in Rural Areas	----	----	----	50 if Q50>=4000 cfs	" "	----	AH - max HWD = 1.2 for 84" to <120"
		Two Lane Road in Rural Areas	----	----	----	50 if Q50<4000 cfs	" "	----	AH - max HWD = 1.0 for 120" and >
TXDOT (Texas) ( d )	Existing Culverts ( see Comment)	Interstate Highways	----	----	----	----	----	----	
		Principal Arterials	----	----	----	----	----	----	No distinction between existing and new culverts
		Minor Arterials	----	----	----	----	----	----	See New Culverts Table
		Local Roads	----	----	----	----	----	----	
AZDOT (e,f)	Existing Culverts (see Comment)	Interstate Highways	----	----	----	----	----	----	
		Principal Arterials	----	----	----	----	----	----	
		Minor Arterials Collectors and Local Roads	----	----	----	----	----	----	no discussion of existing culverts
		Minor Arterials Collectors and Local Roads	----	----	----	----	----	----	
CALTRANS (California) ( g )	Existing Culverts (see comment)	Does not distinguish between Roadway Types	----	----	----	----	----	----	no discussion of existing culverts
UDOT ( h )	Existing Culverts (see comment)	Interstate Highways - Rural Principal Arterial	----	----	----	----	----	----	
		Urban Principal Arterial and Rural Major Collector	----	----	----	----	----	----	no discussion of existing culverts
		Urban Collect - Urban Local, Rural Minor Collector, Rural Local	----	----	----	----	----	----	

NDOT (i)	Existing Culverts - No discussion of existing culverts	Interstate Highways (includes Ramps)	----	----	----	50 year,	100-year	----	maximum allowable headwater depth is edge of pavement for the design storm for all roadways
		Principal Arterials (and Other Freeways and Expressways - includes ramps)	----	----	----	50 year	" "	----	Existing and Future conditions must be evaluated for the 100-year event. Mitigation must be provided if 100-year flow within or out of ROW is unreasonably aggravated.
		Principal Arterials (Other Principal Arterial)	----	----	----	25 year	" "	----	
		Minor Arterials Collectors and Local Roads (Minor Arterials)	----	----	----	25 year	" "	----	
		Rural Major Collector	----	----	----	25 year	" "	----	
		Urban or Rural Minor Collector	----	----	----	10 year	" "	----	
		Frontage Roads (if not classified)	----	----	----	10 year	" "	----	
WYDOT (n)	Existing Culverts (no distinction for existing culverts)	Interstate Highways (includes Ramps)	----	----	----	small and large culverts and bridges 50 year - no max allowable spread into adjacent travel lane	100 year - depth over crown ,<= 6-in., depth over gutter flow line <=18 in.	----	
		Principal Arterials (and Other Freeways and Expressways - includes ramps)	----	----	----	small culverts 25-year and large culverts and bridges 50 year - no max allowable spread into adjacent travel lane	100 year - depth over crown ,<= 6-in., depth over gutter flow line <=18 in.	----	
		ALL Depressed Highways and Underpasses	----	----	----	small, large culverts and bridges 50 year - no maximum allowable spread on any traffic lanes	100 year - depth over crown ,<= 6-in., depth over gutter flow line <=18 in. adjacent buildings shall not be flooded	----	
		Urban and Rural Minor Arterials Collectors and Local Roads (Minor Arterials)	----	>750	----	small and large culverts and bridges 25 year - no curb overtopping, must leave one lane open, large culverts 25 year	100 year - depth over crown ,<= 6-in., depth over gutter flow line <=18 in., adjacent buildings shall not be flooded	----	
		Urban and Rural Minor Arterials Collectors and Local Roads (Minor Arterials)	----		<750	small culverts 10 year - large culverts and bridges 25-year, no curb overtopping, must leave one lane open	100 year - depth over crown ,<= 6-in., depth over gutter flow line <=18 in., adjacent buildings shall not be flooded	----	
		Urban and Rural Collector	----	>750	----	small and large culverts and bridges 25 year - no curb overtopping, must leave one lane open	100 year - protect adjacent buildings from flooding, depth over gutter flow line <=18 in., adjacent buildings shall not be flooded	----	
		Urban and Rural Collector	----		<750	small culverts 10 year, large culverts and bridges 25 year - no curb overtopping, must leave one lane open	100 year - protect adjacent buildings from flooding, depth over gutter flow line <=18 in., adjacent buildings shall not be flooded	----	
CLARK COUNTY (j)	Existing Culverts (no discussion of Existing Culverts)	Streets with ROW >=80 ft	----	----	----	100	----	----	1004.1 - all bridges shall pass the 100 year design flow, water surface elevation within the bridge shall be a minimum of 2 feet below the bridge low chord. Bridge shall not back up water greater than 1 ft above the natural water surface elevation without mitigation measures, and no adjacent properties adversely affected. NOTE - This does not agree with Roadway Type Design Flood criteria
		Streets with ROW <80 ft	----	----	----	100-, an overflow section is allowed if depth X velocity <=6 and overflow depth does not exceed 1 foot	----	----	
Maricopa County (k)	Existing Culverts (see comment)	---	----	----	----	----	----	----	no discussion of existing culverts
		---	----	----	----	----	----	----	
		---	----	----	----	----	----	----	
		---	----	----	----	----	----	----	

Denver Urban Flood Control District (m)	Existing Culverts (no distinction for existing culverts)	Local -	----	----	----	10-5- or 2- (depends on local municipality). No curb overtopping , flow may spread to crown.	---	----	No specific headwater criteria is given except - The headwater elevation and design discharge should be consistent with the overtopping and freeboard criteria listed here.
		Collector	----	----	----	10-5- or 2- (depends on local municipality). No curb overtopping , flow spread must leave one lane free of water.	---	----	
		Arterial	----	----	----	10-5- or 2- (depends on local municipality). No curb overtopping , flow spread must leave one lane free of water in each direction, but should not flood more than two lanes in each direction.	---	----	
		Freeway	----	----	----	10-5- or 2- (depends on local municipality). No encroachment on any traffic lanes.	---	----	
		Local and Collector	----	----	----	---	100- Residential dwelling should be no less than 12-in. above the 100-year flood at the ground line or lowest water entry of a building. The depth of water over the gutter flow line should not exceed 18-in. for local and 12-in. for collector streets.	----	
		Arterial and Freeway	----	----	----	---	100- Residential dwelling should be no less than 12-in. above the 100-year flood at the ground line or lowest water entry of a building. The depth of water should not exceed the street crown to allow operation of emergency vehicles. The depth of water over the gutter flow line should not exceed 12-in.	----	

---- No Data Found

a - NMDOT - Drainage Design Criteria for New Mexico Department of Transportation Projects Fourth Revision , June 2007

b - AASHTO - American Association of State Highway and Transportation Officials - AASHTO Drainage Manual, Volume 1 - Policy 2014

c - CDOT - Colorado Department of Transportation - Drainage Design Manual, 2004

d - TXDOT - Texas Department of Transportation - Hydraulic Design Manual, Revised May 2014

e - AZDOT - Arizona Department of Transportation, Highway Drainage Design Manual, Hydraulics, Final Report. January, 2007

f - AZDOT - Arizona Department of Transportation, Roadway Engineering Group - Roadway Design Guidelines, May 2012 (RDG)

g - CALTRANS - California Department of Transportation Highway Design Manual, var. dates of revision

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\*Wyoming states for drainage in their ACCESS MANUAL - Drainage in highway side ditches shall not be altered or impeded unless approved by WYDOT when drainage structures are required. Size and type of pipe and other design features shall be as directed by the Engineer having jurisdiction in the area. These costs and the costs of a drainage study, if required, shall be borne by the applicant or grantee.

TABLE 4A NEW CULVERTS  
DRAINAGE CRITERIA SUMMARY COMPARISON

DOT NAME	Design Item	Roadway Type	ADT Range			Highway Level and Condition	Flood Name/ Return Period /No. of Lanes /Criteria			COMMENTS / OTHER CRITERIA
			All	>	<		Design Flood Return Period / Duration and Criteria	Check Flood Return Period / Duration and Criteria	Check Flood Return Period / Duration and Criteria	
							Two, Four and Six Lane Roads	Two Lane Roads	Two, Four and Six Lane Roads	
							note - 50 = 50-year return period storm, etc.			
NMDOT ( a )	New Culverts	Interstate Highways	Y	----	----	----	50 -Ratio of headwater depth to culvert rise shall not exceed 1.5 and limit headwater to edge of shoulder	100 - Limit headwater to one <u>half</u> of a driving lane	100- Limit headwater to one driving lane	
		Principal Arterials	Y	----	----	----	" "	" "	" "	
		Minor Arterials Collectors and Local Roads	----	Rural >= 400 ADT and All	----	----	" "	" "	" "	
		Minor Arterials Collectors and Local Roads	----	----	Rural < 400 ADT	----	25 - Ratio of headwater depth to culvert rise shall not exceed 1.5 and limit headwater to edge of shoulder	50 -Limit headwater to one half of a driving lane	50 - limit headwater to one driving lane	
CDOT (Colorado) ( c )	New Culverts	Multi-lane Roads including Interstate	----	----	----	----	100	100 - does not exceed 1-ft increase over the existing 100-yr. FEMA floodplain or in the vicinity of an insurable building	----	debour culverts-monthly discharges for 2 to 5 years
		In Urban Areas	----	----	----	----	100	" "	----	AH - max HWD = 2 for < 36"
		In Rural Areas	----	----	----	----	50	" "	----	AH - max HWD = 1.7 for 36" - 60"
		Two Lane Roads in Urban Areas	----	----	----	----	100	" "	----	AH - max HWD = 1.5 for > 60" & <84"
		Two Lane Road in Rural Areas	----	----	----	----	50 if Q50>=4000 cfs	" "	----	AH - max HWD = 1.2 for 84" to <120"
		Two Lane Road in Rural Areas	----	----	----	----	50 if Q50<4000 cfs	" "	----	AH - max HWD = 1.0 for 120" and >
TXDOT (Texas) ( d )	New Culverts	Interstate Highways	----	----	----	----	50 year - Allowable headwater, roadway must be passable	---	---	
		Principal Arterials	----	----	----	----	50- 25-, 10- year - Allowable headwater, roadway must be passable	---	---	
			----	----	----	----		---	---	
		Minor Arterials	----	----	----	----	25- , 10-, 5- year, Allowable headwater, roadway must be passable	---	---	
		Local Roads	----	----	----	----	10- 5-, 2- year, Allowable headwater, roadway must be passable	---	---	

AZDOT (Arizona) (e, f)	New Culverts	Interstate Highways	----	----	----	Drainage Class 1				Flood Frequency is based on roadway classification found in Chapter 600 of the Roadway Design Guidelines), the existence of FEMA mapped floodplains and the level of risk associated with the 100 year event to adjacent property
		Principal Arterials	----	----	----	New Construction	50 - Allowable headwater 3-in. below the pavement and HWD should not exceed 1.5	100 - Allowable headwater should not significantly increase the flood damage potential on areas outside the DOT right-of-way		
			----	----	----	Reconstruction	50 - " "	100 - " "		
		Minor Arterials Collectors and Local Roads	----	----	----	Structure affected by major project	50 - " "	100 - " "		
		Minor Arterials Collectors and Local Roads	----	----	----	Drainage Class 2				
			----	----	----	New Construction	50 - " "	100 - " "		
			----	----	----	Reconstruction	50 - " "	100 - " "		
			----	----	----	Drainage Class 3				
			----	----	----	New Construction	25- " "	100 - " "		
			----	----	----	Reconstruction	25- " "	100 - " "		
			----	----	----	Drainage Class 4				
			----	----	----	New Construction	10- " "	100 - " "		
			----	----	----	Reconstruction	10- " "	100 - " "		
			----	----	----	Business Routes	Use local criteria	----		
CALTRANS (California) (g)	New Culverts	Does not distinguish between Roadway Types	----	----	----	----	10 year - Allowable Headwater - maximum headwater elevation at top of the culvert -	100 year - Allowable Headwater - should not rise above an elevation that would cause objectionable backwater depths or outlet velocities	----	
UDOT (h)	New Culverts	Interstate Highways - Rural Principal Arterial and Rural Minor Arterial	----	----	----	----	50-Design flood - allowable headwater below the edge of shoulder	a maximum of 0.5 ft. above the FEMA 100-yr flood elevation, a maximum of 1 ft. rise over the 100-yr. flood in unmapped floodplains	----	
		Urban Principal Arterial and Rural Major Collector	----	----	----	----	" "	" "	----	
		Urban Collect - Urban Local, Rural Minor Collector, Rural Local	----	----	----	----	" "	" "	----	
NDOT (i)	New Culverts	Interstate Highways (includes Ramps)	----	----	----	----	50 year	100-year	----	maximum allowable headwater depth is edge of pavement for the design storm for all roadways
		Principal Arterials (and Other Freeways and Expressways - includes ramps)	----	----	----	----	50 year	" "	----	Existing and Future conditions must be evaluated for the 100-year event. Mitigation must be provided if 100-year flow within or out of ROW is unreasonably aggravated.
		Principal Arterials (Other Principal Arterial)	----	----	----	----	25 year	" "	----	
		Minor Arterials Collectors and Local Roads	----	----	----	----	25 year	" "	----	
		Rural Major Collector	----	----	----	----	25 year	" "	----	
		Urban or Rural Minor Collector	----	----	----	----	10 year	" "	----	
		Frontage Roads (if not classified)	----	----	----	----	10 year	" "	----	

WYDOT (n)	New Culverts	Interstate Highways (includes Ramps)	----	----	----	----	small and large culverts and bridges 50 year - no max allowable spread into adjacent travel lane	100 year - depth over crown ,<= 6-in., depth over gutter flow line <=18 in.	----	No distinction was made in WDOT criteria between new and old culverts, therefore assume same criteria as for existing culvert table
		Principal Arterials (and Other Freeways and Expressways - includes ramps)	----	----	----	----	small culverts 25-year and large culverts and bridges 50 year - no max allowable spread into adjacent travel lane	100 year - depth over crown ,<= 6-in., depth over gutter flow line <=18 in.	----	" "
		ALL Depressed Highways and Underpasses	----	----	----	----	small, large culverts and bridges 50 year - no maximum allowable spread on any traffic lanes	100 year - depth over crown ,<= 6-in., depth over gutter flow line <=18 in. adjacent buildings shall not be flooded	----	" "
		Urban and Rural Minor Arterials Collectors and Local Roads (Minor Arterials)	----	>750	----	----	small and large culverts and bridges 25 year - no curb overtopping, must leave one lane open, large culverts 25 year	100 year - depth over crown ,<= 6-in., depth over gutter flow line <=18 in., adjacent buildings shall not be flooded	----	" "
		Urban and Rural Minor Arterials Collectors and Local Roads (Minor Arterials)	----		<750	----	small culverts 10 year - large culverts and bridges 25-year, no curb overtopping, must leave one lane open	100 year - depth over crown ,<= 6-in., depth over gutter flow line <=18 in., adjacent buildings shall not be flooded	----	" "
		Urban and Rural Collector	----	>750	----	----	small and large culverts and bridges 25 year - no curb overtopping, must leave one lane open	100 year - protect adjacent buildings from flooding, depth over gutter flow line <=18 in., adjacent buildings shall not be flooded	----	" "
		Urban and Rural Collector	----		<750	----	small culverts 10 year, large culverts and bridges 25 year - no curb overtopping, must leave one lane open	100 year - protect adjacent buildings from flooding, depth over gutter flow line <=18 in., adjacent buildings shall not be flooded	----	" "
CLARK COUNTY (j)	New Culverts	Streets with ROW >=80 ft	----	----	----	----	100	----	----	No clear info. Assume the same as for bridges : 1004.1 - all bridges shall pass the 100 year design flow, water surface elevation within the bridge shall be a minimum of 2 feet below the bridge low chord. Bridge shall not back up water greater than 1 ft above the natural water surface elevation without mitigation measures, and no adjacent properties adversely affected. NOTE - This does not agree with Roadway Type Design Flood criteria
		Streets with ROW <80 ft	----	----	----	----	100-, an overflow section is allowed if depth X velocity <=6 and overflow depth does not exceed 1 foot	----	----	

Maricopa County (k)	New Culverts	Arterial - All weather crossings	----	----	----	---	50 - Max water surface at lowest adjacent road subgrade	100-Max depth = 6-inches in travel lane	----	Regardless of the size of the culvert, street crossings shall be designed to convey the 100 year storm runoff under and/or over the road. Dip sections are not recommended.
		Collector Streets	----	----	----	---	25 - Max water surface at lowest adjacent road subgrade	100-Max depth = 6-inches in travel lane	----	
		Local Streets	----	----	----	---	10 - Max water surface at lowest adjacent road subgrade	100-Max depth = 8-inches in travel lane	----	
Denver Urban Flood Control District (m)	New Culverts	Local -	----	----	----	---	10-5- or 2- (depends on local municipality). No curb overtopping, flow may spread to crown.	---	----	No specific headwater criteria is given except - The headwater elevation and design discharge should be consistent with the overtopping and freeboard criteria listed here.
		Collector	----	----	----	---	10-5- or 2- (depends on local municipality). No curb overtopping, flow spread must leave one lane free of water.	---	----	
		Arterial	----	----	----	---	10-5- or 2- (depends on local municipality). No curb overtopping, flow spread must leave one lane free of water in each direction, but should not flood more than two lanes in each direction.	---	----	
		Freeway	----	----	----	---	10-5- or 2- (depends on local municipality). No encroachment on any traffic lanes.	---	----	
		Local and Collector	----	----	----	---	---	100- Residential dwelling should be no less than 12-in. above the 100-year flood at the ground line or lowest water entry of a building. The depth of water over the gutter flow line should not exceed 18-in. for local and 12-in. for collector streets.	----	
		Arterial and Freeway	----	----	----	---	---	100- Residential dwelling should be no less than 12-in. above the 100-year flood at the ground line or lowest water entry of a building. The depth of water should not exceed the street crown to allow operation of emergency vehicles. The depth of water over the gutter flow line should not exceed 12-in.	----	

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\*Wyoming states for drainage in their ACCESS MANUAL - Drainage in highway side ditches shall not be altered or impeded unless approved by WYDOT when drainage structures are required. Size and type of pipe and other design features shall be as directed by the Engineer having jurisdiction in the area. These costs and the costs of a drainage study, if required, shall be borne by the applicant or grantee.

TABLE 5A - SIDEWALK CULVERTS  
DRAINAGE CRITERIA SUMMARY COMPARISON

DOT NAME	Design Item	Roadway Type	ADT Range			Flood Name/ Return Period /No. of Lanes /Criteria			COMMENTS / OTHER CRITERIA
			All	>	<	<u>Design</u> Flood Return Period / Duration and Criteria	<u>Check</u> Flood Return Period / Duration and Criteria	<u>Check</u> Flood Return Period / Duration and Criteria	
						Two, Four and Six Lane Roads	Two Lane Roads	Two, Four and Six Lane Roads	
						note - 50 = 50-year return period storm, etc.			
NMDOT (a)	Sidewalk Culverts	Interstate Highways	Y	----	----	50 - 24 - Limit headwater depth to top of sidewalk	100 - 24 - Overtopping the sidewalk is allowed	100- 24 - Overtopping the sidewalk is allowed	
		Principal Arterials	Y	----	----	" "	" "	" "	
		Minor Arterials Collectors and Local Roads	----	Rural >= 400 ADT and All Urban	----	" "	" "	" "	
		Minor Arterials Collectors and Local Roads	----	----	Rural < 400 ADT	25 - 24 - Limit headwater depth to top of sidewalk	50 - 24 - Overtopping the sidewalk is allowed	50 - 24 - Overtopping the sidewalk is allowed	
CDOT (Colorado) (c)	Sidewalk Culverts	No Roadway given	----	----	----	----	----	----	no info
			----	----	----	----	----	----	
			----	----	----	----	----	----	
			----	----	----	----	----	----	
TXDOT (Texas) (d)	Sidewalk Culverts	Interstate Highways	----	----	----	----	----	----	no info
		Principal Arterials	----	----	----	----	----	----	
		Minor Arterials Collectors and Local Roads	----	----	----	----	----	----	
		Minor Arterials Collectors and Local Roads	----	----	----	----	----	----	
AZDOT (Arizona) (e, f)	Sidewalk Culverts	Interstate Highways	----	----	----	----	----	----	no info
		Principal Arterials	----	----	----	----	----	----	
		Minor Arterials Collectors and Local Roads	----	----	----	----	----	----	
		Minor Arterials Collectors and Local Roads	----	----	----	----	----	----	
CALTRANS (California) (g)	Sidewalk Culverts	Does not distinguish between Roadway Types	----	----	----	----	----	----	no info
			----	----	----	----	----	----	
UDOT (h)	Sidewalk Culverts	Interstate Highways - Rural Principal Arterial	----	----	----	----	----	----	
		Urban Principal Arterial and Rural Major Collector	----	----	----	----	----	----	no info
		Urban Collect - Urban Local, Rural Minor Collector, Rural Local	----	----	----	----	----	----	
NDOT (i)	Sidewalk Culverts	No Roadway Classification	----	----	----	----	----	----	no info
			----	----	----	----	----	----	
			----	----	----	----	----	----	



WYDOT (n)	Sidewalk Culverts	Interstate Highways (includes Ramps)	----	----	----	----	----	----	no info
		Principal Arterials (and Other Freeways and Expressways - includes ramps)	----	----	----	----	----	----	
		ALL Depressed Highways and Underpasses	----	----	----	----	----	----	
		Urban and Rural Minor Arterials Collectors and Local Roads (Minor Arterials)	----	----	----	----	----	----	
		Urban and Rural Minor Arterials Collectors and Local Roads (Minor Arterials)	----	----	----	----	----	----	
		Urban and Rural Collector	----	----	----	----	----	----	
		Urban and Rural Collector	----	----	----	----	----	----	
CLARK COUNTY (j)	Sidewalk Culverts	----	----	----	----	----	----	----	no info
		----	----	----	----	----	----	----	
		----	----	----	----	----	----	----	
Maricopa County (k)	Sidewalk Culverts	----	----	----	----	----	----	----	no info
		----	----	----	----	----	----	----	
		----	----	----	----	----	----	----	
		----	----	----	----	----	----	----	
Denver Urban Flood Control District (m)	Sidewalk Culverts	----	----	----	----	----	----	----	no info
		----	----	----	----	----	----	----	
		----	----	----	----	----	----	----	
		----	----	----	----	----	----	----	

---- No Data Found

a - NMDOT - Drainage Design Criteria for New Mexico Department of Transportation Projects Fourth Revision , June 2007

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## RAINAGE CRITERIA SUMMARY COMPARISON

DOT NAME	Design Item	Roadway Type	ADT Range			Highway Design Speed		Flood Name/ Return Period /No. of Lanes /Criteria			COMMENTS / OTHER CRITERIA	
			All	>	<	<= 45 mph	> 45 mph	<u>Design</u> Flood Return Period / Duration and Criteria	<u>Check</u> Flood Return Period / Duration and Criteria	<u>Check</u> Flood Return Period / Duration and Criteria		
								Two, Four and Six Lane Roads	Two Lane Roads	Two, Four and Six Lane Roads		
								note - 50 = 50-year return period storm, etc.				
NMDOT ( a )	Bridge Deck Drains	Interstate Highways	Y	NA	NA	----	----	50 - Limit water spread to edge of driving lane	100 - Limit water spread to one <u>half</u> of a driving lane	100- Limit water spread to one driving lane		
		Principal Arterials	Y	NA	NA	----	----	" "	" "	" "		
		Minor Arterials Collectors and Local Roads	NA	Rural >= 400 ADT and All Urban	NA	----	----	" "	" "	" "		
		Minor Arterials Collectors and Local Roads	NA	NA	Rural < 400 ADT	----	----	25 - Limit water spread to edge of driving lane	50 - Limit water spread to one <u>half</u> of a driving lane	50 - Limit water spread to one driving lane		
CDOT (Colorado) ( c )	Bridge Deck Drains		----	----	----	----	----	----	----	----	p. 10-28	
		No Roadway Classification given	----	----	----	----	----	----	----	----	No data found for hyd criteria	
			----	----	----	----	----	----	----	----		
TXDOT (Texas) ( d )	Bridge Deck Drains	Interstate Highways	----	----	----	----	----	----	----	----	mentions that there should be drainage but there is no design storm or design criteria given	
		Principal Arterials	----	----	----	----	----	----	----	----		
			----	----	----	----	----	----	----	----		
		Minor Arterials Collectors and Local Roads	----	----	----	----	----	----	----	----		
		Minor Arterials Collectors and Local Roads	----	----	----	----	----	----	----	----		
AZDOT (Arizona) ( e , f )	Bridge Deck Drains	Interstate Highways	----	----	----	----	----	----	----	----	the Rational equation and the spread equation can be combined to determine the length of the deck possible without drainage structures and without exceeding the allowable spread.	
		Principal Arterials	----	----	----	----	----	----	----	----	should be spaced to satisfy the design spread criteria given in section 603.2 in the RDG	
			----	----	----	----	----	----	----	----	Drainage of decks is similar to other curbed roadway sections. Often less efficient, bc cross slopes are flatter, parapets collect large amts of debris and small drainage inlets or scoupers have a higher potential for clogging by debris. Zero gradients and sag vertical curves and super elevation transitions with flat pavement sections should be avoided on bridges. Min desirable longitudinal slope for bridge deck drainage should be 0.5%	
		Minor Arterials Collectors and Local Roads	----	----	----	----	----	----	----	----		
		Minor Arterials Collectors and Local Roads	----	----	----	----	----	----	----	----		
CALTRANS (California) ( g )	Bridge Deck Drains	Interstate Highways	----	----	----	----	----	----	----	----	no info	
		Principal Arterials	----	----	----	----	----	----	----	----		
			----	----	----	----	----	----	----	----		
		Minor Arterials Collectors and Local Roads	----	----	----	----	----	----	----	----		
		Minor Arterials Collectors and Local Roads	----	----	----	----	----	----	----	----		

UDOT (h)	Bridge Deck Drains	Interstate Highways - Rural Principal Arterial and Rural Minor Arterial	----	----	----	----	----	----	----	----	where it is necessary to intercept deck drainage at intermediate points along the bridge, the design of the interceptors shall conform to the procedures in Chapter 13, HEC 21 (6) and HEC 22 (9), min. bridge deck longitudinal slope=0.5%
		Urban Principal Arterial and Rural Major Collector	----	----	----	----	----	----	----	----	
		Urban Collect - Urban Local, Rural Minor Collector, Rural Local	----	----	----	----	----	----	----	----	
		-----	-----	-----	-----	-----	-----	-----	-----	-----	
		-----	-----	-----	-----	-----	-----	-----	-----	-----	
NDOT (i)	Bridge Deck Drains	Interstate Highways	----	----	----	----	----	----	----	----	13.7.7 - many bridges will not require any drainage structures at all, to determine the length of deck permitted without drainage and without exceeding allowable spread, use eqn which is based on a uniform cross slope
		Principal Arterials	----	----	----	----	----	----	----	----	
		Minor Arterials Collectors and Local Roads	----	----	----	----	----	----	----	----	
		-----	-----	-----	-----	-----	-----	-----	-----	-----	
WYDOT (n)	Bridge Deck Drains	-----	----	----	----	----	----	----	----	----	no info
CLARK COUNTY (j)	Bridge Deck Drains	-----	----	----	----	----	----	----	----	----	no info
		-----	----	----	----	----	----	----	----	----	
		-----	----	----	----	----	----	----	----	----	
		-----	----	----	----	----	----	----	----	----	
Maricopa County (k)	Bridge Deck Drains	-----	----	----	----	----	----	----	----	----	no info
		-----	----	----	----	----	----	----	----	----	
		-----	----	----	----	----	----	----	----	----	
		-----	----	----	----	----	----	----	----	----	
Denver Urban Flood Control District (m)	Bridge Deck Drains	-----	----	----	----	----	----	----	----	----	no info
		-----	----	----	----	----	----	----	----	----	
		-----	----	----	----	----	----	----	----	----	
		-----	----	----	----	----	----	----	----	----	

---- No Data Found

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DOT NAME	Design Item	Roadway Type	ADT Range			Speed and Sag			Flood Name/ Return Period /No. of Lanes /Criteria			COMMENTS / OTHER CRITERIA
			All	>	<	<45 mph	>45 mph	sag point	<u>Design Flood Return Period / Duration and Criteria</u>	<u>Check Flood Return Period / Duration and Criteria</u>	<u>Check Flood Return Period / Duration and Criteria</u>	
									Two, Four and Six Lane Roads	Two Lane Roads	Two, Four and Six Lane Roads	
									note - 50 = 50-year return period storm, etc.			
<b>NMDOT ( a )</b>	<b>Roadside Ditches and Inlets</b>	Interstate Highways	Y	NA	NA				50 - Limit water depth to edge of shoulder	100 - Limit water depth to one <u>half</u> of a driving lane	100- Limit water depth to one driving lane	
		Principal Arterials	Y	NA	NA				" "	" "	" "	
		Minor Arterials Collectors and Local Roads	NA	Rural >= 400 ADT and All Urban	NA				10 - Limit water depth to edge of shoulder	50 - Limit water depth to one <u>half</u> of a driving lane	50 - Limit water depth to one driving lane	
		Minor Arterials Collectors and Local Roads	NA	NA	Rural < 400 ADT				10 - Limit water depth to edge of shoulder	25 - Limit water depth to one <u>half</u> of a driving lane	25- Limit water depth to one driving lane	
<b>CDOT (Colorado) ( c )</b>	<b>Roadside Ditches and Inlets</b>	Multi-lane Roads including Interstate	----	----	----	----	----	----	100	100 - does not exceed 1-ft increase over the existing 100-yr. FEMA floodplain or in the vicinity of an insurable building	----	same as for cross-drainage culverts and bridges - NOTE Channels shall have min. 1 ft freeboard
		In Urban Areas	----	----	----	----	----	----	100	" "	----	
		In Rural Areas	----	----	----	----	----	----	50	" "	----	
		Two Lane Roads in Urban Areas	----	----	----	----	----	----	100	" "	----	
		Two Lane Road in Rural Areas	----	----	----	----	----	----	50 if Q50>=4000 cfs	" "	----	
		Two Lane Road in Rural Areas	----	----	----	----	----	----	50 if Q50<4000 cfs	" "	----	
<b>TXDOT (Texas) ( d )</b>	<b>Roadside Ditches and Inlets</b>	Interstate Highways	----	----	----	----	----	----	10 year, roadway must be passable	----	----	
		Principal Arterials	----	----	----	----	----	----	2-, 5-, 10- year with recommendation on 5 year, roadway must be passable	----	----	
			----	----	----	----	----	----	-----	----	----	
		Minor Arterials	----	----	----	----	----	----	2-, 5-, 10- year with recommendation on 5 year, roadway must be passable	----	----	
		Local Roads	----	----	----	----	----	----	-----	----	----	

AZDOT (Arizona) ( e , f )	Roadside Ditches and Inlets	Interstate Highways	----	----	----	----	----	----	100 year -	----	----	where overtopping would permit storm water to breakout of ADOT ROW, the min freeboard shall be 1 foot
		Principal Arterials	----	----	----	----	----	----	----	----	----	where the failure of roadside channels would increase the flood hazard of adjacent properties the channel shall safely convey the 100 year event within the ADOT ROW. The min design discharge for permanent roadside ditch linings shall be a 10-year frequency while temporary linings may be a 2-year frequency flow
			----	----	----	----	----	----	----	----	----	it is preferable to slope median areas and inside shoulders to a center swale, to prevent drainage from the median area from running across the pavement. This is particularly important for high-speed facilities and for facilities with more than two lanes of traffic in each direction.
		Minor Arterials Collectors and Local Roads	----	----	----	----	----	----	----	----	----	
		Minor Arterials Collectors and Local Roads	----	----	----	----	----	----	----	----	----	
CALTRANS (California) ( g )	Roadside Ditches and Inlets (assume the same as spread for roadway drainage (Caltrans Table 831.3)	<b>FREEWAYS -</b>	----	----	----	----	----	----	----	----	----	
		Though traffic lanes, branch connections, and other major ramp connections	----	----	----	----	----	----	25 - water spread shoulder or parking lane	----	----	
		Minor Ramps	----	----	----	----	----	----	10 - water spread shoulder or parking lane	----	----	
		Frontage Road	----	----	----	----	----	----	10 - water spread as local standard	----	----	
		<b>CONVENTIONAL HIGHWAYS</b>	----	----	----	----	----	----	----	----	----	
		High volume, multilane Speeds over 45 mph	----	----	----	----	----	----	25 - water spread shoulder or parking lane	----	----	
		High volume, multilane Speeds under 45 mph	----	----	----	----	----	----	10 - water spread 1/2 of outer lane	----	----	
		Low Volume, rural Speeds over 45 mph	----	----	----	----	----	----	25 - water spread shoulder or parking lane	----	----	
		Urban Speeds under 45 mph	----	----	----	----	----	----	10 - water spread as local standard	----	----	
		<b>ALL STATE HIGHWAYS</b>	----	----	----	----	----	----	----	----	----	
		Depressed Sections that require pumping :	----	----	----	----	----	----	---	----	----	
		Freeways and conventional State Highways	----	----	----	----	----	----	50-, water spread should not exceed that of adjacent roadway sections	----	----	
		Local Streets or road undercrossing	----	----	----	----	----	----	25-, water spread should not exceed that of adjacent roadway sections	----	----	

UDOT (h)	Channels - (no guidance for Roadside Ditches and Inlets)	Interstate Highways - Rural Principal Arterial and Rural Minor Arterial	----	----	----	----	----	----	10-year	----	----	channel freeboard shall be 1 foot or two velocity heads, whichever is greater
		Urban Principal Arterial and Rural Major Collector	----	----	----	----	----	----	10-year	----	----	"
		Urban Collect - Urban Local, Rural Minor Collector, Rural Local	----	----	----	----	----	----	10-year	----	----	channel side slopes at angle of repose or 1V:2H or flatter for rip-rap lining
NDOT (i)	Roadside Ditches and Inlets (assume info for channels)	Interstate Highways (includes Ramps)	----	----	----	----	----	----	50 year,	100-year	----	earthen channel side slopes shall be 3:1 or flatter and riprap-lined channels shall have 2:1 or flatter side slopes
		Principal Arterials (and Other Freeways and Expressways - includes ramps)	----	----	----	----	----	----	50 year	" "	----	channel freeboard shall be a minimum of 1 foot for the design storm
		Principal Arterials (Other Principal Arterial)	----	----	----	----	----	----	25 year	" "	----	
		Minor Arterials Collectors and Local Roads (Minor Arterials)	----	----	----	----	----	----	25 year	" "	----	
		Rural Major Collector	----	----	----	----	----	----	25 year	" "	----	
		Urban or Rural Minor Collector	----	----	----	----	----	----	10 year	" "	----	
		Frontage Roads (if not classified)	----	----	----	----	----	----	10 year	" "	----	
WYDOT (n)	Roadside Ditches and Inlets	----	----	----	----	----	----	----	----	----	----	----
CLARK COUNTY (j)	Roadside Ditches and Inlets	----	----	----	----	----	----	----	----	----	----	
		----	----	----	----	----	----	----	----	----	----	
		----	----	----	----	----	----	----	----	----	----	no info
		----	----	----	----	----	----	----	----	----	----	
Maricopa County (k)	Channels (not Roadside Ditches and Inlets)	Arterial - All weather crossings	----	----	----	----	----	----	50 - Max water surface at lowest adjacent road subgrade	100-Max depth = 6-inches in travel lane	----	Freeboard - 1-ft minimum for subcritical or greater based on an equation. 2 ft. minimum if supercritical or greater based on an equation
		Collector Streets	----	----	----	----	----	----	25 - Max water surface at lowest adjacent road subgrade	100-Max depth = 6-inches in travel lane	----	
		Local Streets	----	----	----	----	----	----	10 - Max water surface at lowest adjacent road subgrade	100-Max depth = 8-inches in travel lane	----	
Denver Urban Flood Control District (m)	Roadside Ditches and Inlets	----	----	----	----	----	----	----	----	----	----	no info
		----	----	----	----	----	----	----	----	----	----	
		----	----	----	----	----	----	----	----	----	----	
		----	----	----	----	----	----	----	----	----	----	

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DOT Name	Design Item	Roadway Type	ADT Range			Flood Name/ Return Period /No. of Lanes /Criteria			Comments / Other Criteria	
			All	>	<	Design Flood Return Period / Duration and Criteria	Check Flood Return Period / Duration and Criteria	Check Flood Return Period / Duration and Criteria		
						Two, Four and Six Lane Roads	Two Lane Roads	Two, Four and Six Lane Roads		
						note - 50 = 50-year return period storm, etc.				
NMDOT ( a )	Median Ditches and Inlets	Interstate Highways	Y	NA	NA	50 - Limit water depth to edge of shoulder	100 - Limit water depth to edge of driving lane	100- Limit water depth to edge of driving lane		
		Principal Arterials	Y	NA	NA	" "	" "	" "		
		Minor Arterials Collectors and Local Roads	NA	Rural >= 400 ADT and All Urban	NA	10 - Limit water depth to edge of shoulder	50 - Limit water depth to edge of driving lane	50- Limit water depth to edge of driving lane		
		Minor Arterials Collectors and Local Roads	NA	NA	Rural < 400 ADT	10 - Limit water depth to edge of shoulder	25 - Limit water depth to edge of driving lane	25- Limit water depth to edge of driving lane		
CDOT (Colorado) ( c )	Median Ditches and Inlets	Multi-lane Roads including Interstate	----	----	----	100	100 - does not exceed 1-ft increase over the existing 100-yr. FEMA floodplain or in the vicinity of an insurable building	----	same as for cross-drainage culverts and bridges	
		In Urban Areas	----	----	----	100	" "	----	" "	
		In Rural Areas	----	----	----	50	" "	----	" "	
		Two Lane Roads in Urban Areas	----	----	----	100	" "	----	" "	
		Two Lane Road in Rural Areas	----	----	----	50 if Q50>=4000 cfs	" "	----	" "	
		Two Lane Road in Rural Areas	----	----	----	50 if Q50<4000 cfs	" "	----	" "	
TXDOT (Texas) ( d )	Median Ditches and Inlets	Interstate Highways	----	----	----	----	----	----	assuming that the same criteria applies as for roadside ditches, but there is no mention of median ditches except for drains in median ditches	
		Principal Arterials	----	----	----	----	----	----		
			----	----	----	----	----	----		
		Minor Arterials	----	----	----	----	----	----		
		Local Roads	----	----	----	----	----	----		
AZDOT (Arizona) ( e , f )	Median Ditches and Inlets	Interstate Highways	----	----	----	----	----	----	it is preferable to slope median areas and inside shoulders to a center swale, to prevent drainage from the median area from running across the pavement. This is particularly important for high-speed facilities and for facilities with more than two lanes of traffic in each direction.	
		Principal Arterials	----	----	----	----	----	----		
		Minor Arterials Collectors and Local Roads	----	----	----	----	----	----		
		Minor Arterials Collectors and Local Roads	----	----	----	----	----	----		
			----	----	----	----	----	----		

CALTRANS (California) (g)	Median Ditches and Inlets (assume the same as spread for roadway drainage (Caltrans Table 831.3)	FREEWAYS -	----	----	----	----	----	----	
		Though traffic lanes, branch connections, and other major ramp connections	----	----	----	25 - water spread shoulder or parking lane	----	----	
		Minor Ramps	----	----	----	10 - water spread shoulder or parking lane	----	----	
		Frontage Road	----	----	----	10 - water spread as local standard	----	----	
		CONVENTIONAL HIGHWAYS	----	----	----	----	----	----	
		High volume, multilane Speeds over 45 mph	----	----	----	25 - water spread shoulder or parking lane	----	----	
		High volume, multilane Speeds under 45 mph	----	----	----	10 - water spread 1/2 of outer lane	----	----	
		Low Volume, rural Speeds over 45 mph	----	----	----	25 - water spread shoulder or parking lane	----	----	
		Urban Speeds under 45 mph	----	----	----	10 - water spread as local standard	----	----	
		ALL STATE HIGHWAYS	----	----	----	----	----	----	
UDOT (h)	Median Ditches and Inlets	Interstate Highways - Rural Principal Arterial and Rural Minor Arterial	----	----	----	10-year	----	----	Assume same as for roadside ditches - channel freeboard shall be 1 foot or two velocity heads, whichever is greater
		Urban Principal Arterial and Rural Major Collector	----	----	----	10-year	----	----	" "
		Urban Collect - Urban Local, Rural Minor Collector, Rural Local	----	----	----	10-year	----	----	Assume same as for roadside ditches - channel side slopes at angle of repose or 1V:2H or flatter for rip-rap lining
NDOT (i)	Median Ditches and Inlets (assume info for channels)	Interstate Highways (includes Ramps)	----	----	----	50 year	----	----	earthen channel side slopes shall be 3:1 or flatter and riprap-lined channels shall have 2:1 or flatter side slopes
		Principal Arterials (and Other Freeways and Expressways - includes ramps)	----	----	----	50 year	----	----	channel freeboard shall be a minimum of 1 foot for the design storm
		Principal Arterials (Other Principal Arterial)	----	----	----	25 year	----	----	
		Minor Arterials Collectors and Local Roads (Minor Arterials)	----	----	----	25 year	----	----	
		Rural Major Collector	----	----	----	25 year	----	----	
		Urban or Rural Minor Collector	----	----	----	10 year	----	----	
		Frontage Roads (if not classified)	----	----	----	10 year	----	----	
WYDOT (n)	Median Ditches and Inlets	----	----	----	----	----	----	----	
CLARK COUNTY (i)	Median Ditches and Inlets	----	----	----	----	----	----	no info	
		----	----	----	----	----	----		
		----	----	----	----	----	----		



Maricopa County (k)	Median Ditches and Inlets								
		----	----	----	----	----	----	----	
		----	----	----	----	----	----	----	no info
Denver Urban Flood Control District (m)	Median Ditches and Inlets	----	----	----	----	----	----	----	
		----	----	----	----	----	----	----	
		----	----	----	----	----	----	----	no info
		----	----	----	----	----	----	----	

---- No Data Found

a - NMDOT - Drainage Design Criteria for New Mexico Department of Transportation Projects Fourth Revision , June 2007

b - AASHTO - American Association of State Highway and Transportation Officials - AASHTO Drainage Manual, Volume 1 - Policy 2014

c - CDOT - Colorado Department of Transportation - Drainage Design Manual, 2004

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e - AZDOT - Arizona Department of Transportation, Highway Drainage Design Manual, Hydraulics, Final Report January, 2007

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g - CALTRANS - California Department of Transportation Highway Design Manual, var. dates of revision

h - UDOT - Utah Department of Transportation Manual of Instruction, Roadway Drainage, January 2004

i - NDOT - Nevada Department of Transportation Drainage Manual, NDOT Hydraulics, 2nd Edition, December 2006

j - Clark County Regional Flood Control District, Hydrologic Criteria and Drainage Design Manual, Adopted August 12, 1999

k - Drainage Design Manual for Maricopa County, Arizona - Hydraulics, 3rd Edition, August 2013

m - Urban Drainage and Flood Control District, Urban Storm Drainage Criteria Manual Vol. 1 and 2, April 2008, Vol. 3 2010

n - WYDOT - Operating Policy 18-6 - Drainage Design for Highway Systems, Feb. 15, 1979. (not available on the internet)

\*Wyoming states for drainage in their ACCESS MANUAL - Drainage in highway side ditches shall not be altered or impeded unless approved by WYDOT when drainage structures are required. Size and type of pipe and other design features shall be as directed by the Engineer having jurisdiction in the area. These costs and the costs of a drainage study, if required, shall be borne by the applicant or grantee.

DOT NAME	Design Item	Roadway Type	ADT Range			Flood Name/ Return Period /No. of Lanes /Criteria			COMMENTS / OTHER CRITERIA
			All	>	<	<u>Design</u> Flood Return Period / Duration and Criteria	<u>Check</u> Flood Return Period / Duration and Criteria	<u>Check</u> Flood Return Period / Duration and Criteria	
						Two, Four and Six Lane Roads	Two Lane Roads	Two, Four and Six Lane Roads	
						note - 50 = 50-year return period storm, etc.			
NMDOT ( a )	Trunk Lines	Interstate Highways	Y	NA	NA	50 - Limit hydraulic grade line to 1 foot below top of grate elevation	100 - Limit hydraulic grade line to the top of grate	100- Limit hydraulic grade line to the top of grate	
		Principal Arterials	Y	NA	NA	" "	" "	" "	
		Minor Arterials Collectors and Local Roads	NA	Rural >= 400 ADT and All Urban	NA	10 - Limit hydraulic grade line to 1 foot below top of grate elevation	50 - Limit hydraulic grade line to the top of grate	50- Limit hydraulic grade line to the top of grate	
		Minor Arterials Collectors and Local Roads	NA	NA	Rural < 400 ADT	10 - Limit hydraulic grade line to 1 foot below top of grate elevation	25 - Limit hydraulic grade line to the top of grate	25- Limit hydraulic grade line to the top of grate	
CDOT (Colorado) ( c )	Trunk Lines	Storm Drains (Major System)	----	----	----	100 - Multiple lanes - At least 1 lane open, for single lanes at least 8 ft of roadway open	----	----	specific HGL criteria not given, use allowable spread as HGL limit
		Storm Drains (Minor System)	----	----	----	2 to 10 year:	----	----	
		INTERSTATE	----	----	----	2-5, spread to shoulder	----	----	
		" "	----	----	----	10-yr, spread = shoulder + 3 ft	----	----	For all storm drains, velocities of flow will not be any greater than 22 ft/s, and no less than 3 ft/s
		ARTERIALS	----	----	----	< 45 mph, 2-5, spread = shoulder +4 ft	----	----	
		" "	----	----	----	< 45 mph, 10-, spread = shoulder + 3 ft	----	----	
		" "	----	----	----	>45 mph, 2-10-, spread = shoulder	----	----	
		" " sag point	----	----	----	< 45 mph, 50-, spread = shoulder	----	----	
		COLLECTORS	----	----	----	< 45 mph, 2-10 , spread = 1/2 driving lane	----	----	
		" "	----	----	----	> 45 mph, 2-5, spread = shoulder + 4 ft	----	----	
		" "	----	----	----	> 45 mph, 10-, spread = shoulder	----	----	
" " sag point	----	----	----	< 45 mph, 10-, spread = 1/2 driving lane	----	----			
LOCAL STREETS	----	----	----	2-10 , spread = 1/2 driving lane	----	----			
" "	----	----	----	10-, spread = 1/2 driving lane	----	----			
TXDOT (Texas) ( d )	Trunk Lines	Interstate Highways	----	----	----	10 year	----	----	NOTE - Interstates - 50-year for depressed roadways
		Principal Arterials	----	----	----	10 year	----	----	NOTE - Interstates - 50-year for depressed roadways
			----	----	----	----	----	----	
		Minor Arterials	----	----	----	10-, 5-, 2-year with recommendation on 5 year	----	----	NOTE - 50-yr and 25-yr (25 recommended) for depressed roadways
		Local Roads	----	----	----	----	----	----	
AZDOT (Arizona) ( e , f )	Trunk Lines	Interstate Highways	----	----	----	----	----	----	Non-depressed roadways: Storm drain systems - 10 year - Hydraulic grade line 6 in. below top of grate
		Principal Arterials	----	----	----	----	----	----	Depressed Roadways - storm drain systems - 50 year - Hydraulic grade line 6 in. below top of grate
		Minor Arterials Collectors and Local Roads	----	----	----	----	----	----	Both of the above apply
		Minor Arterials Collectors and Local Roads	----	----	----	----	----	----	Both of the above apply

CALTRANS (California) (g)	Trunk Lines	FREEWAYS -	----	----	----	----	----	closed conduit should be designed for the full flow condition. May be allowed to operated under pressure, provided the hydraulic gradient is 0.75 foot or more below the intake lip of any inlet that may be affected. The energy gradient should not rise above the lip of the intake.
		Though traffic lanes, branch connections, and other major ramp connections	----	----	----	25 - water spread shoulder or parking lane	----	
		Minor Ramps	----	----	----	10 - water spread shoulder or parking lane	----	
		Frontage Road	----	----	----	10 - water spread as local standard	----	
		CONVENTIONAL HIGHWAYS	----	----	----	----	----	
		High volume, multilane Speeds over 45 mph	----	----	----	25 - water spread shoulder or parking lane	----	NOTE - minimum storm drain is 18-inches
		High volume, multilane Speeds under 45 mph	----	----	----	10 - water spread 1/2 of outer lane	----	
		Low Volume, rural Speeds over 45 mph	----	----	----	25 - water spread shoulder or parking lane	----	
		Urban Speeds under 45 mph	----	----	----	10 - water spread as local standard	----	
		ALL STATE HIGHWAYS	----	----	----	----	----	
UDOT (h)	Trunk Lines	Depressed Sections that require pumping :	----	----	----	----	----	
		Freeways and conventional State Highways	----	----	----	50-, water spread should not exceed that of adjacent roadway sections	----	
		Local Streets or road undercrossing	----	----	----	25-, water spread should not exceed that of adjacent roadway sections	----	
		High Volume < 45 mph	---	---	---	10-yr spread - shoulder + 3 ft	---	these criteria apply to shoulder widths of 6 ft or greater, where shoulder widths are less than 6 ft, a minimum design spread of 6 feet should be considered.
		High Volume > 45 mph	---	---	---	10-yr. shoulder	---	" "
		High Volume sag point	---	---	---	50-year should + 3 ft	---	" "
		Collector < 45 mph	---	---	---	10-yr. 1/2 driving lane	---	" "
		Collector > 45 mph	---	---	---	10-yr. shoulder	---	" "
		Collector sag point	---	---	---	10-yr. 1/2 driving lane	---	" "
NDOT (i)	Trunk Lines	Local Streets high ADT	---	---	---	5-yr. 1/2 driving lane	---	" "
		Local Streets low ADT	---	---	---	10-yr. 1/2 driving lane	---	" "
		Local Streets sag point	---	---	---	10-yr. 1/2 driving lane	---	" "
		Interstate Highways (includes Ramps)	---	---	---	25-yr, No spread into adjacent travel lane	100-yr	For Design Storm hydraulic gradient shall remain at least one foot below that ground surface for the design flow. The intent is to ensure trunk line flows do not exit the system through inlets or manholes (3.3.2.2.3 Storm Drains). For 100-yr, evaluate as necessary to ensure no adverse impacts
		Principal Arterials (and Other Freeways and Expressways - includes ramps)	---	---	---	25-yr, No spread into adjacent travel lane	" "	
		Principal Arterials (Other Principal Arterial)	---	---	---	25-yr, Maximum spread to 1/2 lane	" "	
		Minor Arterials Collectors and Local Roads (Minor Arterials)	---	---	---	10 yr, Maximum spread to 1/2 lane	" "	
		Rural Major Collector	---	---	---	10 yr, Maximum spread to 1/2 lane	" "	
		Urban or Rural Minor Collector	---	---	---	10 yr, Maximum spread to 1/2 lane	" "	
		Frontage Roads (if not classified)	---	---	---	10 yr, Maximum spread to 1/2 lane	" "	
WYDOT (n)	Trunk Lines							
		Interstate Highways (includes Ramps)	----	----	----	small and large culverts and bridges 50 year - no max allowable spread into adjacent travel lane	100 year - depth over crown ,<= 6-in., depth over gutter flow line <=18 in.	----
		Principal Arterials (and Other Freeways and Expressways - includes ramps)	----	----	----	small culverts 25-year and large culverts and bridges 50 year - no max allowable spread into adjacent travel lane	100 year - depth over crown ,<= 6-in., depth over gutter flow line <=18 in.	----
		ALL Depressed Highways and Underpasses	----	----	----	small, large culverts and bridges 50 year - no maximum allowable spread on any traffic lanes	100 year - depth over crown ,<= 6-in., depth over gutter flow line <=18 in. adjacent buildings shall not be flooded	----
		Urban and Rural Minor Arterials Collectors and Local Roads (Minor Arterials)	----	>750	----	small and large culverts and bridges 25 year - no curb overtopping, must leave one lane open, large culverts 25 year	100 year - depth over crown ,<= 6-in., depth over gutter flow line <=18 in., adjacent buildings shall not be flooded	----
		Urban and Rural Minor Arterials Collectors and Local Roads (Minor Arterials)	----		<750	small culverts 10 year - large culverts and bridges 25-year, no curb overtopping, must leave one lane open	100 year - depth over crown ,<= 6-in., depth over gutter flow line <=18 in., adjacent buildings shall not be flooded	----
		Urban and Rural Collector	----	>750	----	small and large culverts and bridges 25 year - no curb overtopping, must leave one lane open	100 year - protect adjacent buildings from flooding, depth over gutter flow line <=18 in., adjacent buildings shall not be flooded	----
		Urban and Rural Collector	----		<750	small culverts 10 year, large culverts and bridges 25 year - no curb overtopping, must leave one lane open	100 year - protect adjacent buildings from flooding, depth over gutter flow line <=18 in., adjacent buildings shall not be flooded	----

CLARK COUNTY (j)	Trunk Lines	A Major Storm is defined as the 100-year storm	---	---	---	---	100-year, HGL 1 foot below street grade elevation	---	maximum velocity all storm drains is 25 ft / sec
		A Minor Storm is defined as the 10-year storm	---	---	---	10-year, HGL 1 foot below street grade elevation	---	---	
Maricopa County (k)	Trunk Lines and STREETS	Arterial - All weather crossings	---	---	---	10 - One 12-ft driving lane maintained in each direction, and flow depth not to exceed curb height	100-Max depth = 6-inches in travel lane	----	For all storm frequencies up to and including the 100-year :
		Minor Collector and Local streets	---	---	---	10 - flow depth not to exceed curb height	100-Max depth = 8-inches in travel lane	----	1. Channel and/or storm drain system installed as needed to meet street drainage criteria. 2. Historic drainage divides should be retained. Flows within existing streets should follow historic drainage paths. 3. Runoff to be contained 12 -inches below the finished floor of adjacent buildings. 4. Qmax = 100 cfs. 5. Vmax = Refer to Standard 6.2.2
Denver Urban Flood Control District (m)	Trunk Lines	Local -	----	----	----	10-5- or 2- (depends on local municipality). No curb overtopping , flow may spread to crown.	---	----	
		Collector	----	----	----	10-5- or 2- (depends on local municipality). No curb overtopping , flow spread must leave one lane free of water.	---	----	
		Arterial	----	----	----	10-5- or 2- (depends on local municipality). No curb overtopping , flow spread must leave one lane free of water in each direction, but should not flood more than two lanes in each direction.	---	----	
		Freeway	----	----	----	10-5- or 2- (depends on local municipality). No encroachment on any traffic lanes.	---	----	
		Local and Collector	----	----	----	---	100- Residential dwelling should be no less than 12-in. above the 100-year flood at the ground line or lowest water entry of a building. The depth of water over the gutter flow line should not exceed 18-in. for local and 12-in. for collector streets.	----	
		Arterial and Freeway	----	----	----	---	100- Residential dwelling should be no less than 12-in. above the 100-year flood at the ground line or lowest water entry of a building. The depth of water should not exceed the street crown to allow operation of emergency vehicles. The depth of water over the gutter flow line should not exceed 12-in.	----	

---- No Data Found

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DOT NAME	Design Item	Roadway Classification	ADT Range			Speed and Sag			Flood Name/ Return Period /No. of Lanes /Criteria			COMMENTS / OTHER CRITERIA
			All	>	<	< mph	> mph	sag point	<u>Design Flood Return Period / Duration and Criteria</u>	<u>Check Flood Return Period / Duration and Criteria</u>	<u>Check Flood Return Period / Duration and Criteria</u>	
									Two, Four and Six Lane Roads	Two Lane Roads	Two, Four and Six Lane Roads	
									note - 50 = 50-year return period storm , etc.			
NMDOT ( a )	Curb Drop Inlets	Interstate Highways	Y	----	----	----	----	----	50 -Limit water spread to half of driving lane	100 -Limit water spread to top of curb	100- Limit water spread to top of curb	
		Principal Arterials	Y	----	----	----	----	----	" "	" "	" "	
		Minor Arterials Collectors and Local Roads	----	Rural >= 400 ADT and All Urban	----				10 -Limit water spread to half of driving lane	50 -Limit water spread to top of curb	50-Limit water spread to top of curb	
		Minor Arterials Collectors and Local Roads	----	Y	Rural < 400 ADT	----	----	----	10 -Limit water spread to half of driving lane	25 -Limit water spread to top of curb	25- Limit water spread to top of curb	
		Minor Arterials Collectors and Local Roads	----	----	Y	----	----	----	10 - Limit water spread to half of driving lane	25 - Limit water spread to top of curb	25- Limit water spread to top of curb	
CDOT (Colorado) ( c )	Curb Drop Inlets	Storm Drains (Major System)	----	----	----	----	----	----	100 - Multiple lanes - At least 1 lane open, for single lanes at least 8 ft of roadway open	----	----	
		Storm Drains (Minor System)	----	----	----	----	----	----	2 to 10 year:	----	----	
		INTERSTATE	----	----	----	----	----	----	2-5, spread to shoulder	----	----	
		" "	----	----	----	----	----	----	10-yr, spread = shoulder + 3 ft	----	----	
		ARTERIALS	----	----	----	----	----	----	< 45 mph, 2-5, spread = shoulder +4 ft	----	----	
		" "	----	----	----	----	----	----	< 45 mph, 10-, spread = shoulder + 3 ft	----	----	
		" "	----	----	----	----	----	----	>45 mph, 2-10-, spread = shoulder	----	----	
		" " sag point	----	----	----	----	----	----	< 45 mph, 50-, spread = shoulder	----	----	
		COLLECTORS	----	----	----	----	----	----	< 45 mph, 2-10 , spread = 1/2 driving lane	----	----	
		" "	----	----	----	----	----	----	> 45 mph, 2-5, spread = shoulder + 4 ft	----	----	
		" "	----	----	----	----	----	----	> 45 mph, 10-, spread = shoulder	----	----	
		" " sag point	----	----	----	----	----	----	< 45 mph, 10-, spread = 1/2 driving lane	----	----	
TXDOT (Texas) ( d )	Curb Drop Inlets	LOCAL STREETS	----	----	----	----	----	----	2-10 , spread = 1/2 driving lane	----	----	
		" "	----	----	----	----	----	----	10-, spread = 1/2 driving lane	----	----	
		Interstate Highways	----	----	----	----	Y	----	10 year, roadway must be passable	----	----	NOTE - Interstates - 50-year for depressed roadways
		Principal Arterials	----	----	----	----	----	Y	10 year, roadway must be passable	----	----	NOTE - Interstates - 50-year for depressed roadways
		Minor Arterials	----	----	----	----	----	Y	10-, 5-, 2-year with recommendation on 5 year, roadway must be passable	----	----	NOTE - 50-yr and 25-yr (25 recommended) for depressed roadways
		Local Roads	----	----	----	----	----	----		----	----	

AZDOT (Arizona) ( e , f )	Allowable Spread and Pavement Drainage Systems	Two-lane roadway and two-way frontage road	----	----	----	----	----	----	10 year - shoulder, turn lane, or parking lane	----	----	12.5.9 Inlets The term "inlets" refers to all types of inlets such as grate inlets, curb inlets and slotted inlets. Drainage inlets are sized and located to limit the spread of water on traffic lanes to tolerable widths for the design storm in accordance with the design criteria specified in Chapter 600 of the Roadway Design Guidelines, RDG. The width of water spread on the pavement at sags should not be substantially greater than the width of spread encountered on continuous grades. Grate inlets and depression of curb opening inlets should be located outside the through traffic lanes to minimize the shifting of vehicles attempting to avoid them. All grate inlets shall be bicycle safe when used on roadways that allow bicycle travel. Curb inlets are preferred to grate inlets at major sag locations because of their debris handling capabilities.
		Multi-lane roadway and one-way frontage road	----	----	----	----	----	----	10 year - half lane + shoulder, turn lane, or parking lane	----	----	
		Ramp - one lane	----	----	----	----	----	----	10 year - Unponded width of 12 feet	----	----	
		Ramp - two lane	----	----	----	----	----	----	10 year - half lane plus the shoulder	----	----	
		One-lane directional ramp	----	----	----	----	----	----	10 - year - less than or equal to 8 feet	----	----	
		Two-lane directional ramp	----	----	----	----	----	----	10 - year - half lane plus shoulder	----	----	
		At ramp gores	----	----	----	----	----	----	fig 603.2 A in RDG	----	----	
		Auxiliary lanes	----	----	----	----	----	----	10 -year - half auxiliary lane plus shoulder	----	----	
									50-year for depressed roadways, HGL 6-in. below top of grate	----	----	
		* for one directional crowned roadways, the half-lane spread shall be included only on one side										
CALTRANS (California) ( g )	Curb Drop Inlets / ALLOWABLE SPREAD	FREEWAYS -	----	----	----	----	----	----	----	----	----	closed conduit should be designed for the full flow condition. May be allowed to operated under pressure, provided the hydraulic gradient is 0.75 foot or more below the intake lip of any inlet that may be affected. The energy gradient should not rise above the lip of the intake.
		Though traffic lanes, branch connections, and other major ramp connections	----	----	----	----	----	----	25 - water spread shoulder or parking lane	----	----	
		Minor Ramps	----	----	----	----	----	----	10 - water spread shoulder or parking lane	----	----	
		Frontage Road	----	----	----	----	----	----	10 - water spread as local standard	----	----	
		CONVENTIONAL HIGHWAYS	----	----	----	----	----	----	----	----	----	
		High volume, multilane Speeds over 45 mph	----	----	----	----	----	----	25 - water spread shoulder or parking lane	----	----	
		High volume, multilane Speeds under 45 mph	----	----	----	----	----	----	10 - water spread 1/2 of outer lane	----	----	
		Low Volume, rural Speeds over 45 mph	----	----	----	----	----	----	25 - water spread shoulder or parking lane	----	----	
		Urban Speeds under 45 mph	----	----	----	----	----	----	10 - water spread as local standard	----	----	
		ALL STATE HIGHWAYS	----	----	----	----	----	----	----	----	----	
		Depressed Sections that require pumping :	----	----	----	----	----	----	----	----	----	
		Freeways and conventional State Highways	----	----	----	----	----	----	50-, water spread should not exceed that of adjacent roadway sections	----	----	
		Local Streets or road undercrossing	----	----	----	----	----	----	25-, water spread should not exceed that of adjacent roadway sections	----	----	

[illegible]

MARICOPA COUNTY (K)	Curb Drop Inlets	Arterial - All weather crossings	----	----	----	----	----	----	10 - One 12-ft driving lane maintained in each direction, and flow depth not to exceed curb height	100-Max depth = 6-inches in travel lane	----	For all storm frequencies up to and including the 100-year :
		Minor Collector and Local streets	----	----	----	----	----	----	10 - flow depth not to exceed curb height	100-Max depth = 8-inches in travel lane	----	
Denver Urban Flood Control District (m)	Curb Drop Inlets	Local -	----	----	----	----	----	----	10-5- or 2- (depends on local municipality).	---	----	
		Collector	----	----	----	----	----	----	10-5- or 2- (depends on local municipality). No curb overtopping, flow spread must leave one lane free of water.	---	----	
		Arterial	----	----	----	----	----	----	10-5- or 2- (depends on local municipality). No curb overtopping, flow spread must leave one lane free of water in each direction, but should not flood more than two lanes in each direction.	---	----	
		Freeway	----	----	----	----	----	----	10-5- or 2- (depends on local municipality). No encroachment on any traffic lanes.	---	----	
		Local and Collector	----	----	----	----	----	----	---	100- Residential dwelling should be no less than 12-in. above the 100-year flood at the ground line or lowest water entry of a building. The depth of water over the gutter flow line should not exceed 18-in. for local and 12-in. for collector streets.	----	
		Arterial and Freeway	----	----	----	----	----	----	---	100- Residential dwelling should be no less than 12-in. above the 100-year flood at the ground line or lowest water entry of a building. The depth of water should not exceed the street crown to allow operation of emergency vehicles. The depth of water over the gutter flow line should not exceed 12-in.	----	

---- No Data Found

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j - Clark County Regional Flood Control District, Hydrologic Criteria and Drainage Design Manual, Adopted August 12, 1999

k - Drainage Design Manual for Maricopa County, Arizona - Hydraulics, 3rd Edition, August 2013

m - Urban Drainage and Flood Control District, Urban Storm Drainage Criteria Manual Vol. 1 and 2, April 2008, Vol. 3 2010

n - WYDOT - Operating Policy 18-6 - Drainage Design for Highway Systems, Feb. 15, 1979. (not available on the internet)

\*Wyoming states for drainage in their ACCESS MANUAL - Drainage in highway side ditches shall not be altered or impeded unless approved by WYDOT when drainage structures are required. Size and type of pipe and other design features shall be as directed by the Engineer having jurisdiction in the area. These costs and the costs of a drainage study, if required, shall be borne by the applicant or grantee.