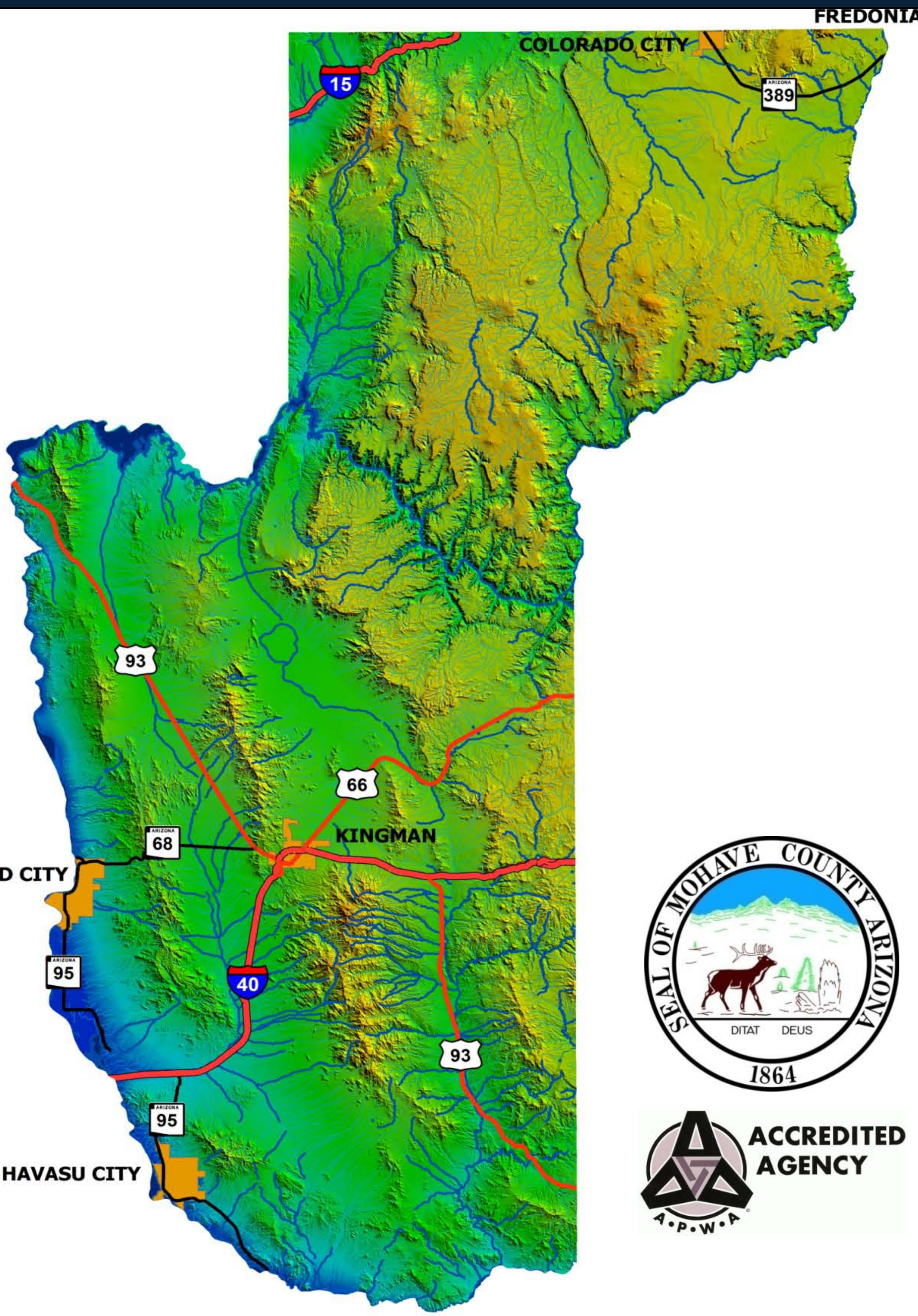
DRAINAGE DESIGN MANUAL FOR MOHAVE COUNTY



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ACCREDITED AGENCY
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ACKNOWLEDEMENTS

The information, procedures, and recommendations that are presented in this manual are mainly the result of previously published efforts of many diligent and talented engineers and scientists. The author of this manual has made every effort to cite the original authors and researchers whose contributions to the profession are greatly appreciated.

The author of this manual is indebted to the many individuals and organizations, including the staff at Mohave County, that have supported this effort through recommendations, technical guidance, encouragement, and review of draft sections. Special thanks go to Mr. Philip Wisely, who started the process of creation of this manual while with Mohave County.

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Agencies that have contributed greatly to this manual through their research efforts in producing technical material for use by the profession are:

Arizona Department of Transportation

City of Phoenix, AZ

Flood Control District of Maricopa County

Urban Drainage and Flood Control District (Denver)

US Department of Agriculture, Agricultural Research Service

US Department of Agriculture, Natural Resources Conservation Service

US Army Corps of Engineers

US Department of Transportation

COMMENTS

Users of this manual are strongly encouraged to submit any comments, criticisms, or findings of errors. This information should be addressed to:

Flood Control District Engineer
Mohave County Flood Control District
P.O. Box 7000
Kingman, AZ 86402-7000

Because of ongoing legal and technical changes in the field of stormwater management, revisions to this manual will be required from time to time. Such revisions will take place on an ongoing, as needed basis and will be posted on the Mohave County Drainage Design Manual web page (http://www.co.mohave.az.us/FloodControlDrainageDesignManual). A separate document available on the Mohave County web page will summarize revisions made after the release of this first edition.

REVISIONS

1st Edition: August 24, 2009

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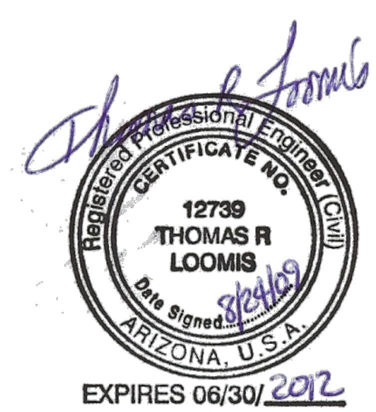

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LIST OF ACRONYMS AND ABBREVIATIONS

The following are abbreviations used throughout the document pertaining to approvals that must be obtained before proceeding:

MOHAVE COUNTY ENGINEER: Requires written approval by the Mohave County Engineer.

COUNTY: Requires written approval by a Mohave County staff member designated by the MOHAVE COUNTY ENGINEER as having approval authority.

DISTRICT ENGINEER: Requires written approval by the Mohave County Flood Control District Director.

DISTRICT: Requires written approval by a Mohave County Flood Control District staff member designated by the DISTRICT ENGINEER as having approval authority.

COUNTY/DISTRICT: Requires written approval by either COUNTY or DISTRICT or both, at the discretion of the Mohave County Engineer.

Board of Supervisors: Requires approval action by the Mohave County Board of Supervisors.

Board of Directors: Requires approval action by the Mohave County Flood Control District Board of Directors.

The following are general abbreviations and acronyms used throughout this document:

A.R.S.	Arizona Revised Statute
ACPA	American Concrete Pipe Association
ADA	Americans with Disabilities Act
ADEQ	Arizona Department of Environmental Quality
ADOT	Arizona Department of Transportation
ADOT Standards	ADOT Standard Specifications for Road & Bridge Construction and Standard Drawings
ADWR	Arizona Department of Water Resources
AISI	American Iron and Steel Institute

APP	Aquifer Protection Permit
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
AZPDES	Arizona Pollutant Discharge Elimination System
BFE	Base Flood Elevation. The elevation of the 100-year water surface
BMP	Best Management Practice
CAFO	Concentrated Animal Feeding Operation
CGP	Construction General Permit
CLOMA	Conditional Letter of Map Amendment
CLOMR	Conditional Letter of Map Revision
CLOMR-F	Conditional Letter of Map Revision Based on Fill
CSP	Corrugated Steel Pipe
CSPI	Corrugated Steel Pipe Institute
CWA	Clean Water Act
D-D-F	Depth -Duration - Frequency
DDM	Drainage Design Manual for Mohave County
DDMSW	Drainage Design Modeling System for Windows computer software by KVL Consultants, Inc. Refers to the Mohave County-specific version
DGP	De Minimus General Permit
DTHETA	Volumetric soil moisture deficit at the start of rainfall
ELEV	Elevation
EPA	U.S. Environmental Protection Agency
FCDMC	Flood Control District of Maricopa County
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration

FIRM	Flood Insurance Rate Map
FR	Federal Register
GIS	Geographical Information System
HEC-1	United States Army Corps of Engineers Hydrologic Engineering Center computer program for hydrologic modeling (DOS-based)
HEC-HMS	United States Army Corps of Engineers Hydrologic Engineering Center Hydrologic Modeling System computer program for hydrologic modeling (Windows-based)
HEC-RAS	United States Army Corps of Engineers Hydrologic Engineering Center River Analysis System computer program for hydraulic modeling of open channels and rivers (Windows-based)
Hydraulics Manual	FCDMC Drainage Design Manual - Hydraulics (most recent draft)
IA	Initial abstraction
I-D-F	Intensity - Duration - Frequency
LOMA	Letter of Map Amendment
LOMR	Letter of Map Revision
LOMR-F	Letter of Map Revision Based on Fill
MAG	Maricopa Association of Governments
MAG Standards	MAG Uniform Standard Specifications and Details for Public Works Construction
MCFCD	Mohave County Flood Control District
MSGP	Multi-Sector General Permit
MUTCD	Manual on Uniform Traffic Control Devices
NFIP	National Flood Insurance Program
NMIN	HEC-1 computation time interval
NOAA	National Oceanic and Atmospheric Administration

NOI	Notice of Intent
NOT	Notice of Termination
NPDES	National Pollutant Discharge Elimination System
NRCS	Natural Resources Conservation Service
NSTPS	HEC-1 number of computation steps for routing operations
NWP	Nationwide General Permit
NWS	National Weather Service
PMR	Physical Map Revision
PSIF	Wetting front capillary suction
R/W	Right-of-way
RGRCP	Rubber gasketed reinforced concrete pipe
RLNTH	HEC-1 routing reach length
RTIMP	Percent impervious
RUSLE	Revised Universal Soil Loss Equation
SEL	Slope of the energy gradeline
SIC	Standard Industrial Classification
SMU	Soil Map Unit.
SS	ADWR State Standard
SSA	ADWR State Standard Attachment
Standard Details	Uniform Standard Details for Mohave County
SWMP	Storm Water Management Plans
SWPPP	Storm Water Pollution Prevention Plans
TMDL	Total Maximum Daily Load
U.S.	United States
UDFCD	Urban Drainage and Flood Control District (Denver)

USACE	United States Army Corps of Engineers
USBR	U.S. Bureau of Reclamation
USFS	Unites States Forest Service
USGS	United States Geological Survey
WMS	Watershed Modeling System windows-based computer program by Environmental Modeling Systems, Inc
XKSAT	Hydraulic conductivity at natural saturation

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1 INTRODUCTION

It is the intent of Mohave County to have a comprehensive storm water management program that protects the health, safety and welfare of its citizens, their property, and the environment. The County's storm water management documents include the following:

Mohave County Flood Control Ordinance - 2000

Mohave County Land Division Regulations (2004)

Mohave County Zoning Ordinance (2005)

Mohave County Engineering Design Standards, Specifications and Details (2002)

Drainage Design Manual for Mohave County (this document)

The addition of this Drainage Design Manual provides the technical basis for sound storm water management. This is a joint document of Mohave County and the Mohave County Flood Control District (COUNTY/DISTRICT).

1.1 PURPOSE

The purpose of the Drainage Design Manual for Mohave County (DDM) is to supplement the Mohave County Land Division Regulations by providing minimum requirements and guidelines for addressing storm water issues associated with new and existing development. A secondary purpose is to provide guidance and detail for implementation of the DISTRICT's Floodplain Ordinance. It is intended that drainage studies, plans, design reports, construction drawings and accompanying drainage/floodplain use permit applications prepared in accordance with the philosophies, policies and minimum standards contained herein will meet the minimum requirements of the governing regulations. This will expedite the review, approval and permitting processes.

The document presents the COUNTY/DISTRICT philosophy on drainage and floodplain management, and planning for drainage facilities. It contains descriptions of federal, state and county regulations pertaining to such facilities, including links to the various COUNTY/DISTRICT regulations that can be found on the Internet. Most importantly, the policies and minimum technical standards for addressing drainage and floodplain management issues are presented. These policies and standards are based on flood and erosion hazard mitigation strategies that are intended to meet the minimum requirements under state and federal law with the effect of reducing or eliminating cumulative impacts resulting from development and enhancing public safety.

1.2 DISCLAIMER

COUNTY/DISTRICT will review and approve flood hazard delineation studies, drainage reports and plans for construction projects for conformance with the DDM, the DISTRICT's floodplain ordinance, and the COUNTY land division regulations and zoning ordinance, as appropriate under their separate authorities (refer to Chapter 2). This notwithstanding, COUNTY/DISTRICT assumes no liability for insufficient design or improper construction. Review and approval does not absolve the owner, developer, design engineer, or contractor of liability for inadequate design or poor construction. The design engineer has the responsibility to design drainage facilities that meet standards of practice for the industry and promote public safety. Compliance with the regulatory elements, and meeting the policies and minimum design standards, does not guarantee that properties will be free from flooding or flood damage. COUNTY/DISTRICT, and their officials or employees assume no liability for information, data, or conclusions prepared by private engineers or environmental professionals and make no warranty expressed or implied in their review/approval of drainage/floodplain projects or studies including stormwater quality submittals, or for use of the technical data and procedures provided in the DDM.

The Mohave County Flood Control District, a political subdivision of the State of Arizona, has compiled for its use certain information, including but not limited to, point rainfall data and soils parameters. This information is provided as public record as a part of the DDM and is available to assist in identifying general areas of concern only. The information provided should only be relied upon with corroboration of the methods, assumptions, and results by a qualified independent source. The user's reliance upon the accuracy, reliability and authority of this information is solely the user's responsibility. The user of this information releases, indemnifies and holds free the Mohave County Flood Control District and Mohave County from any and all liabilities, damages, lawsuits and causes of action that result as a consequence of his/her reliance on and use of information provided as a public record.

The Drainage Design Management System for Windows (DDMSW) computer software program is provided as a public service to aid in implementation of the technical information, data, and procedures presented in the DDM. The user of this information releases, indemnifies and holds free the Mohave County Flood Control District and Mohave County from any and all liabilities, damages, lawsuits and causes of action that result as a consequence of his/her reliance on and use of the DDMSW computer program and the data supplied with it.

1.3 APPLICATION

Policies, standards, and philosophies set forth in this document apply to private development projects within the unincorporated areas of Mohave County, projects funded entirely by COUNTY/DISTRICT, and projects funded in cooperation with COUNTY and/or the DISTRICT and/or other agencies, or for those communities where the DISTRICT has floodplain management responsibilities. These policies and standards also apply, in an advisory capacity, to state funded and federally funded projects sponsored by COUNTY and/or the DISTRICT.

It is understood that there may be exceptions to the policies and standards that may be granted by COUNTY and/or the DISTRICT. The standards are minimum standards. There may be more stringent requirements in the event that public health, safety and welfare could be adversely affected by application of the minimum standard.

1.4 SCOPE

The Drainage Design Manual for Mohave County (DDM) is divided into twenty-one (21) chapters that address the major administrative areas of drainage and stormwater management. The intent of this manual is to provide rules and guidelines for the design of drainage and stormwater facilities. The purpose of each chapter is as follows:

Chapter 2, Legal Aspects: Federal, state and local regulatory requirements are outlined for the convenience of the user. These requirements change periodically. It is the user's responsibility to verify the content.

Chapter 3, Drainage Planning: This chapter stresses the COUNTY/DISTRICT vision for drainage and stormwater management and provides guidance for the drainage master planning process.

Chapter 4, Public Safety: The DDM is focused on public safety. Therefore, a separate chapter is provided to emphasize key policies and standards directly addressing safety issues.

Chapter 5, General Policies and Standards: The standard details and specifications accepted for use in Mohave County are listed as well as the Mohave County policy for accepting existing structures and/or facilities into the Mohave County maintenance system.

Chapter 6, Stormwater Quality: This chapter sets forth the COUNTY/DISTRICT approach to improving stormwater quality, which is based in the use of stormwater retention basins for new development.

Chapter 7, Hydrology: The design storm criteria for the various aspects of drainage design and floodplain management are defined, and the hydrologic methodologies for estimating peak discharges are set forth.

Chapter 8, Floodplain Management: Policies and standards for administering the Mohave County Flood Control Ordinance and meeting state and federal requirements for floodplain management are provided.

Chapter 9, Roadway Drainage: Policies and standards for design of surface roadway drainage, including catch basins, are provided.

Chapter 10, Storm Drains: Policies and standards for design of underground storm drains are provided.

Chapter 11, Culverts and Bridges: Policies and standards for design of culverts and bridges under roadways are provided.

Chapter 12, Open Channels: Standards for design of open channels, including lined and unlined channels, are provided.

Chapter 13, Friction Losses in Open Channels and Pipes: Standards for estimating roughness coefficients for design of open channels, pipe culverts and storm drains, and for floodplain delineations are provided.

Chapter 14, Hydraulic Structures: Standards for design of hydraulic structures, including drop and grade control structures, and trash racks are provided.

Chapter 15, Stormwater Storage: Policies and standards for design of retention and detention basins are provided.

Chapter 16, Erosion Control During Construction: Appropriate erosion control measures at construction sites are defined.

Chapter 17, Sedimentation: The policies and standards in this chapter define the conditions where sedimentation should be considered as an important design parameter.

Chapter 18, Documentation Requirements: The requirements for preparing hydrology and hydraulics drainage reports and drainage improvement construction drawings are defined.

Chapter 19, Revision Process: The formal process for revising the DDM is described.

Chapter 20, Reference: Documents cited as references in the DDM are listed.

Chapter 21, Glossary: Technical terms used in the DDM are defined.

Various appendices are also provided containing technical documentation and examples to supplement the information in the main body of the DDM.

1.5 SOURCE REFERENCES

There are many documents cited as references in the DDM. However, there are three documents that are very important key references and warrant special acknowledgement.

1.5.1 ADOT HYDROLOGY MANUAL

The ADOT Hydrology Manual (ADOT, 1993) is the basis for Chapter 7 Hydrology. Many of the procedures are adapted from this manual and portions of the text are used with minor modifications specific to Mohave County morphology. The user is referred to ADOT (1993) for more detailed explanations and descriptions of most hydrologic procedures, including the Clark unit hydrograph. Mohave County acknowledges the significant benefit to the citizens of the State of Arizona and its practicing civil engineering professionals from the research and extensive effort that went into creation of the ADOT Hydrology Manual. The hydrologic methodology and procedures recommended for use in Mohave County would not be possible without this document.

1.5.2 FCDMC HYDRAULICS MANUAL

The Flood Control District of Maricopa County Drainage Design Manual - Hydraulics (Hydraulics Manual) (FCDMC, 2009b) is the key reference for chapters 10 through 15, and 17. The Hydraulics Manual is a state-of-the-art document for design of drainage facilities in arid regions and is recognized on a national level by its use and/or adoption by agencies in Arizona and outside of Arizona. The policies and standards in the listed chapters of the DDM are intended to provide Mohave County-specific guidance for application of FCDMC (2009b). The Flood Control District of Maricopa County is commended for the contribution made by creation of this document so that the public as a whole can benefit from the extensive research effort that went into its creation.

1.5.3 FCDMC EROSION CONTROL MANUAL

The Flood Control District of Maricopa County Drainage Design Manual - Erosion Control (FCDMC, 2009c) is the key reference for chapter 16. FCDMC (2009c) is a state-of-the-art document for "Best Management Practices" for control of erosion during construction in arid regions and is recognized on a national level by its use and/or adoption by agencies in Arizona and outside of Arizona. The policies and standards in chapter 16 of the DDM are intended to provide Mohave County-specific guidance for application of FCDMC (2009c). The Flood Control District of Maricopa County is again commended for the contribution made by creation of this document so that the public as a whole can benefit from the extensive effort that went into its creation.

1.5.4 CITY OF PHOENIX STORM WATER POLICIES AND STANDARDS

Special acknowledgement is due the City of Phoenix for their *Storm Water Policies and Standards* manual (Phoenix, 2004). That manual is a key reference and source of information for Chapters 2, 3, 4, and 8 through 18. The City of Phoenix was the first agency in Maricopa County to write a stormwater policies and standards document that tailored the technical guidance in the FCDMC Drainage Design Manual to the unique characteristics and needs of their jurisdiction. The City of Phoenix thereby set a standard for other agencies within Maricopa County to follow and build on.

1.6 STATUTORY AUTHORITY

This manual is adopted pursuant to Arizona Revised Statutes (ARS) 11-151, Sections 30 and 36 and ARS 11-251.05 which authorizes the Board of Supervisors to adopt and enforce all ordinances necessary to the full discharge of the duties of the Board of Supervisors as the legislative authority of the county government; and to enforce standards for excavation, landfill and grading to prevent unnecessary loss from erosion, flooding and landslides. This manual provides policies and standards in support of the Mohave County Land Division Regulations.

Also, sections 48-3603 and 48-3609 of the ARS direct each County Flood Control District Board of Directors to adopt and enforce floodplain regulations consistent with criteria adopted by the Director of Arizona Department of Water Resources pursuant to ARS 48-3605. Therefore, the Board of Directors of the Mohave County Flood Control District adopts these policies and standards applicable to floodplain management in support of the Mohave County Flood Control Ordinance.

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2 LEGAL ASPECTS

2.1 INTRODUCTION

This chapter is intended to provide an overview of pertinent federal and state regulations that address drainage and drainage related issues. COUNTY/DISTRICT regulations, policies and standards meet these minimum requirements. Local regulations are also included in this chapter with a description of the permitting process pertinent to the unincorporated areas of Mohave County. The differences between Federal, State, and local regulations affecting drainage and floodplain management are not set forth in this chapter.

Engineers responsible for drainage design must conform to all regulations that may affect their project including federal, state and local acts, codes, laws, regulations, ordinances, standards and policies. Although these regulations are constantly changing, the following discussion provides some guidance as to the areas where federal, state and local governmental agencies exercise control over drainage related activities in unincorporated Mohave County.

2.2 WATER AND CULTURAL RESOURCE AGENCY CONTACT LIST

The list that follows identifies the various agencies one may need to contact to obtain information or file a permit for drainage projects. This list is provided as assistance and for information purposes only. This list may not include all agencies or environmental reviews or permits that are required for a given project. Telephone numbers and addresses are subject to change.

Table 2.1 Water and cultural resource agency contact list

General Information	
Arizona Department of Environmental Quality (ADEQ) (602) 771-2300, Main Number (602) 771-4881, Ombudsman (602) 771-2330, Emergency Response Line web site: http://www.azdeq.gov/	Arizona Department of Water Resources (ADWR) (602) 771-8500 web site: http://www.azwater.gov/azdwr
Environmental Protection Agency (EPA) Public Information Center: (415) 947-8000 (866) EPA-WEST web site: www.epa.gov/region9	
Floodplain Information	
Federal Emergency Management Agency (510) 627-7100 (Oakland) (202) 566-1600 (Washington D.C.) (800) 621-FEMA web site: www.fema.gov	Mohave County Flood Control District (928) 757-0925 Web site: http://legacy.co.mohave.az.us/pw/Flood%20Control/Flood%20Control.htm
Clean Water Act Section 404 Permits	
US Army Corps of Engineers (602) 640-2015 web site: http://www.usace.army.mil	
National Pollutant Discharge Elimination System (NPDES) Permits	
EPA (415) 972-3510	ADEQ (602) 771-2300
Aquifer Protection Permits	**Drywell Permits**
ADEQ (602) 771-2300	ADEQ (602) 771-2300 (877) 800-3207 – Hotline
Groundwater & other Water Permits	
ADEQ (602) 771-2300	ADWR (602) 771-8500
Water Quality Certification 401 Permits	**State Species of Concern**
ADEQ (602) 771-2300	Arizona Game & Fish Department (602) 942-3000 http://www.azgfd.gov

Table 2.1 Water and cultural resource agency contact list

Native Plant Law	Endangered Species Act
Arizona Dept. of Agriculture Plant Services Division (602) 542-0994 web site: http://www.azda.gov	U.S. Fish & Wildlife Service (602) 242-0210 web site: http://www.fws.gov/southwest/es/arizona/
Historic & Prehistoric Sites	
State Historic Preservation Office (602) 542-4009 web site: http://www.pr.state.az.us	
Native American Community Contacts, Mohave County	
Chemehuevi Tribal Council (760) 858-4301	Hualapai Tribe (928) 769-2216
Fort Mohave Indian Tribe (760) 629-4591	Kaibab Paiute Tribe (928) 643-7245
Havasupai Tribe (928) 448-2731	

2.3 FEDERAL

2.3.1 NATIONAL FLOOD INSURANCE PROGRAM

2.3.1.1 Introduction

The National Flood Insurance Act of 1968, as amended in 1973, provides for a federally subsidized National Flood Insurance Program (NFIP) conditioned on active management and regulation of development by states and local governments. The Federal Emergency Management Agency (FEMA) administers the NFIP as a part of its overall responsibilities in preventing and responding to natural events that damage private and public property and any life-threatening natural event including floods. The NFIP provides flood insurance at affordable rates through Federal subsidy of the insurance offered by licensed insurance agents. This insurance is designed to provide an insurance alternative to disaster assistance to meet the escalating costs of repairing damage to buildings and their contents caused by floods.

Participation in the NFIP is based on an agreement between local communities and the Federal Government. This agreement states if a community will adopt and enforce a floodplain management ordinance(s) to reduce future flood risks to new construction, the Federal Government will make flood insurance available within the community as a financial protection against flood losses.

Availability of the subsidized flood insurance is contingent upon the development of a floodplain management system by the local municipality. Prevention of flood related property damage is achieved through the delineation of property subject to flood events and the establishment of specific rules concerning development within these identified Special Flood Hazard Areas (SFHA). FEMA publishes Flood Insurance Rate Maps (FIRM) for certain flood prone areas that delineate different SFHA's.

Mohave County has participated in the NFIP since the 1970's, received their first FIRM panels in March 1982, and has adopted floodplain regulations, through the MCFCD, and ordinances so that its citizens have access to the subsidized insurance. The role of the community is to enact and implement floodplain management ordinances required for participation in the NFIP.

2.3.1.2 Community Rating System

The NFIP Community Rating System (CRS) was implemented in 1990 as a program for recognizing and encouraging community floodplain management activities that exceed the minimum NFIP standards. The National Flood Insurance Reform Act of 1994 codified the Community Rating System in the NFIP. Under the CRS, flood insurance premium rates are adjusted to reflect the reduced flood risk resulting from community activities that meet the three goals of the CRS: (1) reduce flood losses; (2) facilitate accurate insurance rating; and (3) promote the awareness of flood insurance. Mohave County has been a member of the CRS program since 1995.

2.3.1.3 FEMA Special Flood Hazard Areas

Citizens within Mohave County are required to ascertain whether or not their respective property is located in a FEMA SFHA before commencing with any building or land disturbance activity. FEMA FIRM's are available for review at the MCFCD, Mohave County, and the Arizona Department of Water Resources. The FIRM's are used to determine if a property is located within a SFHA regulated by FEMA.

2.3.1.4 Flood Hazard Zones

The flood hazard maps are subdivided into zones that relate to flooding hazards. These are defined as follows:

1. **100-year Floodplain:** Floodplain resulting from the occurrence of the 100-year rainfall. FEMA sets its jurisdictional limits to the 100-year event, which is cited as the base flood elevation. The 100-year event is an event that has a one (1) percent chance of occurring in any given year. Jurisdictional limits are defined by horizontal flooding limits using the base flood elevation. The 100-year floodplain is divided by FEMA into the following hazard zones for flood insurance rating purposes:
   a. Zone A: No base flood elevations determined.
   b. Zone AE: Base flood elevations determined.
   c. Zone AH: Flood depths of 1 to 3 feet (usually areas of ponding), base flood elevations determined.
   d. Zone AO: Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain), average depths determined (and velocities determined for alluvial fan floodplains).
   e. Zone X (shaded): Areas of 500-year flood; areas of 100-year flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 100-year flood.
   f. Zone X (unshaded): Areas determined to be outside 500-year floodplain.
2. **Floodway:** That portion of the 100-year floodplain that is required to convey the 100-year flood and that cannot be obstructed to an extent causing a rise in water surface elevation greater than 1 foot. The allowable rise and the limits of the floodway are predetermined by the governing municipality.

2.3.1.5 Application Process

Refer to Section 8.3.5 for the floodplain use permit process for a single building lot in Mohave County.

2.3.1.6 Approval Actions Taken by FEMA

If a property is determined to be located within a FEMA SFHA after reviewing the appropriate FIRM, there are several approval options available that, if desired and applicable, the landowner must process through FEMA. The landowner must select the permit option that best fits the need of the property and satisfies FEMA requirements. Each permit option requires completion of specific application forms and may require that a registered land surveyor or professional engineer complete the forms. Each permit/application form is identified below by name followed by a brief description of the approval response to be expected from FEMA.

1. **Conditional Letter of Map Amendment (CLOMA)** - A letter from FEMA stating that a proposed structure that is not to be elevated by fill would not be inundated by the 100-year flood if built to the proposed finished floor elevation.

2. **Letter of Map Amendment (LOMA)** - A letter from FEMA stating that an existing structure or parcel of land that has not been elevated by fill would not be inundated by the 100-year flood.

3. **Conditional Letter of Map Revision Based on Fill (CLOMR-F)** - A letter from FEMA stating that a parcel of land or proposed structure that is to be elevated by fill would not be inundated by the 100-year flood if fill is placed on the parcel as proposed or the structure is built as proposed.

4. **Letter of Map Revision Based on Fill (LOMR-F)** - A letter from FEMA stating that an existing structure or parcel of land that has been elevated by fill would not be inundated by the 100-year flood.

Application forms for the four items listed above can be obtained from FEMA by reference MT-1 FEMA FORM 81-87 SERIES. FEMA's contact address is provided at the end of this section.

1. **Conditional Letter of Map Revision (CLOMR)** - A letter from FEMA commenting on whether a proposed project, if built as proposed, would justify a map revision.

2. **Letter of Map Revision (LOMR)** - A letter from FEMA officially revising the current FIRM to show changes to floodplains, floodways, or flood elevation. Physical changes include watershed development, flood control structures, etc.

3. **Physical Map Revision (PMR)** - A reprinted FIRM incorporating changes to floodplains, floodways, or flood elevations. Because of the time and cost involved to change, reprint, and redistribute a FIRM, a PMR is usually processed when a revision reflects increased flood hazards or large-scope changes.

Application forms for the three items listed above can be obtained from FEMA by reference MT-2 FEMA FORM 81-89 SERIES. FEMA's contact address is provided at the end of this section.

Projects receiving a conditional letter must re-apply for a letter of amendment or revision upon completion of construction. The conditional letter allows financing and local approvals, and/or occupancy of the structure to take place. To initiate FEMA review for a specific activity or location, a letter to FEMA requesting one of the “conditional” letters is sent to FEMA along with supporting data which includes a signed letter from Mohave County Flood Control District indicating its concurrence with the request. Supporting data may be in the form of improved methodology or improved survey data. Improved methodology may be a different technique (model) or adjustments to models used in the effective FIS. Improved survey data include revised as well as new data. Floodway revisions involve any shift in the FEMA-designated floodway boundaries, regardless of whether the shift results in a change that is measurable at the scale of a DFIRM panel.

2.3.1.7 Construction in Special Flood Hazard Areas

The lowest floor of all residential structures constructed in the SFHA must be constructed to a minimum of the Base Flood Elevation (BFE)[1]. Building structures located within the SFHA (but not within the Floodway) may be protected from floods up to and including the 100-year flood by placement of fill to elevate the structure to or above the BFE. See FEMA guidelines for further specifications. Basements of residential structures located in the SFHA must be elevated above the BFE. In Arizona, the minimum elevation above the BFE is one (1) foot. The NFIP regulations allow nonresidential buildings (commercial structures, garages, warehouses, etc.) the option to flood-proof rather than elevate as a means of protection from the base flood. Non-residential structures can be flood-proofed to one (1) foot above the BFE instead of being elevated. Modular buildings, manufactured homes, and mobile homes must have the bottom of the structure (bottom of lowest beam and utilities) raised, as a minimum, to or above the BFE regardless of its use. Detached garages, barns, and storage sheds are some examples of buildings that may not have to be elevated or dry flood-proofed if openings are installed to allow floodwaters to enter or exit a structure and meet all other wet flood-proofing requirements. Wet flood-proofing requires the use of flood-resistant materials below the BFE and elevating items subject to flood damage above the BFE. Flood-proofed structures must comply with appropriate sections of the NFIP regulation 60.3.

All new construction and substantial improvements shall be constructed with electrical, HVAC, plumbing, and other service facilities that are designed and/or located so as to prevent water from entering or accumulating within the components during conditions of flooding. Mechanical and electrical equipment must be installed at or above the BFE as a minimum. Septic tanks are not allowed within floodways, but are allowed within the SFHA as long as ADEQ requirements are met. All below ground tanks within the SFHA must be anchored against flotation. Above ground tanks are considered structures for floodplain management purposes.

[1] All new construction and substantial improvements of residential structures located within Zones A1-30, AE, and AH shall have the lowest floor, including the basement, elevated at or above the Base Flood Elevation.

The community must require new and replacement water supply systems within floodprone areas to be designed to minimize or eliminate infiltration of floodwaters into the systems. Wastewater disposal systems are allowed within floodplains and floodway fringe areas, but not within floodways. Septic tanks and leach fields shall be a minimum of 50 feet from a wash or channel, measured from the bank of the normal flow channel. The location and design of on-site waste disposal systems should be reviewed in order to prevent possible operational failure and potential contamination to the environment during flooding. The system should be protected from flood damage such that it can resume operation after the flood recedes. Manholes should be raised above the 100-year flood level or equipped with seals to prevent leakage. Pump stations should be located to allow access during a flood and designed to not release contamination. Automatic backflow valves should be installed to prevent sewage from backing up into buildings during a flood event.

Under no circumstances can filling or other construction activity be allowed within a floodway that may cause any rise in the water surface elevation above the designated floodway elevation.

An “Elevation Certificate” FEMA Form 81-31) must be completed for each structure constructed in the SFHA prior to the electrical clearance and final inspection for that structure. One copy of the “Elevation Certificate” is to be submitted to the General Building Safety Inspector on site. See Federal Code for a complete list of requirements. The original document must be submitted to the MCFCD, which has floodplain administrator responsibilities for Mohave County.

2.3.1.8 Floodplain Requirements for Alluvial Fans

In addition to or in place of the above requirements, the following is required for alluvial fan floodplains. The lowest floor of all new construction and substantial improvements of residential structures in an AO zone SFHA shall be elevated above the highest adjacent grade at least as high as the depth number specified in feet on the FIRM (at least two feet if no depth number is specified) in accordance with the Code of Federal Regulations (CFR) Section 60.3c(7). Non-residential structures may be flood-proofed in lieu of elevation. Adequate drainage paths must be provided in accordance with Section 60.3 c(11) of the CFR.

2.3.1.9 Post Construction Review

After the proposed improvements have been constructed, the owner/developer is required, within 6-months, to submit as-built/documents of record to FEMA and the community Floodplain Administrator along with a request for a letter of map revision or amendment as appropriate.

2.3.1.10 Fees

Fees will be assessed by FEMA for its review of proposed and "as-built" projects as outlined in NFIP regulations 44 CFR Ch. 1, Part 72. In addition, Mohave County levies a fee to help defray its cost for administering floodplain management in conformance with the NFIP.

2.3.1.11 Additional Information

FEMA publishes numerous documents to inform those within or adjacent to a SFHA. Those documents can be located using FEMA's contact address at the end of this section. The most recent version of the following documents are very useful to consult if a property is determined to be within a SFHA:

1. "*National Flood Insurance Program (Regulations for Floodplain Management and Flood Hazard Identification)*", Federal Emergency Management Agency, 44 CFR, Part 1 most current revision.

2. "*Guidelines and Specifications for Flood Hazard Mapping Partners*, Federal Emergency Management Agency, April, 2003.

3. "*Technical Bulletin 2-93, Flood-Resistant Materials Requirements for Buildings Located in Special Flood Hazard Areas in accordance with National Flood Insurance Program*", Federal Emergency Management Agency, April, 1993.

4. "*Technical Bulletin 3-93, Non-Residential Flood Proofing Requirements and Certification for Buildings Located in Special Flood Hazard Areas in Accordance with the National Flood Insurance Program*", Federal Emergency Management Agency, April, 1993.

5. "*Technical Bulletin 10-01, Ensuring That Structures Built on Fill In or Near Special Flood Hazard Areas Are Reasonably Safe From Flooding in Accordance with the National Flood Insurance Program*", Federal Emergency Management Agency, May, 2001.

Other publications about the NFIP can be found online at:

http://www.fema.gov/business/nfip/libfacts.shtm.

2.3.1.12 Contact Information

Department of Homeland Security
Emergency Response and Recovery Directorate
Federal Emergency Management Agency
National Flood Insurance Program
Region IX
Federal Insurance and Mitigation Division
1111 Broadway, Suite 1200
Oakland, CA 94607-4052
(510) 627-7260
web site: http://www.fema.gov

2.3.2 SECTION 404 PERMIT FOR WATERS OF THE UNITED STATES

The U.S. Army Corps of Engineers (USACE) has been involved in regulating certain activities in the nation's waterways since the 1890's (Section 10 of the River and Harbors Act of 1899). Until 1972, the primary thrust of the USACE regulatory program was the protection of navigation. As a result of the environmental movement in the 1960's, several new environmental laws and judicial decisions (Clean Water Act of 1968; Marine Protection, Research, and Sanctuaries Act of 1972), the program evolved to water resource protection which focused on the environmental (archeological, biological and the ecological) aspects of both arid and aquatic environments. The program includes one that considers the full public interest by balancing the favorable impacts against the detrimental impacts. Therefore, Section 404 of the Federal Water Pollution Control Act Amendments of 1972 insures that the physical, biological, and chemical quality of our nation's water is protected from irresponsible and unregulated discharges of dredged or fill material that could permanently alter or destroy these valuable resources.

Section 404 of the Clean Water Act regulates the discharge of dredge and fill activities in waters of the US. Any person, firm, or agency (including federal, state, and local government agencies) planning to work in or place dredged or fill material in Waters of the United States, must first obtain a permit from the USACE. The regulatory area is designated "Waters of the United States" or jurisdictional waters". Waters of the United States includes essentially all surface waters such as all navigable waters and their tributaries, all interstate waters and their tributaries, all wetlands adjacent to these waters, and all impoundments of these waters. In Mohave County, ephemeral streams (washes) may be jurisdictional if they exhibit certain characteristics, such as the width of the wash, presence of hydraulic sorting, and the presence of riparian habitat. The regulations governing Waters of the United States (including wetlands) apply to both public and private property.

Determination of the presence and extent (if present) of jurisdictional waters should be undertaken during the early stages of project planning. A jurisdictional delineation establishes the USACE regulatory area. It is highly recommended that the inexperienced seek guidance from the USACE or other environmental professionals.

The ADOT Environmental Planning Group Section 404/401 Manual (ADOT, 2007) provides guidance on the preparation of jurisdictional delineations, Nationwide Permit preconstruction notification submittals, and Individual Permit applications. The manual can be found at:

http://www.azdot.gov/Highways/EPG/EPG_common/section_404.asp.

The manual is divided into 5 easy-to-follow steps beginning with jurisdictional delineations and ending with Section 401 certification. Although focused on ADOT highway projects, this is a must resource for all practitioners involved with Section 404 permitting. The five basic steps of the Section 404/401 process described in ADOT (2007), and some of the key terms and documents pertaining to each step, are summarized in Table 2.2. The following information is not inclusive. Refer to ADOT (2007) for more detail.

Table 2.2 Five basic steps of the Section 404/401 process

Step	Description
1	**Are waters of the US present?** Complete a jurisdictional delineation: Key components: aerial photograph, topographic map, ground photographs, text and/or table describing site conditions.
2	**Would the activity result in a discharge of dredged or fill material into waters of the US?** **Is the activity exempt from regulation?**
3	**Would the impact on waters of the US be a temporary disturbance or a permanent loss?** **What quantity of impact on waters of the US would result from the activity?** Most commonly quantified is surface area/acreage, cubic yards, or linear feet. **What mitigation measures would be feasible and prudent?** Types: avoidance, minimization, rectification, reduction, compensation mitigation.
4	**Can the activity be authorized under a Nationwide Permit or is an Individual Permit required?** **Nationwide Permit** **What type of permit is required?** Select from 43 Nationwide Permits **Is preconstruction notification required?** No—non-notifying Yes: Key components: topographic map, ENG Form 4345, plan sheets depicting impacts, General Conditions Compliance, mitigation plan if applicable. **Individual Permit** Key components: topographic map, ENG Form 4345, plan sheets depicting impacts, Section 404(b)(1) draft decision document, mitigation plan if applicable, copy of Section 401 Individual Certification application.
5	**Is the activity certified under Section 401?** Certified Conditionally certified Individual certification required: Key components: vary depending on agency/tribe with oversight.

2.3.2.2 Permits

Physical work in a watercourse or wetland may require a USACE permit. Common activities that require Section 404 and Section 401 compliance, if conducted within waters of the US, include but are not limited to:

- Culvert installations and extensions.
- Bridge scour countermeasures and bridge abutment and pier construction.
- Channel bank protection.
- Wash realignment and channelization.
- Roadway and utility crossings.
- Geotechnical borings.
- The removal of sediment buildup from culverts if removal is through blading or bulldozing.

The program provides for the consideration of all concerns of the public, such as environmental, social, and economic aspects, in the USACE 404 permit decision-making process. As part of this responsibility, the USACE Section 404 permit program extends its jurisdiction to areas that were not regulated prior to the Clean Water Act.

Capital improvement projects undertaken on behalf of and paid for by Mohave County must coordinate their efforts with their client department[2] and/or the MCFCD prior to contacting the USACE. Joint ventures between the MCFCD or Mohave County and private entities must coordinate with the appropriate division prior to any inquiries or submittals to the USACE. Close coordination between client personnel (Mohave County staff and/or consultants, or the private entity project owner) and design personnel is critical to ensure that Section 404 and Section 401 calculations and documentation are accurate. Typically, design personnel provide important activity scope and impact information to the client for inclusion in the Section 404 and Section 401 documentation.

In addition, client personnel must be provided the opportunity to review all Section 404 or Section 401 conditions and mitigation measures applicable to an activity in order to identify

[2] Consultants should contact their client department to determine the best means of communication.

construction constraints as early as possible. Conditions and mitigation measures are ultimately included in the environmental clearance and/or project specifications, as appropriate.

Should a permit be required, there are several options depending on the type of land disturbance activity. There are essentially two types of Section 404 permits: Nationwide Permits (NWP), which are basically general permits that can be used for projects with impacts that will not exceed certain defined thresholds, and Individual Permits, which must be utilized for all projects with impacts that exceed such thresholds. Notably, the type of permit that an entity chooses to pursue, and the manner in which the permitting process is handled, can have a direct bearing on whether the project succeeds or fails.

2.3.2.3 Individual Permits

Individual permits are issued following a full public interest review of an individual application for a USACE permit. A public notice is distributed primarily to adjacent property owners and all known interested persons. After evaluating all comments and information received, final decision on the application is made.

The permit decision is generally based on the outcome of a public interest balancing process where the environmental benefits of the project are balanced against the detriments. A permit will be granted unless the project is not found to be the least environmental damaging and practicable alternative, exhibiting avoidance and minimization of impacts to the natural resources. Public interest, economics, engineering and other factors can also play a part in the final decision.

An individual permit also requires a 401 Water Quality Certification from the State of Arizona. Application forms for individual permits are available from all USACE regulatory offices. Refer to Section for information on the 401 Water Quality Certification program.

2.3.2.4 Nationwide Permits

A NWP is a form of general permit that authorizes a category of specific activities that exhibit minimal impact to the environment. These permits are valid only if the conditions applicable to the permits are met. If the conditions cannot be met, a regional or individual permit may be required. Please note that the NWP program was revised per Federal Register (FR) Vol. 72, No. 47, March 12, 2002 and corrected by notice FR Vol. No. 88, May 8, 2007. Refer to http://www.usace.army.mil/CECW/Documents/cecwo/reg/nwp/nwp_2007_final.pdf and http://www.usace.army.mil/CECW/Documents/cecwo/reg/nwp/nwp2007_docs.pdf. Nationwide permits listed below may be modified to accommodate regional conditions. Contact the USACE office using the information provided at the end of this section to obtain the most current information on the NWP program changes. The reader should contact the USACE for a complete listing, permit details, and regional limitations placed upon nationwide permits. Some activities under nationwide permits require preconstruction notification submittals to the USACE prior to the carrying out of those activities. Notification requirements are described in General Condition 13, 65 FR 52 12818-12899. All nationwide permits must comply with the requirements of the particular nationwide permit, and meet the general conditions (27) required for each one, the 401 conditions (for water quality), and, if adopted, the Los Angeles District regional conditions. A list of the more pertinent, presently available, nationwide permits follows. Refer to http://www.usace.army.mil/CECW/Pages/nw_permits.aspx for more information regarding Nationwide Permits.

NWP 3: Maintenance. The repair, rehabilitation, or replacement of any previously authorized, currently serviceable, structure or fill, or of any currently serviceable structure or fill authorized by 33 CFR 330.3. Discharges of dredged or fill material, including excavation, into all Waters of the United States to remove accumulated sediments and debris in the vicinity of, and within, existing structures and the placement of new or additional rip rap to protect the structure.

NWP 6: Survey Activities. Survey activities including core sampling, seismic exploratory operations, plugging of seismic shot holes and other exploratory-type bore holes, soil survey and sampling, and historic resources surveys.

NWP 7: Outfall Structures. Activities related to construction of outfall structures and associated intake structures where the effluent from the outfall is authorized, conditionally authorized, or specifically exempted, or are otherwise in compliance with regulations issued under the National Pollutant Discharge Elimination System program (NPDES) (Section 402 of the Clean Water Act).

NWP 12: Utility Line Activities. The construction, maintenance, or repair of utility lines, including outfall and intake structures and the associated excavation, backfill, or bedding for the utility lines, in all Waters of the United States, provided there is no change in preconstruction contours.

NWP 13: Bank Stabilization Projects. Bank stabilization activities necessary for erosion prevention, provided the activity meets all of the following criteria:

a. No material is placed in excess of the minimum needed for erosion protection;
b. The activity is no more than 500 feet in length along the bank, unless this criterion is waived in writing by the district engineer;
c. The activity will not exceed an average of one cubic yard per running foot placed along the bank below the plane of the ordinary high water mark or the high tide line, unless this criterion is waived in writing by the district engineer;
d. The activity does not involve discharges of dredged or fill material into special aquatic sites, unless this criterion is waived in writing by the district engineer;
e. No material is of the type, or is placed in any location, or in any manner, to impair surface water flow into or out of any water of the United States;
f. No material is placed in a manner that will be eroded by normal or expected high flows (properly anchored trees and treetops may be used in low energy areas); and,
g. The activity is not a stream channelization activity.

NWP 14: Linear Transportation Projects. Activities required for the construction, expansion, modification, or improvement of linear transportation crossings (e.g., highways, railways, trail, and airport runways and taxiways) in waters of the United State subject to acreage limitations.

NWP 18: Minor Discharges. Minor discharges of dredged or fill material into all Waters of the United States subject to volume or acreage limitations.

NWP 19: Minor Dredging. Dredging of no more than 25 cubic yards below the plane of the ordinary high water mark or the mean high water mark from navigable Waters of the United States.

NWP 20: Oil Spill Cleanup. Activities required for the containment and cleanup of oil and hazardous substances which are subject to the National Oil and Hazardous Substances Pollution Contingency Plan (40 CFR Part 300) in accordance with certain state and federal requirements.

NWP 23: Approved Categorical Exclusions. Activities undertaken, assisted, authorized, regulated, funded, or financed, in whole or in part, by another Federal agency or department.

NWP 25: Structural Discharges. Discharges of material such as concrete, sand, rock, etc. into tightly sealed forms or cells where the material will be used as a structural member for standard pile supported structures, such as bridges, transmission line footings, and walkways.

NWP 29: Residential Developments. Discharges of dredged or fill material into non-tidal Waters of the United States, including non-tidal wetlands for the construction or expansion of a single-family home and attendant features (such as a garage, driveway, storage shed, and/or septic field) for an individual permittee.

NWP 31: Maintenance of Existing Flood Control Facilities. Discharges of dredged or fill material for the maintenance of existing flood control facilities, including debris basins, stormwater storage basins, and channels. The maintenance is limited to that approved in a maintenance baseline determination made by the District Engineer.

NPW-33: Temporary Construction, Access, and Dewatering. Temporary structures, work, and discharges, including cofferdams, necessary for construction activities or access fills or dewatering of construction sites, provided that the associated primary activity is authorized by the Corps of Engineers or the U.S. Coast Guard. This NWP also authorizes temporary structures, work, and discharges, including cofferdams, necessary for construction activities not otherwise subject to the Corps or U.S. Coast Guard permit requirements. Appropriate measures must be taken to maintain near normal downstream flows and to minimize flooding. Fill must consist of materials, and be placed in a manner, that will not be eroded by expected high flows.

NWP 38: Cleanup of Hazardous and Toxic Waste. Specific activities required to effect the containment, stabilization, or removal of hazardous or toxic waste materials that are performed, ordered, or sponsored by a government agency.

NWP 39: Commercial and Institutional Developments. Discharges of dredged or fill material into non-tidal Waters of the United States for the construction or expansion of residential, commercial, and institutional building foundations and building pads and attendant features that are necessary for the use and maintenance of the structures.

NWP 40: Agricultural Activities. Discharges of dredged or fill material into non-tidal Waters of the United States for the purpose of improving agricultural production and the construction of building pads for farm buildings. Authorized activities include the installation, placement, or construction of drainage tiles, ditches, or levees; mechanized land clearing; land leveling; the relocation of existing serviceable drainage ditches constructed in Waters of the United States; and similar activities.

NWP 41: Reshaping Existing Drainage Ditches. Discharges of dredged or fill material into non-tidal Waters of the United States to modify the cross-sectional configuration of currently serviceable drainage ditches constructed in these waters. The reshaping of the ditch cannot increase drainage capacity beyond the original design capacity or expand the area drained by the ditch as originally designed (i.e., the capacity of the ditch must be the same as originally designed and it cannot drain additional wetlands or other Waters of the United States).

NWP 42: Recreational Facilities. Discharges of dredged or fill material into non-tidal Waters of the United States, excluding non-tidal wetlands adjacent to tidal waters, for the construction or expansion of recreational facilities.

NWP 43: Stormwater Management Facilities. Discharges of dredged or fill material into non-tidal Waters of the United States for the construction and maintenance of stormwater management facilities, including activities for the excavation of stormwater ponds/facilities, detention basins, and retention basins; the installation and maintenance of water control structures, outfall structures and emergency spillways; and the maintenance dredging of existing stormwater management ponds/facilities and detention and retention basins.

NWP 44: Mining Activities. Discharges of dredged or fill material into: 1) Isolated waters, streams where the annual average flow is 1 cubic foot per second or less, and 2) non-tidal wetlands adjacent to headwater streams, for aggregate mining and other mining activities subject to certain limitations.

To apply for a nationwide permit, an application must be completed. USACE application forms for the permits are available from the local USACE regulatory offices (see contact information below).

NWP 45: Repair of Uplands Damaged by Discrete Events. This NWP authorizes discharges of dredged or fill material, including dredging or excavation, into all waters of the United States for activities associated with the restoration of upland areas damaged by storms, floods, or other discrete events. This NWP authorizes bank stabilization to protect the restored uplands. The restoration of the damaged areas, including any bank stabilization, must not exceed the contours, or ordinary high water mark, that existed before the damage occurred.

NWP-46: Discharge in Ditches. Discharges of dredged or fill material into non-tidal ditches that are: (1) Constructed in uplands, (2) receive water from an area determined to be a water of the United States prior to the construction of the ditch, (3) divert water to an area determined to be a water of the United States prior to the construction of the ditch, and (4) are determined to be waters of the United States. The discharge must not cause the loss of greater than one acre of waters of the United States.

2.3.2.5 Regional Permits

Regional permits are issued by the USACE District Engineer for a general category of activities when:

1. the activities are similar in nature and cause minimal environmental impact (both individually and cumulatively), and
2. the regional permit reduces duplication of regulatory control by State and Federal agencies.

Contact the USACE District Regulatory office in your area for information regarding regional permits.

2.3.2.6 Web Site Links

Additional web site links include:

404 Permit Application: http://www.spl.usace.army.mil/regulatory/eng4345.pdf

Guide to Watercourse Permitting: http://www.spl.usace.army.mil/regulatory/water.html

CWA Section 401 Certification: http://www.spl.usace.army.mil/regulatory/401.html

Minimum Requirements for a Complete Application:
http://www.spl.usace.army.mil/regulatory/list.html

2.3.2.7 Contact Information

U.S. Army Corps of Engineers
Los Angeles District, Regulatory Branch
3636 North Central Avenue, Suite 900
Phoenix, AZ 85012-1936 (602) 640-5385
web site: http://www.spl.usace.army.mil

2.3.3 STORMWATER QUALITY

Section 405 of the Water Quality Act of 1987 added section 402(p) of the Clean Water Act (CWA) which required the U.S. Environmental Protection Agency (EPA) to develop a phased approach to regulate stormwater discharges under the National Pollutant Discharge Elimination System (NPDES) program. Section 402(p) of the CWA states that stormwater discharges associated with construction activities to waters of the United States must be authorized by an NPDES permit

2.3.3.1 Phase I

EPA published final regulations on the first phase of the stormwater program on November 16, 1990. These rules established permit application requirements for "stormwater discharges associated with industrial activity" (including construction activities) and for discharges from municipal separate storm sewer systems located in municipalities with a population of 100,000 or more. EPA defined the term "stormwater discharge associated with industrial activity" in a comprehensive manner to cover a wide variety of facilities. Construction activities (including clearing, grading and excavation activities) that disturb at least five acres of land (including smaller areas that are part of a larger common plan of development or sale) are defined as an "industrial activity" per 40 CFR 122.26(b)(14)(x).

EPA issued the first round of the Phase I construction general permit in September 1992. The Phase I permit was commonly referred to as the Baseline Construction General Permit and was administered by EPA as the NPDES Permitting Authority in Arizona.

2.3.3.2 Phase II

The regulation titled "National Pollutant Discharge Elimination System - Regulations for Revision of the Water Pollution Control Program Addressing Storm Water Discharges" (64 FR 68722) was published by EPA on December 8, 1999. This regulation, which is considered Phase II of the stormwater program, expanded the existing NPDES stormwater program to address discharges from small construction activities, defined in 40 CFR 122.26(b)(15)(i) as construction activities including clearing, grading, and excavating that result in land disturbance of equal to or greater than one acre and less than five acres. Small construction activity also includes the disturbance of less than one acre of land area if it is part of a larger common plan of development or sale. Phase II of the stormwater program also added small municipal separate storm sewer systems from any other municipalities located wholly or partially in urbanized areas if they were not already covered by Phase I of the stormwater program.

The Stormwater Phase II Rule automatically designates these small sites; however, this rule allows for the exclusion of certain sources from the program based on a demonstration of the lack of impact on water quality, as well as the inclusion of others based on a higher likelihood of localized adverse impact on water quality. Exclusion from the requirement to obtain a permit is available through a waiver to operators of small construction activity who qualify and is effectively limited to those doing short-term projects during the dry times of the year.

The second-round construction general permit, issued February 17, 1998, was administered by EPA as the NPDES Permitting Authority in Arizona.

2.3.3.3 Delegation

On December 5, 2002, ADEQ received authorization from the EPA to implement the Arizona Pollutant Discharge Elimination System (AZPDES) program in Arizona. In Arizona except for Indian Community Lands, the NPDES program is administered as the AZPDES program. This authority was challenged, and on June 25, 2007, the U.S. Supreme Court upheld the authority of ADEQ to operate the NPDES Permit Program on the state level. Refer to Section 2.4.2 for more information.

2.4 STATE OF ARIZONA

2.4.1 FLOODPLAIN MANAGEMENT

The State of Arizona has set minimum floodplain management requirements for both areas that are not studied and areas identified by FEMA as a SFHA. The Arizona Department of Water Resources (ADWR) is responsible for floodplain management statewide and for administering the NFIP at the state level. ADWR has developed a series of State Standards to aid in floodplain management for the FEMA and non-FEMA studied areas of the state. Each State Standard has a companion document called the State Standard Attachment (SSA). The SSA is the technical document that provides the methodology and examples of how to apply the standard.

The following is a list of State Standards (SS) currently available from ADWR. It is the responsibility of each person to ensure that they have the most current version or new State Standard available. ADWR does update existing State Standards periodically and is developing new State Standards where a need exists. These standards are available online at: http://www.azwater.gov/dwr/Content/Find_by_Program/Dam_Safety_and_Flood_Mitigation/default.htm.

SS 1-97 - Requirement for Flood Study Technical Documentation
SS 2-96 - Requirement for Floodplain and Floodway Delineation in Riverine Environments

SS 3-94 - State Standard for Supercritical Flow (Floodway Modeling)

SS 4-95 - State Standard for Identification of and Development within Sheet Flow Areas

SS 5-96 - State Standard for Watercourse System Sediment Balance

SS 6-05 - State Standard for Development of Individual Residential Lots within Floodprone Areas

SS 7-98 - State Standard for Watercourse Bank Stabilization

SS 8-99 - State Standard for Retention/Detention

SS 9-02 - State Standard for Floodplain Hydraulic Modeling

SS10-07 - State Standard for Hydrologic Modeling Guidelines

In addition, ADWR provides training documents in the appropriate use of the State Standards. The *Floodplain Issues in Transportation Design* training document is very appropriate for use in conjunction with this manual. It can be found on the same web page as the State Standards listed above.

2.4.1.1 Contact Information

State of Arizona
Department of Water Resources
Flood Mitigation Section
3550 N. Central Avenue
Phoenix, AZ 85012
(602) 771-8500
web site: http://www.azwater.gov/dwr

2.4.2 STORMWATER QUALITY

2.4.2.1 Delegation

On December 5, 2002, ADEQ received authorization to implement the AZPDES program in Arizona. In Arizona except for Indian Community Lands, the NPDES program is administered as the AZPDES program. This authority was challenged, and on June 25, 2007, the U.S. Supreme Court upheld the authority of ADEQ to operate the NPDES Permit Program on the state level.

The AZPDES Construction General Permit (AZG2003-001) was issued for a five-year term by ADEQ in February 2003 and expired February 28, 2008. If a new permit was not re-issued by this date, those who had coverage under the 2003 permit would have an administrative extension to continue operation under that permit. However, operators of new construction sites would not be able to obtain coverage for stormwater discharges under a general permit until a new permit is issued. The new Construction General Permit (CGP) No. AZG2008-001 was issued on February 28th, 2008. It replaces the expired CGP No. AZG2003-001. Copies of the new permit and other related information can be obtained at: http://www.azdeq.gov/function/forms/appswater.html#cgp.

An AZPDES permit is required for any point source discharge of pollutants to a Water of the United States. Because stormwater runoff can transport pollutants to either municipal storm sewer systems or to Waters of the United States, permits are required for those discharges. In addition to stormwater permits, there are also NPDES/AZPDES permits required for the discharge of processed wastewater and the land application of sludge. The application process for both general permits is similar.

2.4.2.2 Permits

ADEQ issues individual and general AZPDES permits. An individual permit is tailored for a specific facility based on an individual application. ADEQ develops the permit based on this information and incorporates technology-based requirements, water quality standards and other conditions appropriate to the facility. The permit is then issued for a specified period of time not to exceed five years.

A general permit is developed and issued to cover multiple facilities within a specific category, industry or area. General permits offer a cost-effective and efficient option for agencies to cover a large number of facilities with elements in common under one permit. In addition, the permittee is ensured consistency in permit conditions for similar facilities. A general permit could be written to include all facilities within a common geographic area that:

1. Involve the same or substantially similar types of operations.
2. Discharge the same types of wastes.
3. Require the same effluent limitations or operating conditions.
4. Require the same or similar monitoring requirements.

Most stormwater discharges are permitted under various general permits. However, an individual permit is required when the general permit requirements do not accurately represent the activity at a facility/municipality and a permit is customized to the site/for the permittee.

An individual permit may be necessary if the Limitations of Coverage section of a general permit does not allow the facility's discharge to be covered within the general permit. It is the responsibility of every applicant to determine if any of the Limitations of Coverage apply to the facility seeking a general permit.

2.4.2.3 Construction Activities

Stormwater discharges generated during construction activities can cause an array of physical, chemical and biological water quality impacts. Specifically, the biological, chemical and physical integrity of the waters may become severely compromised. Water quality impairment results, in part, because a number of pollutants are preferentially absorbed onto mineral or organic particles found in fine sediment. The interconnected process of erosion (detachment of the soil particles), sediment transport and delivery is the primary pathway for introducing key pollutants such as nutrients (particularly phosphorus), metals, and organic compounds into aquatic systems.

Stormwater runoff from construction sites can include pollutants other than sediment such as phosphorous and nitrogen, pesticides, petroleum derivatives, construction chemicals and solid wastes that may become mobilized when land surfaces are disturbed. Generally, properly implemented and enforced construction site ordinances effectively reduce these pollutants.

The new Construction General Permit (CGP) No. AZG2008-001 was issued on February 28th, 2008. It replaces the expired CGP No. AZG2003-001. Copies of the new permit and other related information can be obtained at: http://www.azdeq.gov/function/forms/appswater.html#cgp. The final permit is significantly different from the first draft released in 2007.

Construction General Permit Coverage

This general permit authorizes discharges of stormwater associated with construction activity provided the operator complies with all the requirements of the general permit and submits a Notice of Intent (NOI) in accordance with the general permit.

Stormwater associated with large construction activity refers to the disturbance of five or more acres, as well as the disturbance of less than 5 acres of total land area that is a part of a larger common plan of development or sale if the larger common plan will ultimately disturb five acres or more (40 CFR 122.26(b)(14)(x)).

Stormwater associated with small construction activity, as defined in 40 CFR 122.26(b)(15), refers to the disturbance of equal to or greater than 1 and less than 5 acres of land for construction, or the disturbance of less than 1 acre of total land area that is part of a larger common plan of development or sale if the larger common plan will ultimately disturb equal to or greater than 1 and less than five acres.

Permit Waivers

There are two waivers available for small construction activities. The first is where the construction site operator has determined that the rainfall erosivity factor (R) in the revised universal soil loss equation RUSLE) is less than 5. The second waiver is available where the operator certifies that stormwater controls are not needed based upon a total maximum daily load (TMDL). Currently Arizona TMDL's do not address this issue, but the permit includes the TMDL waiver as a potential future option.

How to Obtain Coverage

The operator of a construction site is responsible for obtaining coverage under an AZPDES permit and for preparing and maintaining a Storm Water Pollution Prevention Plan (SWPPP). The operator could be the owner, the developer, the general contractor or individual contractor. When responsibility for operational control is shared, all operators must apply. Thus, a single construction site may have a number of operators who may operate under a common or separate SWPPP. A SWPPP is a site-specific, written document that:

- Identifies potential sources of stormwater pollution at the construction site.
- Describes practices to reduce pollutants in stormwater discharges from the construction site. Reduction of pollutants is often achieved by controlling the volume of stormwater runoff (e.g., taking steps to allow stormwater to infiltrate into the soil).
- Identifies procedures the operator will implement to comply with the terms and conditions of a construction general permit.

Guidance for preparation of an SWPPP can be found at: http://cfpub.epa.gov/npdes/stormwater/swppp.cfm.

The Arizona 2008 Construction General Permit SWPPP Guidance Checklist can be found at: http://www.azdeq.gov/environ/water/permits/download/cswppp.pdf.

Submit a Notice of Intent (NOI) to the Arizona Department of Environmental Quality, Surface Water Section - Stormwater and General Permits Unit, 1110 W. Washington St., Phoenix, Arizona 85007 or fax the form to (602) 771-4528. The form can be found at:

http://www.azdeq.gov/environ/water/permits/download/constnoi.pdf

This form must be complete and accurate and signed by the appropriate signatory in order for coverage to be obtained. The form also serves as a commitment by the operator that there will be compliance with the permit conditions. ADEQ now offers the Stormwater SMART NOI system (http://az.gov/webapp/noi/main.do), which is a Web-based service to assist individuals in applying for construction stormwater discharge permits.

The operator must develop and implement a SWPPP that satisfies the conditions of the permit. If your site is located within 1/4 mile of a unique or impaired water, the SWPPP must be submitted with your NOI. ADEQ will notify you, within 32 business days after receiving the SWPPP, if the SWPPP needs revisions, or if permit coverage is granted or denied. In all other cases, you are not required to submit your SWPPP to the department for review, unless specifically required by ADEQ. However, the SWPPP must be on-site whenever construction activities are actively underway and you must continue to fully implement and maintain the SWPPP as construction activities progress. If the department does not issue the authorization certificate within seven days of receiving the NOI or otherwise notify the operator that the submitted NOI is deficient, the operator may commence construction activities without an authorization certificate; however, it is the operator's responsibility to verify the date the NOI was received by ADEQ prior to initiating construction activities. Whether or not ADEQ notifies the operator of a deficiency in the NOI, discharges are not authorized under this permit if the operator submits an incomplete or incorrect NOI.

Notice of Termination

After the construction project is complete and the project's disturbed area is stabilized to at least 70 percent of natural background density or responsibility for the project has been assumed by another operator, the permittee must submit a Notice of Termination (NOT) to end participation in the AZPDES stormwater program.

ADEQ's Construction General Permit

A draft of the 2008 Construction General Permit was published in the Arizona Administrative Register on Dec. 7, 2007. ADEQ accepted comments on the draft general permit until Jan. 25, 2008. Comments were received by construction industry representatives and municipalities.

This permit replaces the previous construction general permit that was issued for a five-year term by ADEQ in February 2003.

The CGP authorizes stormwater discharges from construction-related activities where those discharges have a potential to enter surface waters of the United States or a storm drain system. Note the AZPDES authorizing statute uses the term navigable waters, which is defined as equivalent to the waters of the United States. However, because the term navigable waters can be confusing to the general public (i.e., the definition of navigable waters also includes ephemeral washes, intermittent streams, playas, and wetlands, that may not be able to be traveled by conventional vessels), this permit generally references discharges to waters of the United States.

To obtain authorization for discharges of stormwater associated with construction activity, the operator must comply with all the requirements of the general permit and submit a Notice of Intent (NOI) in accordance with Part II of the general permit. Construction General Permit Forms can be found at:

http://www.azdeq.gov/function/forms/appswater.html#cgp

2.4.2.4 Industrial Activities

Activities that take place at industrial facilities, such as material handling and storage, are often exposed to stormwater. The runoff from these activities discharges industrial pollutants into nearby storm sewer systems and water bodies. This may adversely impact water quality. The initial focus of the NPDES permitting program was to regulate discharges of industrial process wastewater and municipal wastewater treatment plants. Most industrial facilities have permit coverage under a general permit because it is the most efficient permit option. General permits contain requirements for numerous types of industrial activities, allowing a facility operator to quickly obtain permit coverage. The Multi-Sector General Permit (MSGP) is the general permit currently unavailable to facility operators.

Multi-Sector General Permit

On September 29, 2008, EPA announced in the Federal Register (Vol. 73, No. 189) publication of the final 2008 MSGP (http://www.epa.gov/npdes/pubs/msgp2008_finalpermit.pdf). This permit replaces the 2000 MSGP, which expired on October 30, 2005. The 2008 MSGP provides coverage for industrial facilities located in 5 States, and in certain Indian Country lands, as well as at various Federal Facilities in other States (see Appendix C of the 2008 MSGP) where EPA still remains the NPDES permit authority. Operators are expected to develop and implement stormwater pollution prevention plans, best management practices and implement the appropriate sector-specific requirements as described in the 2008 MSGP.

The MSGP is designed for discharges of stormwater from certain industrial sites that are of a non-construction nature. The MSGP is one large permit divided into numerous separate sectors. Each sector represents a different type of activity and is dependent upon its standard industrial classification (SIC) code or narrative description. Review the information on Facilities Required to Apply for a Stormwater Permit (40 CFR 122.26(b)(14)) for applicable SIC codes and descriptions. Once a SIC code or narrative description is determined, review the document "What's My Sector?" at the following web link to determine which sector of the MSGP contains the specific permit requirements for a facility. Once the necessity for a permit is determined, a facility will be subject to the requirements of more than one sector if it has operations that can be described by other sectors.

http://www.azdeq.gov/function/forms/appswater.html#ms4

Application for this general permit is achieved by the completion of a simple one-page form called a notice of intent (NOI). The NOI is a promise by the applicant that there will be compliance with the permit conditions. However, before the NOI is submitted, a SWPPP must be prepared. The MSGP details the requirements EPA considers necessary for each sector to produce an acceptable SWPPP. There is no requirement to submit the SWPPP to ADEQ, but ADEQ, EPA or Mohave County can request that the SWPPP be available for review. Once the SWPPP is prepared and the NOI submitted, there is a waiting period of two days. If ADEQ does not contact the applicant within the waiting period, the applicant may assume permit coverage has been granted. After the two-day waiting period the permittee may implement the SWPPP and begin activities. ADEQ will confirm permit coverage with the permittee by a letter containing the discharge authorization number. If the NOI is submitted with missing, nonconforming or incorrect information, ADEQ will inform the applicant of the inadequacies and request additional information. Permit authorization to discharge stormwater is only possible after the submittal of a complete and accurate NOI. The permittee submits a notice of termination to end participation in the NPDES stormwater program. Failure to develop specific

Best Management Practices (BMP) or to implement these BMPs identified in the SWPPP may subject the Permittee(s) to fines of up to $25,000 per day per violation.

Permit information and forms may be obtained from the agencies provided in Section 2.4.2.6.

2.4.2.5 Other Permits

For information on other permits available through ADEQ, check out ADEQ's website at: http://www.azdeq.gov/environ/water/permits/azpdes.html. The following is ADEQ's summary of the DeMinimus Discharge Permit and the Concentrated Animal Feeding Operations program.

De Minimus Discharge Permit

ADEQ issued the first AZPDES De Minimus General Permit (DGP) No. AZG2004-001 on March 7, 2004. The permit allows for the discharge of pollutants associated with potable and reclaimed water systems, subterranean dewatering, well development, aquifer testing, hydrostatic testing of specific pipelines, residential cooling water, charitable car washes, building and street washing, and de-chlorinated swimming pool water. The permit also allows ADEQ to review and approve other case-by-case short-term and/or low volume discharges that are considered De Minimus. By definition (DGP, Part VII), De Minimus discharges contain relatively low levels of pollutants, are of limited flow and/or frequency, and shall not last for more than 30 days unless approved in advance by ADEQ.

The DGP authorizes discharges where they have potential to enter a water of the U.S. Note: the AZPDES authorizing statute uses the term "navigable waters," which is defined as equivalent to the waters of the U.S. However, because the term 'navigable waters' can be confusing to the general public (i.e., the definition of 'navigable waters' also includes ephemeral washes, intermittent streams, playas, and wetlands, that may not be able to be traveled by conventional vessels), this permit references discharges to waters of the U.S.

Authorization under this permit will require the owner or operator of the discharge facility to implement various BMPs and conduct discharge monitoring based on the type of discharge activity and the type of receiving water. For further information on this permitting program, visit ADEQ's website at:

http://www.azdeq.gov/environ/water/permits/gen.html#demi.

Concentrated Animal Feed Operations

ADEQ revised the AZPDES program rules (18 A.A.C. 9, Article 9) to conform with the updated federal regulations for Concentrated Animal Feeding Operations (CAFOs). The rule revisions became effective on Feb. 2, 2004. Under the new rule all CAFOs are required to apply for a permit, submit an annual report and develop and follow a plan for handling manure and wastewater. In addition, the rule moves efforts to protect the environment forward by placing controls on land application of manure and wastewater, covering all major animal agriculture sectors, and increasing public access to information through CAFO annual reports. The rule also eliminates current permitting exemptions and expands coverage over types of animals in three important ways: the rule eliminates the exemption that excuses CAFOs from applying for permits if they only discharge during large storms; second, the rule eliminates the exemption for operations that raise chickens with dry manure handling systems; and third, the rule extends coverage to immature swine and immature dairy cows. ADEQ issued the AZG2004-002 general permit on April 16, 2004. For further information on this permitting program, visit ADEQ's website at: http://www.azdeq.gov/environ/water/permits/cafo.html.

Application or approval of any permit from ADEQ does not grant approval for any other permits required by other federal, state, or local entities including the Mohave County Flood Control District (i.e. the granting of a DeMinimus Discharge permit does not give anyone the right to discharge into a Mohave County structure without Mohave County's prior approval/permit. A Mohave County Public Works right-of-way permit is still required).

2.4.2.6 Contact Information

Arizona Department of Environmental Quality
1110 W. Washington Street
Phoenix, AZ 85007
(602) 771-4449
web site: http://www.azdeq.gov

2.4.3 SECTION 401 WATER QUALITY CERTIFICATION

2.4.3.1 General

Section 404 of the Clean Water Act establishes a permitting program to regulate excavation and the discharge of dredged and fill material into waters of the of the United States, including wetlands. Examples of activities that might be regulated under this program include:

- stream crossings,

- dam construction and flow regulation,
- water diversion for canals, irrigation systems, and stock tanks,
- streambed modification and stabilization, and
- building subdivisions, master planned communities, highways and airports.

The EPA and the USACE jointly administer the program. In addition, the U.S. Fish & Wildlife, the National Marine Fisheries Service and State resources agencies (e.g., Department of Environmental Quality, Game and Fish Department, Water Resources) have important advisory roles.

While the Corps issues the permit, Section 401(a) of the CWA requires the state to provide certification, including permit conditions that the draft permit is in compliance with effluent limits, the State's water quality standards and any other appropriate requirements of state law.

The basic premise of the program is that no discharge of dredged or fill material can be permitted if a practicable alternative exists that is less damaging to the aquatic environment or if the nation's waters would be significantly degraded. In applying for a permit, the applicant must show that they have:

1. taken steps to avoid wetland impacts where practicable,
2. minimized potential impacts to wetlands, and
3. provided compensation for any remaining, unavoidable impacts through activities to restore or create wetlands.

The program is jointly administered between the US Army Corps of Engineers and the EPA. The Corps' duties include:

1. administering the day-to-day program, including individual permit decisions and jurisdictional determinations,
2. developing policy and guidance, and
3. enforcing Section 404 provisions.

The EPA is responsible for:

1. Developing and interpreting environmental criteria used in evaluating permit applications,
2. Determining the scope of geographic jurisdiction.
3. Identifying activities that are exempt.
4. Review/comments on individual permit applications.

5. Has the authority to veto the Corps' permit decision.
6. Elevating specific cases.
7. Enforcing Section 404 provisions.

The Corps has a number of authorization mechanisms including permits, letters of permission, and regional or state specific permissions.

An individual permit is required for projects that have potentially significant impacts. Individual permits require an application form describing the proposed activity be submitted to the Corps. Once the application is complete, the Corps issues a public notice containing the information needed to evaluate the likely impact of the activity. Notice is sent to all interested parties including adjacent property owners, government agencies and others who have requested notice. A hearing may be requested for cause.

However, for discharges that have only minimal adverse effects, the Corps has developed general permits that can be issued on a nationwide, regional or state basis for particular types of activities (e.g., minor road crossings, utility line backfill, flood control projects). General permits are developed and require the same public notice requirements and opportunity for public hearing. Once issued, the general permit may be modified or revoked if the activities are found to have any adverse impacts. General permits are issued for a specified time period, usually five years.

Currently there are 40 NWPs. These NWPs have been CWA 401 certified by ADEQ and many contain State specific conditions in addition to the Corps requirements. Refer to Section 2.3.2 for more information.

ADEQ has authority under section 401 of the CWA to grant, deny or waive water quality certification for both individual and nationwide permits. The Corps cannot issue a permit, individual or general, where ADEQ hasn't approved or waived certification or where ADEQ has denied certification. The application form for ADEQ Water Quality Certification can be found at: http://www.azdeq.gov/environ/water/permits/download/401app2.pdf.

2.4.3.2 Contact Information

Arizona Department of Environmental Quality
Water Quality, Section 401
1110 W. Washington Street
Phoenix, AZ 85007
(602) 771-4502
web site: http://www.azdeq.gov

2.4.4 DAMS

All dams in the state, except those owned or operated by an agency or instrumentality of the federal government, are under the jurisdiction of the Arizona Department of Water Resources (ADWR). A dam is any artificial barrier that impounds or diverts water above the natural ground surface. A detention basin or retention basin that impounds stormwater above the natural ground surface may be considered as being a dam under the authority of ADWR. The following do not fall under the authority of ADWR.

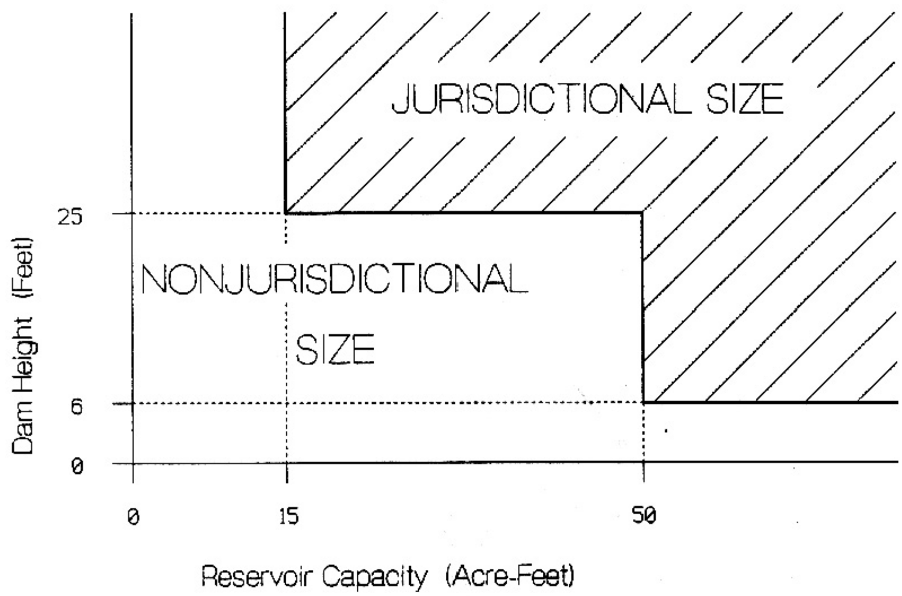
Any artificial barrier:

1. Less than 6 feet in height, regardless of storage capacity.
2. Fifteen acre-feet or less of storage capacity, regardless of height.
3. Between 6 and 25 feet in height, with a storage capacity less than 50 acre-feet.

Any impoundment or diversion structure that exceeds the criteria above will require a permit from ADWR. Individuals having questions should contact the Dam Safety Section of ADWR.

The distinction between a jurisdictional and non-jurisdictional dam is shown on Figure 2.1. A jurisdictional dam is either 25 or more feet in height or has capacity to store more than 50 acre-feet. Height is the vertical distance from the lowest point on the downstream toe (at natural ground) to the emergency spillway crest. Capacity is the maximum storage that can be impounded when there is no discharge of water.

Figure 2.1 ADWR Jurisdictional Dam Chart



2.4.4.2 Permits

A permit is required for all new dams or the repair, alteration or removal of an existing dam. Application forms are available from ADWR. An administrative review fee is required by ADWR.

2.4.4.3 Contact Information

State of Arizona
Department of Water Resources
Dam Safety Section
3550 N. Central Avenue
Phoenix, AZ 85004
(602) 771-8500
web site: http://www.azwater.gov/dwr

2.4.5 DRYWELL REGISTRATION

A person who owns an existing drywell that is or has been used for stormwater disposal shall register the drywell with the ADEQ. A drywell is a bored, drilled, or driven shaft or hole whose depth is greater than its width and is designed and constructed specifically for the disposal of stormwater. Drywells must be registered by completing a form from ADEQ, and submitting a registration fee for each drywell.

2.4.5.1 Permits

Drywells are regulated by Arizona Revised Statute (A.R.S.) § 49-241 and § 49-331 through 336, and Aquifer Protection Permit statutes and rules. Drywells that drain areas where hazardous substances are used, stored, loaded, or treated are subject to the General Permit or full Aquifer Protection Permit (see Section 2.4.6). Specific rules regarding dry wells are found in R-18-9-102-A and R18-9-A301. Program guidance documents are available from ADEQ, and should be followed for dry well construction, maintenance, siting, investigation, decommissioning, and closure. Registration is generally not required for dry wells used in conjunction with golf course maintenance, and they are exempted from regulation under the dry well program. However, vadose zone injection wells (including dry wells) that receive stormwater mixed with reclaimed wastewater or groundwater from manmade bodies of water associated with golf courses, parks, and residential areas must be registered. In this situation, a general permit is issued by statute in lieu of an individual permit, provided that six criteria, including registration, are met (A.R.S. § 49 - 245.02).

Dry well registration and permit information and forms may be obtained from ADEQ at the location provided below.

2.4.5.2 Contact Information

Arizona Department of Environmental Quality
1110 W. Washington Street
Phoenix, AZ 85007
(602) 771-2300
web site: http://www.azdeq.gov

2.4.6 AQUIFER PROTECTION PERMIT

An individual will need to obtain an APP if they own or operate a dry well that discharges a pollutant either directly to an aquifer or to the land surface or the vadose zone in such a manner that there is a reasonable probability that a pollutant will reach an aquifer. ADEQ may provide an "APP Determination of Applicability Form" for dry wells in areas where hazardous substances are used, stored, loaded, or treated. Dry wells that are used solely for the disposal of stormwater runoff do not require an Aquifer Protection Permit; however, dry well registration is still a requirement.

2.4.6.1 Permits

The following APP Permits are available:

Individual Permits

Individual permits are issued for a term not to exceed the operational lifetime of the facility. Approval of individual permits can take, on average, from 6 months to 2 + years. Processing time is approximately 6 months; however, incomplete applications often result in delays.

Area-Wide Permits

Area-wide permits may be issued in lieu of an individual permit to cover facilities under common ownership in a contiguous geographic area. Discharge reduction in the pollutant management area and the demonstration that aquifer water quality standards will not be violated or further degraded can be evaluated collectively for existing facilities. This type of permit is most applicable to large mining and industrial sites.

General Permits

There are currently 15 different types of general permits. These are issued by rule or statute, and the facility is automatically permitted, provided that certain conditions are adhered to. A separate permit document is not required to operate under these conditions and no fee is required.

Information regarding APP's is available from ADEQ at the location provided below.

2.4.6.2 Contact Information

Arizona Department of Environmental Quality
1110 W. Washington Street
Phoenix, AZ 85007
602) 771-2300
web site: http://www.azdeq.gov

2.5 LOCAL

2.5.1 GENERAL

In addition to the Federal and state regulations discussed in Sections 2.3 and 2.4, engineers responsible for drainage design must conform to Mohave County and other local regulations that may affect their project including local acts, codes, laws, regulations, ordinances, standards and policies. Sections 2.5.2 through 2.5.5 list the COUNTY/DISTRICT regulations that apply, and contain hyperlinks to the sites on the Internet where each document can be obtained. The COUNTY/DISTRICT stormwater management program, which is in the development process, is discussed in Chapter 6. The following are the Mohave County agencies that may be contacted to obtain assistance with application of these regulations.

Mohave County Environmental Health Division
700 W. Beale Street
Kingman, AZ 86401
Telephone: (928) 753-0743
web site: http://www.co.mohave.az.us/ContentPage.aspx?id=127&cid=340

Mohave County Flood Control District
Physical: 3675 E. Andy Devine Ave., Kingman, AZ 86401
Mailing: PO Box 7000, Kingman, AZ 86402-7000
(928) 757-0925
web site: http://www.co.mohave.az.us/FloodControl

Mohave County Public Works
Physical: 3675 E. Andy Devine Ave., Kingman, AZ 86401
Mailing: PO Box 7000, Kingman, AZ 86402-7000
(928) 757-0910
web site: http://www.co.mohave.az.us/ContentPage.aspx?id=128

2.5.2 MOHAVE COUNTY FLOOD CONTROL ORDINANCE - 2000

The DISTRICT flood control ordinance can be found at:

http://resource.co.mohave.az.us/File/Public%20Works/Flood%20Control/MohaveCountyFloodControlOrdinance.pdf

2.5.3 MOHAVE COUNTY LAND DIVISION REGULATIONS (2004)

The Mohave County land division regulations can be found at:

http://resource.co.mohave.az.us/File/PlanningAndZoning/Mohave_County_Land_Division_Regulations.pdf

2.5.4 MOHAVE COUNTY ZONING ORDINANCE (2005)

The Mohave County zoning ordinance can be found at:

http://resource.co.mohave.az.us/File/PlanningAndZoning/Mohave_County_Zoning_Ordinance.pdf

2.5.5 MOHAVE COUNTY ENGINEERING DESIGN STANDARDS

Mohave County Engineering Design Standards, Specifications and Details can be found at:

http://www.co.mohave.az.us/ContentPage.aspx?id=128&cid=235&page=3&rid=184

3 DRAINAGE PLANNING

3.1 INTRODUCTION

Storm water runoff facilities are an integral part of public infrastructure systems and should be planned as such. The drainage engineer must be included in the formulation of both site-specific and regional drainage plans and all development planning should be coordinated from the beginning with the drainage engineer. Drainage master plans need to be carefully prepared for all local and regional flood control and flood management projects, and this same general concept should be followed for land development of all sizes. A drainage master plan, in addition to providing a unified drainage plan, should be coordinated with planning for open space and recreation facilities, planning for transportation, and other development considerations. Drainage planning should not be done after all the other decisions have been made for the layout of a new subdivision, commercial or industrial area. It is this latter approach which creates drainage problems which are costly to correct. The benefits of good drainage planning include:

- Lower construction costs for drainage facilities
- Long-term maintenance costs are reduced
- Reduced flood damages for both residents and infrastructure.
- A better community

Natural drainage ways and street drainage patterns should be coordinated to achieve the policies and design criteria presented in the DDM. Supplemental or complimentary benefits and uses from drainage facilities should be considered, including passive and active recreational uses. Any effort made towards increasing local and community-wide benefits is appropriate and is encouraged.

Consideration of multiple uses and multiple benefits in drainage planning and engineering can minimize societal costs and increase benefits to the community. A way to maximize consideration of these multiple uses is by preparing practical drainage master plans so that the overall effort is coordinated with predetermined objectives.

Planning of drainage facilities should be based upon incorporating natural waterways, artificial channels, storm drains, and other drainage works into the development of a desirable and aesthetic community, rather than attempting to superimpose drainage works on a development after it is laid out. Channels and storm water storage facilities that are designed as a focal point of the community minimize misuse (e.g. dumping) and encourage proper maintenance.

Urban drainage should be considered on the basis of two design phases. The first is the preliminary phase where master drainage plans are developed. Work under the preliminary phase includes development of hydrology coupled with preliminary design, location, and verification of adequacy and viability of proposed drainage facilities. These facilities are planned in conjunction with land uses, street layouts and lot density and configuration. The second is the final design phase, which encompasses detailed engineering using the preliminary phase as the basis for the final design. The preliminary phase is a more global view, as discussed herein, and results in the conceptualization of an overall drainage solution. The final design phase is an extension of the first and it is here that the engineering details for the localized issues get worked out, including detailed hydraulic analyses and improvement plans.

The drainage system is the backbone of good urban planning in that a well planned system can reduce or eliminate the need for costly underground storm drains, and it can protect the urban area from extensive property damage and loss of life from flooding. This system is generally designed for the more severe and less frequent storm water runoff, such as the 100-year return period. It generally consists of open channels; however, large storm drains can be used. It must be remembered that the drainage system exists in a community whether or not it is planned and designed, and whether or not development is situated wisely with respect to it. Water will obey the law of gravity and will flow downhill to seek its lowest level whether development and people are in its way or not.

3.2 BENEFITS OF PLANNING

Good drainage planning is a complex process. Basic planning considerations that should be taken up early include planning for the drainage system, developing a grading concept, and planning for the environment. When planning a new subdivision for residential purposes, various drainage concepts should be evaluated before decisions are made as to street location and block layout. It is perhaps at this point of the development process where the greatest impact can be made as to the cost of drainage facilities. When flood hazards are involved, the planner should take these hazards into consideration in land planning to avoid unnecessary complications.

Benefits that can be derived from a good drainage plan include:

1. Reduced street maintenance costs.
2. Reduced street construction costs.
3. Improved movement of traffic.
4. Lower cost open space.
5. Lower cost park areas and more recreational opportunities.
6. Development of otherwise undevelopable land.
7. Opportunities for lower building construction cost.
8. Avoidance of flood damage claims and resultant litigation.
9. Avoidance of fines and fees levied for non-compliance with federal and state regulations, including NPDES Storm Water regulations.

3.3 TYPES OF DRAINAGE PLANS

Drainage plans can be divided into two types: regional and local. Regional plans are those prepared by a governmental agency for continuity on a regional basis. Local drainage plans for private land development or public projects that must conform to the regional plan, or stand on their own merits if a regional plan has not been developed. Both of these types typically have two component phases consisting of a preliminary drainage plan and a final drainage plan. Preliminary drainage plans deal with the broad assessment of existing drainage conditions and development of conceptual alternatives to accommodate drainage. Final drainage plans provide detailed analysis of preferred preliminary solutions, and/or documentation of engineered solutions and details to support the final design of a project. This section describes the two types of plans and their respective component phases.

3.3.1 REGIONAL DRAINAGE PLANS

On a watershed basis, regional master drainage plans are prepared to identify areas of existing flooding problems and present potential alternative solutions. Regional conceptual drainage plans provide detailed hydrologic and hydraulic computer models that are used to identify drainage and flooding problem areas. Solutions typically include an array of storm water conveyance and storage alternatives. These plans are generally an excellent source for hydrology and hydraulic information. The regional master drainage plan is typically a more detailed study providing more robust flood prevention designs and recommended drainage and floodplain management solutions. A watercourse master plan is similar to an regional master drainage plan, except that it has more of a focus on the management of a particular watercourse and associated flood hazard zones.

Developers should check with the MCFCD to determine if new floodplains, regulations, or projects have been identified or developed as part of a regional drainage plan for the area of concern. Construction projects that are defined as a part of a regional drainage plan typically have a Final Drainage Design Report for documenting the basis for the design. Regional drainage planning now also typically includes stormwater quality plans or plan components.

3.3.2 LOCAL DRAINAGE PLANS

Drainage plans are also prepared for land development and public projects. Here, the focus is to identify existing flooding conditions and to develop approaches to prevent the proposed development from exacerbating existing flooding conditions while protecting the proposed development. Within the unincorporated areas of Mohave County, drainage plans are required as described below. Drainage plans for developments or drainage improvements should also consider water quality components to their site development to prevent stormwater runoff water quality concerns.

3.3.2.1 Large Developments

Large developments, which require a Development Master Plan as described below, are typically considered to be greater than or equal to 640 acres in size. However, any significant development divided into phases may be considered a large development. Each development phase shall be designed to function as a standalone development and not dependant on a drainage feature constructed with a later phase. Stormwater quality concerns should be met on a phased basis. It would not be appropriate to address stormwater quality at the final phase of development. By phasing or implementing stormwater BMPs upfront water quality concerns will be met. The drainage plans required for large developments are:

Drainage Master Plan

A Drainage Master Plan is a conceptual plan that establishes the drainage approach and system to be used for the entire development. It also establishes how and when the various drainage system components will be constructed. This in turn has a significant impact on the size and orientation of lot and street layouts. Preparation of a Drainage Master Plan and the overall development plan is an iterative process between the developer, the developer's land planner and drainage engineer, and the regulating agency. The Drainage Master Plan will often significantly impact the definition of development units and phases.

The first step in preparing a Drainage Master Plan is studying the hydrology of the watersheds that contribute stormwater runoff to the master plan study area, and the hydrology of the onsite area.

The second step is definition of existing 100-year floodplains and base flood elevations for watercourses within the development where FEMA regulatory base flood elevations have not been established. This is to be done in accordance with Chapter 8. The definition of erosion hazards and an assessment of the drainage system sediment balance are to be done where necessary in conformance with Chapter 17.

The third step is definition and evaluation of drainage system alternatives, and recommendation of a drainage scheme. The key to preparing Drainage Master Plans for land developments is developing an approach to intercept offsite flow and identifying a workable means of conveying the flow through the project. The method for discharging to the downstream drainage network (whether natural or man-made) is established in a manner that returns the flow to its historical flow path without changing the pre-development flow characteristics. Drainage Master Plans for land developments also identify locations for stormwater storage facilities to accommodate on-site runoff, and identify a stormwater quality plan for the development. Offsite flows are not allowed to drain through the onsite conveyance or storage facilities. The above principles remain valid for conceptual drainage plans for all parcels regardless of size.

Drainage Master Plans are to be prepared in conformance with the report outline presented in Chapter 18 for the technical (Hydrology and Hydraulics) portions of the report document.

Preliminary Drainage Design Report

A Preliminary Drainage Design Report is a conceptual drainage plan for an individual unit or phase of the master planned development. It implements the drainage system recommended in the Drainage Master Plan to the specific unit in question. Adjustments are made to the Drainage Master Plan hydrology and hydraulics, if necessary, and alternatives for drainage facilities specific to the unit/phase are defined that meet the guidelines defined in the Drainage

Master Plan. The alternatives are analyzed and a recommended drainage system, including parameters for use during final design, is presented. These parameters include:

1. Design discharges and design storage volumes.
2. Definition of stormwater conveyance methods, including:
    a. channel locations, geometry, lining types and recommended slope ranges;
    b. storm drain locations, including preliminary sizes and material types;
    c. natural floodplains to be left undisturbed; and
    d. guidelines for use of street sections for stormwater conveyance.
3. Definition of methods to be used for erosion and scour protection.
4. Location, size, and recommended geometry of proposed stormwater storage basins.
5. Recommended stormwater quality design parameters.
6. Proof that the Drainage Master Plan recommendations for handling stormwater along the master-planned area boundaries are being met. This must include any needed addendum to the Drainage Master Plan for revised recommendations for future unit/phases.
7. Stormwater quality concerns must be addressed on a unit or phase basis as construction of the development occurs. In Mohave County, this is normally accomplished by providing the required stormwater storage facilities set forth in Chapter 15.

Preliminary Drainage Design Reports are to be prepared using the report outline presented in Chapter 18.

Final Drainage Design Report

A Final Drainage Design Report constitutes a final drainage plan component. It is the final documentation of the detailed drainage design shown on contract construction drawings for the development project. Refer to Section 3.3.3 for a description of a Final Drainage Design Report, which is common to both the government agency and private land development types of drainage plans.

3.3.2.2 Local Developments

Local developments are typically considered to be less than 640 acres in size. The drainage plans required for local developments are:

Preliminary Drainage Design Report

A Preliminary Drainage Design Report is a conceptual drainage plan for a private or agency project. For simple projects with minimal drainage considerations, the detail and length of the report is intended to be minimal. For larger projects with significant drainage considerations, the submittal requirements and level of detail may be a combination of the Drainage Master Plan and Preliminary Drainage Design Report for Large Developments as described above.

Final Drainage Design Report

A Final Drainage Design Report for Local Developments is the same as for Large Developments. The level of detail required is commensurate with the complexity of the drainage design.

3.3.3 FINAL DRAINAGE DESIGN REPORT

A Final Drainage Design Report constitutes a final drainage plan component and includes the final hydraulic analyses for detailed drainage designs presented on the construction plans. Final drainage construction drawings provide engineered solutions and details to implement the final drainage design of a project. The Final Drainage Design Report documents the supporting calculations and design assumptions the construction drawings are based on. The hydrology and hydraulics of the selected approach from the Drainage Master Plan and Preliminary Drainage Design Report is further refined and documented to apply to the specifics of the chosen drainage solution. The project may be a regional capital improvement project to alleviate existing flooding conditions or improvements resulting from land development. The design report documentation is to be prepared in accordance with Chapter 18.

3.4 INFORMATION FOR DRAINAGE PLANNING

There is a significant amount of existing information available to the hydrologist or drainage engineer that should be considered when undertaking a drainage plan. The following table highlights some of these.

Table 3.1 Types of available drainage information

Item	Source	Description
Flood Insurance Studies	FEMA, ADWR, MCFCD	Watershed peak discharges, floodwater levels, flood risk.
Regional conceptual and master drainage Plans	MCFCD & Municipalities	Watershed hydrographs and peak discharges, conceptual storage and conveyance solutions.

Table 3.1 Types of available drainage information

Item	Source	Description
Watercourse master plans WCMP)	MCFCD & Municipalities	Management of a particular watercourse and its associated flood and erosion hazards.
Studies & plans from existing flood control projects	MCFCD, USACE, USBR, NRCS	
Transportation Plans & Studies	ADOT, Mohave County, Municipalities	Corridor studies address existing and proposed drainage conditions. Plans depicting drainage improvements.
Land Use Zoning Maps	Municipality, Mohave County	Provides insight to future runoff characteristics. Zoning may limit type of drainage solution.
Soil Maps	NRCS & USFS	Identifies runoff characteristics and engineering limitations.
Aerial Photography	public & private	Identifies watershed and existing land-use characteristics.
Topographic Mapping	public & private	Used to determine watershed boundaries, slopes, and watercourse hydraulic characteristics.
ALTA Surveys	Mohave County Recorder's Office	Land ownership, boundary & utility easements (if available).
Drainage plans from adjacent developments	Municipalities/Mohave County/Land Developer/Home Owners Assoc.	Depicts existing or proposed conditions for adjacent properties that may affect the site under study.
Utility Plans	Utility companies	Depicts the location of underground and above ground utilities that may affect the location of drainage facilities and the routing of stormwater.

3.5 MASTER DRAINAGE PLANNING PROCESS

3.5.1 PLAN DEVELOPMENT

The master drainage planning process requires the collection and assimilation of information from most of the sources identified above. Consideration must be given to regulations, permitting, environmental impacts, ordinances, open space, zoning, regional hydrology, flood hazards, safety, and cost. As part of the initial layout design, the designer must consider and accommodate the future need of vehicular access for maintenance purposes. The preliminary design should minimize long-term maintenance requirements.

3.5.2 WATERS OF THE UNITED STATES (SECTION 404)

Waters of the United States, for the purposes of the Section 404 program (refer to Section 2.3), are drainage ways meeting certain criteria that define them by federal law as being under the jurisdiction of the U.S. Army Corps of Engineers. Waters of the United States are often referred to as jurisdictional waters. Construction activities that impact jurisdictional waters require a permit issued through the USACE. For most areas under study, jurisdictional waters exist. Therefore, drainage plans must consider the nuances of jurisdictional waters (See Chapter 2, and Section 5.3). The professional undertaking a drainage plan must have knowledge of 404 requirements to apply to the planning objective or have the jurisdictional waters delineated prior to delving too far into the drainage planning process. It is likely that the jurisdictional waters will have a significant impact on the overall drainage plan, remediation, and on-going maintenance activities.

3.5.3 WATERS OF THE UNITED STATES

Waters of the United States as defined by the EPA has a different context from that defined under Section 404. The EPA definition is included below for reference for those dealing with stormwater quality issues (refer to Chapter 6).

3.5.3.1 EPA Definition of Waters of the United States

1. All waters which are currently used, were used in the past, or may be susceptible to use in interstate or foreign commerce, including all waters which are subject to the ebb and flow of the tide;
2. All interstate waters, including interstate "wetlands;"
3. All other waters such as intrastate lakes, rivers, streams (including intermittent streams), mudflats, sand flats, "wetlands," sloughs, prairie potholes, wet meadows, playa lakes, or

natural ponds the use, degradation, or destruction of which would affect or could affect interstate or foreign commerce including any such waters:

a. Which are or could be used by interstate or foreign travelers for recreational or other purposes;
b. From which fish or shellfish are or could be taken and sold in interstate or foreign commerce; or
c. Which are used or could be used for industrial purposes by industries in interstate commerce.
d. All impoundments of waters otherwise defined as waters of the United States under this definition;
e. Tributaries of waters identified in paragraphs (a) through (d) of this definition;
f. The territorial sea; and
g. "Wetlands'' adjacent to waters (other than waters that are themselves wetlands) identified in paragraphs (a) through (f) of this definition.

3.5.4 REGULATIONS, POLICIES, AND STANDARDS

All drainage plans and construction drawings shall meet COUNTY/DISTRICT regulations. The policies and standards are intended to be an implementation guide for preparing drainage plans and drainage designs that are in conformance with the regulations. The time required for the review process is normally less, and review comments minimized, if the drainage plans are prepared in conformance with the policies and standards. Sometimes additional documentation may be required for submittal and review by the COUNTY/DISTRICT to prove conformance with the regulations. These policies and standards also establish the minimum guidelines for capital improvement projects, both public and private.

3.5.5 WATERCOURSE OPEN SPACE

The concept of combined flood control, environmental considerations, and recreational uses can be applied to drainage corridors (watercourses). Natural or semi-structural drainage/greenbelt corridors can be developed with landscaping, stormwater quality improvements, and multi-use trails incorporated into the drainage design to provide recreation opportunities. This concept can be applied to new drainage channels that are utilized for recreation uses, and existing open channels that currently do not provide recreation opportunities. The multi-use trails should be located above the channel banks to avoid impacting Waters of the United States (Section 404), to minimize effects of erosion, to minimize interaction with nuisance flows, and to minimize maintenance requirements. The COUNTY/DISTRICT stresses the establishment of natural or semi-structural drainage/greenbelt corridors. Utilizing natural/greenbelt corridors to accommodate stormwater is the DISTRICT preferred approach for several reasons, including:

1. Watercourses make excellent natural open spaces of high scenic quality due to their associated vegetation, wildlife and landforms.
2. Natural features such as topography, and natural processes such as erosion, have defined the land along natural watercourses as a drainage and stormwater runoff corridor.
3. Desert adapted vegetation is dependent on natural watercourses for water supply and seed disbursement and germination.
4. Many desert wildlife species are adapted to seek watercourse areas for food and shelter.
5. Impacts to watercourses have environmental consequences such as habitat loss, reduced flood conveyance, loss of a valuable landscape amenity, and reduced ground water recharge and impaired stormwater quality.
6. Impacts to watercourses have public safety consequences adjacent, upstream and downstream of the impact area.
7. Impacts to watercourses often have decreased property value implications as environmental impacts diminish abutting land value.
8. Designating open space along watercourses is often more cost effective for the developer due to the high risk of flooding in these corridors.

3.5.6 STORMWATER STORAGE

In the planning process, it is a COUNTY/DISTRICT goal that stormwater storage basins be combined where feasible with open space, parks, and trails to create focal points for the community instead of isolated tracts. These combined uses should be planned and designed to augment Mohave County parklands. The benefits of this approach are an enhanced sense of community and increased open space with landscape amenities. The COUNTY/DISTRICT encourages combined use of drainage and recreation facilities on both public and private lands. It is recommended that these drainage facilities be non-geometrically designed. Also, design of stormwater storage facilities should be coordinated with the COUNTY/DISTRICT to assure compliance with stormwater quality requirements.

3.5.6.1 Public Stormwater Storage Basins

Given the demand for organized sports fields such as soccer and ball fields, basins may serve multi-use purposes. It is recommended to avoid siting recreational facilities at the very bottom of stormwater storage basins. It is further recommended these basins be designed with tiers or gentle slopes to allow for the collection and conveyance of nuisance water around fields to allow for dry field areas under normal conditions.

The desired location for stormwater storage basins is adjacent to parks to increase the open space. Integrating non-geometric basins into park design is encouraged for both active and passive recreation purposes, subject to meeting Mohave County aesthetic and safety standards.

3.5.6.1 Private Stormwater Storage Basins

The COUNTY/DISTRICT recommends non-geometric designs for stormwater basins in private development projects. In these developments, the use of open space in combination with stormwater storage basins is encouraged in order to provide a more natural and aesthetically pleasing method of addressing runoff, stormwater storage, and stormwater quality. This practice can provide measurable benefits to the residents of the development when a sufficient recreation area is provided. These areas should be made focal points of the community instead of isolated tracts, which helps create a sense of community. Other design considerations include access, multi-use trails and habitat connectivity.

3.5.7 ZONING

Zoning often dictates the nature of watercourse development and open space requirements for land development projects. Rezoning land to address flooding or erosion hazards, either through the use of an overlay or replacement zoning district (such as the City of Phoenix Flood Hazard And Erosion Management District), or through conditions of zoning approval that limit the use of such land, is intended to provide a natural or limited structural design approach to watercourse management. Generally, this results in ideally situated open space. Even small washes lend themselves to regulation in the same manner as larger watercourses if the identification of the flood hazard and erosion impact is initiated early enough. Where regional master drainage plans have been completed, approved implementation plans may dictate land-use/drainage design options. In other areas, individual rezoning applications or zoning overlay districts may include stipulations or design guidelines that address watercourse treatment and the degree to which the watercourse may be altered or disturbed.

3.5.8 DESIGN HYDROLOGY AND HYDRAULICS

The drainage engineer should determine if there is existing hydrologic and hydraulic information available for the upstream watershed and project site that is suitable for use in design of the project improvements. This includes researching the information sources listed in Table 3.1. In particular, review of available regional master drainage plans that encompasse the project area provides the design team with valuable information pertaining to the magnitude of stormwater discharges and volumes affecting the project. The design engineer must either concur with the regional master drainage plan by statement, or submit additional documentation addressing and substantiating differences. The FEMA Flood Insurance Rate Maps (FIRM) should also be reviewed to establish if regulated floodplains cross the project. Where existing studies are not available, the drainage engineer should contact the MCFCD to determine if any regional studies are in progress. In-progress information is often available, and if not, staff experience is extensive.

In the event there is insufficient hydrology or hydraulic information available, then the drainage engineer will have to generate new information using the procedures defined herein. At the preliminary drainage plan level, the drainage engineer should concentrate on quantifying off-site flows that may impact the project, and determine the means for conveying that flow through the project site. A reasonable estimate of the design peak discharge is necessary to approximate the channel or drainage structure capacity and size. Again, the improvements presented in a drainage plan shall not adversely impact adjacent property owners.

3.5.9 OTHER HAZARD CONSIDERATIONS

Drainage plans need to focus on more than flood levels derived from open channel hydraulic analyses. Aggradation of channel beds and overbanks via sedimentation and degradation of channels from erosive processes are threats to the performance of drainage systems that should be considered. In addition, the lateral migration of watercourses may threaten public safety, health and welfare, unless proper erosion hazard zones are identified, prohibiting development in these areas unless remediation of the hazard is accomplished. ADWR (1996b) should be considered and addressed in the planning process. The determination of flood levels on alluvial piedmonts is particularly challenging because of active geomorphic processes. The plan should consider the Flood Control District of Maricopa County's *Piedmont Flood Hazard Assessment for Flood Plain Management for Maricopa County* (Hjalmarson, 2003) or most current version, and the National Research Council (1996), when drainage planning on alluvial piedmonts. Finally, ponding areas up gradient of elevated roads, railroads, and irrigation canals must be considered during the development of the drainage plan to assess finished floor elevations, outfall hydraulics, and compensation for volume displacement.

3.5.10 SAFETY

A basic tenet of any capital improvement project is the promotion of public safety. Public safety must be a consideration throughout the development of a drainage plan. Excessive stormwater depth, velocity, erosion, sedimentation, and/or poor stormwater quality pose a threat to safety and public health.

3.5.11 COST

During the development of a drainage plan, initial capital costs, long term maintenance costs, and stormwater treatment cost should be considered. Ideally, the least societal costs necessary to provide the required level of protection to the public is the desired goal. Attainment of this goal is fostered by adherence to the COUNTY/DISTRICT's policies and standards.

3.6 APPROACH TO DRAINAGE PLANNING

3.6.1 OPEN CHANNEL CONVEYANCE

The alignment of a planned drainage system is often set by following the natural watercourse flow line or low flow channel. In these cases, the alignment need only be defined on available topographic mapping or aerial photographs. In many areas about to be urbanized, the runoff has been so minimal, or is sheet flow, that well-defined natural channels do not exist. However, low flow channels nearly always exist which provide an excellent basis for location of improved channels. Use of these channels to convey stormwater is likely to reduce development costs and minimize drainage problems. In some cases, the wise utilization of natural watercourses in the development of a drainage system will eliminate the need for an underground storm drain system. Where WCMP's have been completed, setbacks for erosion hazard zones may have been identified. If setbacks have not been defined as part of the WCMP, then erosion hazard areas should be approximated following the methodologies identified in ADWR (1996b) and Chapter 17. Detailed lateral migration and long-term erosion analyses would be performed as part of final design in those circumstances.

The preliminary drainage plan is where major decisions are made as to design velocities, location of structures, means of accommodating conflicting utilities, and the potential alternate uses in the case of an open channel. The choices of channel types available to the design team are numerous, depending only upon good hydraulic practice, environmental design (including stormwater quality control and treatment), sociological impact, and basic project requirements.

However, from a practical standpoint, the basic choice to be made initially is whether or not the channel is to be lined for higher velocities or if a natural channel and floodplain already exists that can be effectively utilized with considerations to erosion setbacks and the 100-year flooding limits.

A more natural approach is preferred. The more desirable setting for the channel and overbank floodplain combination is an undisturbed one. The benefits of such a channel are that:

1. Velocities are usually lower, resulting in longer concentration times and lower downstream peak flows.
2. Natural channel and overbank floodplain storage tends to decrease peak flows.
3. Maintenance needs are usually less than artificial channels.
4. The natural channel and overbank floodplain provides desirable open space and recreational area adding significant social benefits. The more closely the character of an artificial channel can be made to emulate that of a natural channel with overbank floodplain, generally the higher the quality of the artificial channel.

For a drainage plan, the level of analysis necessary to establish artificial channel widths varies. If the artificial channel is for a watercourse with a 100-year peak discharge of 500 cfs or greater, a detailed floodplain analysis may be required (see Chapter 8). The level of analysis is also dependent upon the existing or proposed land-use and whether encroachments, such as road culvert embankments, affect the flow regime. Otherwise, simple “normal depth flow” calculations may suffice. Where channel slopes exceed 0.5% to 1.0%, supercritical flow analysis may be warranted.

Another key component of planning for a channel at the preliminary drainage plan level is the transitioning of flow into and out of a proposed channel. Arizona state law requires that proposed facilities do not exacerbate flooding conditions for adjoining properties. Thus, any drainage improvement must not increase water levels or result in erosive velocities greater than pre-development conditions. Interceptor channels, especially in sheet flow areas, may be required to funnel offsite flow into an onsite channel. Similarly, spreading basins or engineered channel expansions may be necessary to transition from an artificial channel to the existing downstream floodplain.

3.6.2 STORAGE

The preliminary drainage plan is where decisions need to be made on the use of stormwater storage facilities and their location. The siting of storage facilities where topography is favorable to the construction of embankments and/or excavation of basins will provide significant benefits including the reduction of peak flows and the settling out of sediment and debris. The latter can help to improve the quality of water downstream.

For conceptual sizing of stormwater storage facilities, a storage per unit area relationship along with a safety factor can be utilized to derive an approximate stormwater volume for storage and stormwater quality treatment. The storage per unit area is primarily dependent upon the land-use of the proposed project within the proposed project area only and upon the design rainfall depth for the area in question. Offsite flows are not allowed to mix with onsite storage facilities.

For land development projects involving large acreage, establishing the contributing drainage area prior to final design can be problematic for the inexperienced. Overlaying the proposed site plan with existing topography allows for the development of a conceptual or preliminary grading plan. Establishing proposed grade breaks for mass grading consistent with existing drainage divides is the preferred method. Taking this approach wherever possible during the drainage planning effort provides an additional benefit in that it minimizes earthwork and storm sewer expenditures pursuant to final design. Undertaking such an approach supports the basis for preliminary stormwater storage design and will tend to minimize the necessity for dramatic design revisions resulting from unforeseen drainage requirements during final design.

3.6.3 ENVIRONMENTAL PROTECTION

There are numerous federal, state, and local regulations that must be adhered to during plan development and implementation. At the federal and state level, Section 404 of the Clean Water Act (Waters of the U.S.) and Section 401 (water quality) permitting are typically required during the project approval process and may be required for maintenance or other activities proposed in conjunction with the drainage facilities. For the MCFCD, the plan must comply with the Federal NPDES (40 CFR 122), the state AZPDES stormwater quality programs, and also any action or restriction they consider reasonably necessary to meet their obligations, if any, to comply with local, state or federal water quality laws. Taking the requirements of these regulations into account during the development of the drainage plan will streamline the design and implementation process. For example, recognition of the trigger points in 404 permitting will provide guidance in developing mitigation plans (see Chapter 2, Federal and State

Regulations). The COUNTY/DISTRICT strongly endorses minimizing disturbances to natural watercourses in order to lessen the impacts on ecology.

3.7 FINAL DESIGN CONSIDERATIONS

The drainage plan serves as the framework for final design. A thorough drainage plan streamlines the final design process. That is not to say that changes will not occur during final design. However, wholesale changes should not occur due to drainage issues.

It is during final design that street drainage is analyzed and catch basins/storm drains are designed. The specifics and supporting analysis for open channels including culverts and bridges, and the influences of sedimentation and scour, are developed during final design. It is here that stormwater storage facility details, including pump stations if appropriate, are enumerated to permit review by the COUNTY/DISTRICT and subsequent construction. During final design, the design engineer applies the DDM to minimize capital cost and long term maintenance of the drainage improvements while accommodating safety and health concerns.

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4 PUBLIC SAFETY

Designs for drainage must address the issue of public safety. This chapter is provided to emphasize the importance of safety considerations for design of drainage improvements, whether it be for open channels, stormwater storage basins, or gutter flow in a roadway. The following sections contain safety-related standards and requirements for design of drainage facilities.

4.1 SPECIAL HAZARDS

The designer should determine if the site is subject to special hazards including, but not limited to, erosion hazards, alluvial fans, or distributary flow areas, and any other special hazards identified by the MOHAVE COUNTY ENGINEER. Alluvial fans and distributary flow areas can be identified using guidance provided by Appendix G of FEMA (2003) and Hjalmarson (2003). Engineering analysis and design are required for development within special hazard areas.

4.2 FLOOD WARNING

The DISTRICT owns and maintains a county-wide flood warning system consisting of rain gages, stream flow gages and weather stations. This system is for the benefit of all Mohave County residents and is a part of the Arizona statewide flood warning system. The flood warning system has multiple benefits and is used by both the public and other government agencies for various purposes that benefit the public. The DISTRICT has an ongoing program for improving and expanding the system. To enhance this effort, new developments may be required to assist by allowing installation of gages within the development on a dedicated parcel or within a dedicated easement that grants the DISTRICT access to install and maintain the gage site. New master planned developments that are one (1) square mile or greater in area are required to fund the purchase and installation (by DISTRICT personnel) of at least one rain gage, and at least one stream flow gage for washes with a 100-year peak discharge of 5,000 cfs or greater. This includes furnishing a parcel or easement for access, installation and ongoing maintenance. Additional weather station devices may also be required as a part of the rain or flow gage installation.

If low water roadway crossings of washes with a 100-year peak discharge of 500 cfs or greater are approved for a development, signage, barricades or other flood warning devices may be required for public safety and welfare. The citizens of the new development will benefit from all these flood warning facilities and they are considered part of the public infrastructure for public safety.

4.3 PROTECTION RELATED TO DEPTH AND VELOCITY

4.3.1 GENERAL

The designer shall carefully consider public safety where standing water depths, and water flow depths and velocities pose a hazard. This should be done for design of all drainage facilities, including stormwater storage facilities, channels, storm drains and street systems. Figure 4.1 and Figure 4.2 (USBR 1988) can be used in this regard to aid in defining the level of hazard, based on criteria such as the type and frequency of use of the facility by the public, access concerns for emergency response vehicles, the statistical frequency of hazardous storm events, and risks associated with public access combined with the frequency of the hazard. Engineering judgment shall be applied in assessing the risks and determining which areas require special attention. With the areas of concern defined, the designer should include mitigation measures appropriate to the risk to discourage or prevent public access to these facilities during a flood event. The measures could include, but are not limited to:

1. Mitigating design criteria such as maximum flow rates and depths.
2. Signage to alert the public to the hazard and/or physical barriers, such as fencing or railings.
3. Flood warning alarm or announcement systems.
4. Higher minimum technical standards for design of drainage facilities.

Figure 4.1 Depth-velocity flood danger relationship for adults

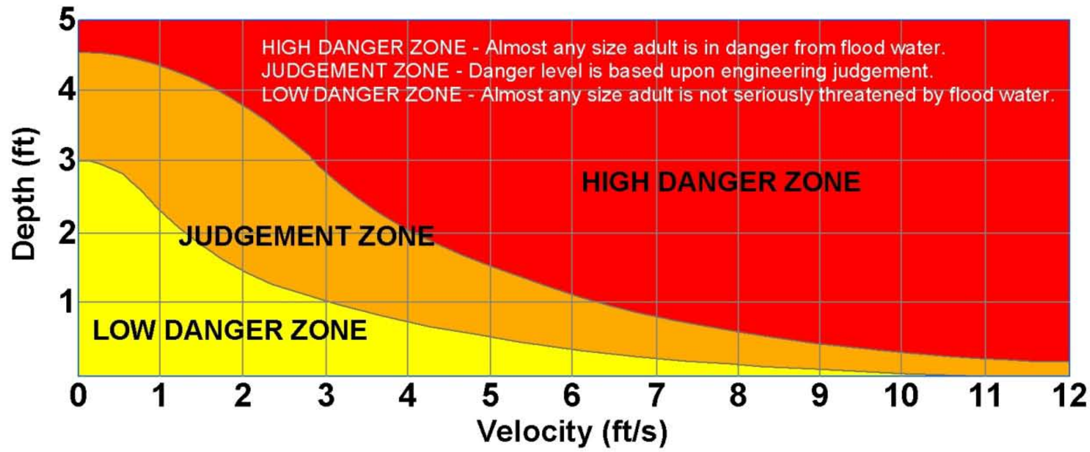
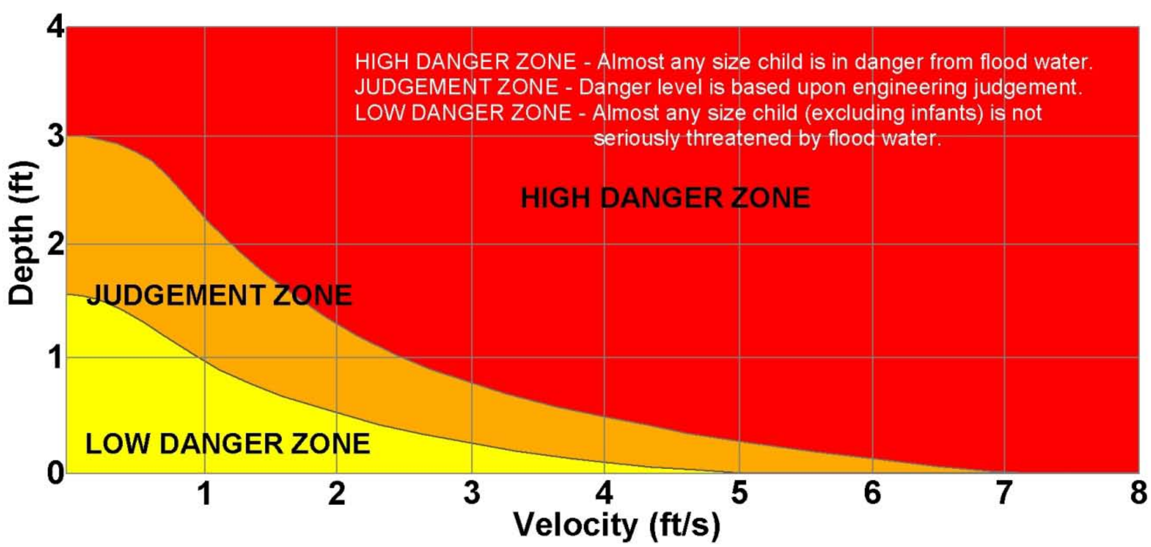


Figure 4.2 Depth-velocity flood danger relationship for children



4.3.2 ENGINEERED OPEN CHANNELS

4.3.2.1 General

For engineered portions of channels with actual water depths greater than three (3) feet in the 100-year event; and for shallow, fast-flowing, engineered channels where the product of maximum depth and average velocity exceeds ten (10) ft^2/sec for the 100-year event:

1. Appropriate measures shall be designed to warn the public and keep them away from these locations.
2. Adequate fencing or railings must be provided along all walls, such as wing walls or training walls, excluding vertical drops in the channel bottom.

4.3.2.2 Channel Drop Structures

For channel drop structures, the maximum vertical drop height from invert crest to invert toe for any single step shall be 2.5 feet. A six foot wide (minimum) horizontal apron shall be provided for every 2.5 feet of vertical drop in a "stair step" fashion. Drop structures constructed of concrete or shotcrete shall have a roughened surface to discourage inappropriate recreational use.

4.3.2.3 Emergency Escape Stairs

All concrete, shotcrete, or smooth sided soil cement channels with design flow depth greater than three feet shall have emergency escape stair steps formed; alternating every 300 feet from one side of the channel to the other.

4.3.2.4 Walkways

Walkways shall meet the Americans with Disabilities Act (ADA) requirements and be elevated at least one foot above the invert of any low flow channel.

4.3.3 STORMWATER STORAGE FACILITIES

4.3.3.1 Depth Criteria for Ponds with a Permanent Water Body

For storm water storage ponds with a permanent water body in the bottom, the pond edge shall be designed to minimize safety hazards. Water depth should be limited to 1.5 to 2 feet within eight feet of the edge of the water feature, and gradually get deeper as needed.

4.3.3.2 Securing Amenities

Amenities placed within the inundation area of a storm water facility shall be adequately secured to prevent them from becoming waterborne debris. Methods for securing items shall be documented in the design report and shown on the approved construction plans.

4.3.3.3 Fencing

Where basins are accessible by the public, adequate fencing is required along portions of engineered basins where the design maximum ponding depth is greater than three-feet deep and side-slopes are steeper than 3:1.

4.3.4 CULVERTS AND STORM DRAINS

Trashracks may be required on the entrances and access barriers on outlets to conduits or other hydraulic structures. Where such barriers are required, they shall be placed on both the inlet and outlet ends. They are required in areas where debris potential and/or public safety indicate they are necessary, such as in developed areas or where a person could likely be injured or trapped. Refer to Table 4.1 for additional guidelines within such areas. An additional reference for design of debris barriers, particularly for larger hydraulic structures, is FHWA (2005a).

Table 4.1 Conduit and Hydraulic Structure Trashrack and Access Barriers

Facility Description	Diameter or Cross Sectional Area (per barrel)	Length	Inlet Trash Rack Required	Outlet Access Barrier Required
Culverts and Storm Drains	Dia < 24” Area < 3.14 sf	All	No	No
Outlets from multiple-use stormwater storage facilities.	Dia≥ 24” Area ≥ 3.14 sf	All	Yes	Yes

Table 4.1 Conduit and Hydraulic Structure Trashrack and Access Barriers

Facility Description	Diameter or Cross Sectional Area (per barrel)	Length	Inlet Trash Rack Required	Outlet Access Barrier Required
Culverts and Storm Drains with sufficient bend that the opposite end cannot be clearly seen when looking into the structure.	Dia ≥ 24″ Area ≥ 3.14 sf	All	Yes	Yes
Culverts and Storm Drains, other than noted above	Dia ≥ 24″ 3.14 sf < Area ≤ 15 sf	L < 200 ft L ≥ 200 ft	No Yes	No Yes
Culverts and Storm Drains, other than noted above	Area > 15 sf	All	No	No

4.3.5 ACCESS TO DRAINAGE FACILITIES

All drainage facilities must be readily accessible by emergency or ordinary maintenance vehicles (e.g., pickup truck, loader, backhoe, dump truck, water truck, etc).

1. For engineered channels and storm water storage facilities/basins with geometric depths greater than three feet deep, access ways to the channel or basin and ramps into the channel or basin shall be required.
2. For engineered channels or storm water storage facilities/basins with geometric depths of three feet deep or shallower with a portion of side slope set at 6:1 or flatter along at least one side to allow emergency or ordinary maintenance vehicle access, ramps into the channel or basin are not required.
3. For all other small engineered channels such as swales, roadside drainage ditches, etc., reasonable access for emergency and ordinary maintenance vehicles shall be provided.
4. For natural washes, a minimum 16-feet wide accessible clear-zone area for emergency and ordinary maintenance vehicle access shall be provided.
5. Access ramps shall be a minimum of 12 feet wide with a longitudinal slope no steeper than 10%. Access ways approaching channels or basins shall be a minimum of 12 feet wide

within a clear 16-feet wide tract such that emergency and ordinary maintenance vehicles can freely maneuver.

6. At a minimum, hard surface paving (such as concrete, soil cement, etc.) shall be required for the portions of access ramps that will be inundated in the 100-year event, and shall be properly "toed-in" to protect the ramp from erosion during storm events.
7. Portions of access ways or ramps may be combined with portions of multi-use trails, subject to approval by COUNTY.
8. The design engineer may propose other means of providing access for maintenance by ordinary maintenance equipment subject to approval by COUNTY.

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5 GENERAL DRAINAGE POLICIES AND STANDARDS

5.1 STANDARD DETAILS AND SPECIFICATIONS

All hydraulic drainage structures are to be designed and constructed, as a minimum, in conformance with the *Uniform Standard Specifications and Details for Public Works Construction* (MAG Standards) by the Maricopa Association of Governments, latest edition, including any Mohave County amendments. Use of the *ADOT Standard Specifications for Road & Bridge Construction* and *Standard Drawings* (ADOT Standards), latest edition of both including any Mohave County amendments, is permissible where the MAG Standards do not provide a standard. Additional details and specifications may be necessary or required, and in all cases, the final approved construction documents, compliant with current design standards, shall control. The latest edition of the *Uniform Standard Details for Mohave County* (Standard Details) is also referenced herein and the drainage policies and standards are tied to the roadway classifications defined in that document.

5.2 ACCEPTANCE OF EXISTING STRUCTURES/FACILITIES

Prior to the acceptance by Mohave County to incorporate existing structures and/or facilities for maintenance, such structures and/or facilities shall be refurbished for the life cycle specified by the MOHAVE COUNTY ENGINEER and constructed or reconstructed as a minimum, in conformance with these policies and standards, and the MAG Standards, latest edition, including any Mohave County amendments. Use of the ADOT Standards, latest edition including any Mohave County amendments, is also permissible provided there is not an applicable MAG Standard. Additional details and specifications may be necessary or required, and in all cases, the final approved construction documents, compliant with current design standards, shall control.

5.3 PERMITS

There are a myriad of federal, state, and county permits that may be required prior to the start of construction of a project (see Chapter 2 and Chapter 8). It is not the COUNTY/DISTRICT's responsibility to ensure that the plans for a proposed project satisfy state and federal permit requirements. It is the COUNTY/DISTRICT's policy that all such permits must be obtained, but it is the owner's responsibility to determine which permits are required and to obtain them as appropriate for the timing of the project. COUNTY/DISTRICT-issued permits may be withheld pending written proof that required State and/or Federal permits have been obtained.

6 STORMWATER QUALITY

The DISTRICT has established a minimum level of control for new development at which stormwater pollution prevention practices are put in place. This minimum standard is "First Flush", and consists of retaining the first 0.5 inches of direct runoff from a storm event. This minimum level of control is met by following the County retention requirement (Chapter 15). In the event that normal County retention standards are waived (100 year, 2 hour storm), or a surface based bleed off for the retention basin is proposed, the following first flush provisions shall apply to stormwater runoff from new development that drain into a stormwater facility (channels or stormwater storage basins) that is owned or operated by the DISTRICT:

The First Flush requirement can be addressed by:

1. Retaining the required minimum First Flush volume, as defined below, or
2. Treating the first flush discharge, or
3. Utilizing a combination of both approaches.

The minimum First Flush volume is calculated as follows:

$$V_{FF} = C\left({}^{P_{FF}}/_{12}\right)A \qquad 6.1$$

where:

V_{FF} = minimum First Flush volume in ac-ft,
C = runoff coefficient (set = 1),
P_{FF} = first 0.5 inches of direct runoff, and
A = area of project site, in acres.

The minimum First Flush treatment discharge is calculated as follows:

$$Q_{FF} = CI_{FF}A \qquad 6.2$$

where:

Q_{FF} = minimum First Flush discharge in cfs,
C = runoff coefficient (set =1),
I_{FF} = maximum first flush intensity in in/hr
where: $I_{FF} = \frac{P_{FF}}{T_c}$
T_c is the Time of Concentration of the upstream watershed in hours.
A = area of project site, in acres

This first flush standard is the result of ARS 48-3622 where the DISTRICT may require any action or impose any restriction that the DISTRICT considers reasonably necessary to meet the DISTRICT's obligations, if any, to comply with local, state or federal water quality laws.

For all other cases the COUNTY, when considering a waiver of the stormwater retention policy, encourages the voluntary implementation of the First Flush provisions.

7 HYDROLOGY

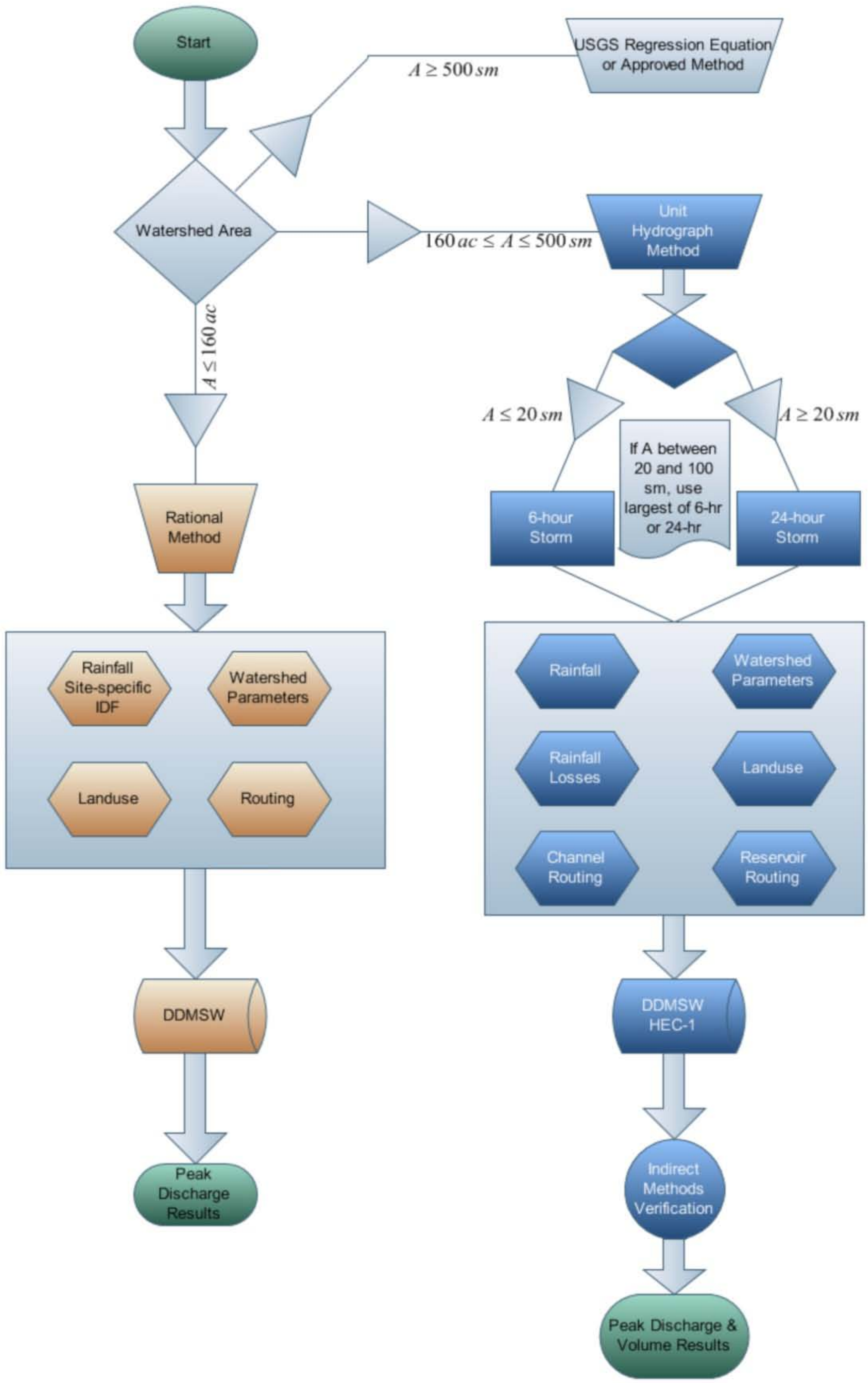
7.1 INTRODUCTION

The hydrologic methodology for Mohave County is based on methods that are accepted state-wide in Arizona. The hydrologic methods are adapted from the Arizona Department of Transportation *Highway Drainage Design Manual, Hydrology* (ADOT, 1993). Some aspects of the methodology are also adapted from the Flood Control District of Maricopa County *Drainage Design Manual for Maricopa County, Hydrology*, 2009 (FCDMC, 2009a). The user is referred to both documents for background information and more detailed technical documentation than is contained herein. The methods set forth in this chapter are summarized as follows:

1. Rainfall. The NOAA Atlas 14 is accepted for use in Mohave County. In general, the median values should be used. Point precipitation data should not be obtained from the NOAA Atlas 14 web site. The specific version of NOAA Atlas 14 adopted for use in Mohave County is included as a part of the DDMSW (KVL, 2009) computer program, shown on isopluvial maps included herein in Appendix B, and available from the Mohave County Flood Control District web site.
2. Rational Method. The Rational Method should be used for estimating peak discharge for various frequency storms when the watershed area is less than or equal to 160 acres. The method includes provision for multiple sub-basins and is implemented using DDMSW. The hand computation method can also be applied.
3. Rainfall Losses. The Green and Ampt method is accepted for use with the Unit Hydrograph Method.
4. Unit Hydrograph Method. The Clark unit hydrograph is accepted for use in Mohave County.
5. Flood Frequency Analysis. Flood frequency analysis methods may be used for estimating peak discharges, particularly when the watershed area exceeds 500 square miles.
6. Hydrologic Routing and Verification. Guidance is provided for channel and storage routing of hydrographs and for verification of model results using indirect methods.

A flow chart depicting the general process for applying Mohave County hydrologic methodology is shown in Figure 7.1.

Figure 7.1 Hydrologic method flow chart



7.2 RAINFALL

7.2.1 INTRODUCTION

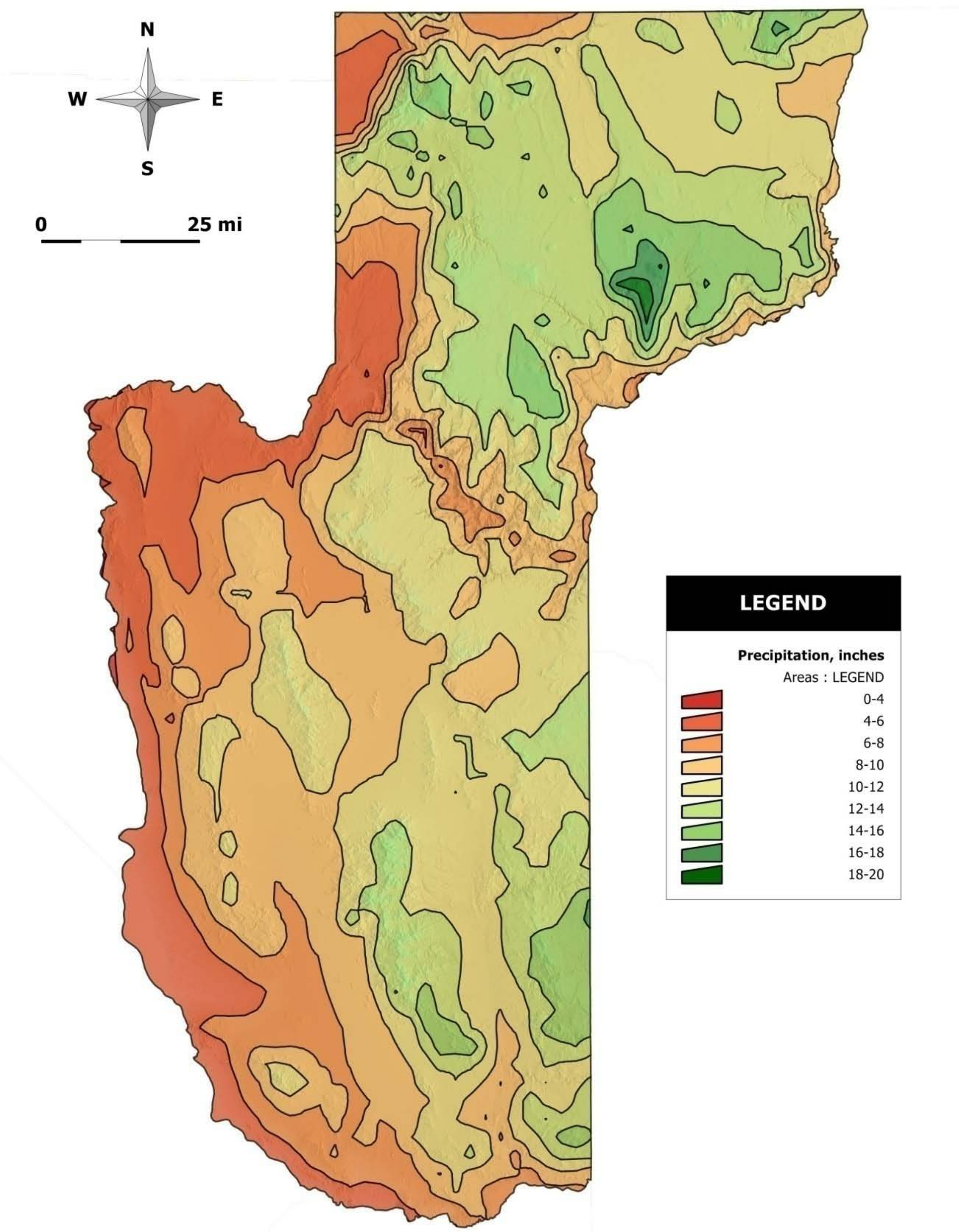
7.2.1.1 General Discussion

Precipitation in Mohave County is strongly influenced by variation in climate, changing from a warm and arid desert environment at lower elevations to a seasonally cool and moderately humid mountain environment. Mean annual precipitation ranges from 4-inches along western Mohave County to 20-inches in the mountain regions of Mohave County (refer to Figure 7.2). Precipitation is typically divided into two seasons of comparative rainfall depths: summer (June through October) and winter (December through March). Warm, moist tropical air can move into Arizona at anytime of the year, but most often does so in the summer months, resulting in severe convective storms and local flooding. Storms of large areal extent are usually associated with frontal or convergence storm activity that may result in long duration rainfall and flooding of major drainage watercourses. These types of storms and flooding usually occur in the winter, but occasionally occur in the summer. The design storm methodology for Mohave County is designed to simulate the peak discharge resulting from both types of storms.

Analytic methods for rainfall-runoff modeling including the Rational Method and the Unit Hydrograph Method require the definition of the rainfall for the desired flood frequency. For the Rational Method, a rainfall intensity-duration-frequency (I-D-F) table or graph is required. Site-specific I-D-F tables or graphs are to be used with the Rational Method in Mohave County. The standard method for developing a site-specific I-D-F and applying the Rational Method is to use the DDMSW computer program developed by KVL Consultants, Inc. for Mohave County. Manual computation methods may also be used.

For rainfall-runoff modeling using the Unit Hydrograph Method, the temporal and spatial distribution of the design rainfall must be provided. For drainage studies in Mohave County, a site-specific symmetric nesting of rainfall depths for specified intra-storm durations is used. That rainfall distribution is called the hypothetical distribution, and is applied using the US Army Corps of Engineers HEC-1 computer program (USACE, 1998), with input data created using DDMSW.

Figure 7.2 Average annual precipitation for Mohave County



Source: Oregon Climate Service, Oregon State University, 2007

7.2.1.2 Source of Design Rainfall Information

The rainfall depth-duration-frequency statistics for Mohave County are derived from information in *NOAA Atlas 14 (*Bonnin, G.M. et al, 2004). The NOAA Atlas 14 data is used to generate the site-specific D-D-F and I-D-F data for use with the Rational Method and the point precipitation values for use with the Unit Hydrograph Method. The depth-area reduction curves for use with Unit Hydrograph Method are from NWS Hydro-40 for zone C. Temporal rainfall distributions for various durations and frequencies are derived from NOAA Atlas 14 and applied using the US Army Corps of Engineers hypothetical distribution.

The NOAA Atlas 14 web site may not be used for obtaining design rainfall information for use in Mohave County. Mohave County has adopted the specific version of NOAA Atlas 14 listed above. Geographical Information System (GIS) coverages of the specific version of NOAA Atlas 14 adopted by Mohave County are available for download from the Mohave County web site at:. http://gis.co.mohave.az.us/. The most current version of NOAA Atlas 14 on the NOAA web site may be a newer version than adopted by Mohave County. Mohave County will review and possibly adopt the most current version of NOAA Atlas 14, at its discretion. But until a newer version is officially adopted, the version posted on the Mohave County web site is to be used for all drainage and floodplain studies in Mohave County.

7.2.1.3 Applications and Limitations

The rainfall statistics that are developed by procedures in this section are dependent upon the information that is provided in the *NOAA Atlas 14* (Bonnin, G.M. et al, 2004). It is recommended that the user become familiar with the assumptions, limitations and theoretical and technical basis for NOAA Atlas 14 prior to applying these procedures. NOAA Atlas 14 is the most current and comprehensive precipitation frequency atlas available for Arizona and will be used for drainage design and floodplain delineation studies in Mohave County.

The hypothetical distribution is a simplified and idealized representation of the temporal distribution of rainfall. It is intended for use in estimating design discharges for drainage facilities and floodplain delineations. It does not necessarily represent the temporal distribution of any historical storm in Mohave County. The use of the hypothetical distribution for design purposes does provide reasonable assurance that design discharges of specified frequency are produced regardless of the size of the watershed.

For very large watersheds (possibly as large as or larger than 500 square miles), where the time of concentration (T_c) exceeds 24 hours, a longer duration hypothetical distribution (or other project specific distribution) should be developed and used. Procedures for estimating the watershed time of concentration are contained in Section 7.5.2.2.

In general, the hypothetical distribution can be used as input to the HEC-1 program for drainage design purposes in Mohave County. Similarly, site-specific I-D-F graphs generated using NOAA Atlas 14 (see Section 7.2.2.2) can be used with the Rational Method, within the limitations specified in that section, for most small watersheds less than 160 acres in area within Mohave County.

7.2.2 DESIGN RAINFALL CRITERIA

7.2.2.1 Modeling Methods

Rational Method

A site-specific I-D-F graph is used when estimating the design rainfall intensity.

Unit Hydrograph Modeling

The storm duration to be used depends on the total watershed area as follows:

1. Stormwater Storage Facilities. The 100-year 2-hour rainfall shall be used, as a minimum, for the design of stormwater storage facilities in Mohave County, regardless of watershed area.
2. Stormwater Conveyance Facilities and Floodplain Delineation. The storm frequency for design of stormwater conveyance facilities, including open channels, bridges, culverts, storm drains, and roadway drainage, varies with the application. Refer to the appropriate chapter for each type of facility for design storm frequency standards. Floodplain delineation studies shall be done using the 100-year storm frequency, as a minimum. The design storm duration varies with watershed size as follows:
    a. If the total watershed area is less than or equal to 20 square miles, the design storm duration is 6-hours.
    b. If the total watershed area is between 20 and 100 square miles, the maximum discharge from the 6- or 24-hour storm is to be used.
    c. If the total watershed area is greater than or equal to 100 square miles, the design storm duration is 24-hours.

7.2.2.2 Site-Specific I-D-F

A site-specific I-D-F may be generated using one of the following three methods, in order of decreasing accuracy:

1. The DDMSW GIS method. If the user has a digital polygon of the study area in ESRI shape file format, DDMSW can be used to generate a site-specific I-D-F. The shape file must be properly georeferenced using the standard Mohave County specification (State Plane Coordinate System, Arizona West, NAD83, and U.S. Survey feet). DDMSW will then overlay a series of ESRI grid file surfaces for the appropriate storm durations and frequencies and generate an I-D-F for the study area. DDMSW will generate point precipitation estimates for the 2-, 5-, 10-, 25-, 50-, and 100-year recurrence interval storms for durations of 5-, 10-, 15-, and 30-minutes and 1-, 2-, 3-, 6-, 12-, and 24-hours.

2. The DDMSW Manual Method. The user may use maps in PDF format that can be viewed from DDMSW to select the approximate location of the subject watershed. Given the section and township and range, the user can use the provided maps to select the map and index grid numbers that cover the study area. DDMSW then generates a site-specific I-D-F for the study watershed using the recurrence intervals and durations listed for method 1.

3. Manual Method. The user may use the isopluvial maps in Appendix B.1 through Appendix B.6 to estimate the point precipitation values for the 2-, 5-, 10-, 25-, 50-, and 100-year recurrence interval storms for durations of 5-, 10-, 15-, and 30-minutes and 1-, 2-, 3-, 6-, 12-, and 24-hours. The user can then tabulate the values as a depth-duration-frequency (D-D-F) table. An example for the Kingman area is shown in Table 7.1, developed using the DDMSW GIS Method. The D-D-F values are then used to compute rainfall intensity and prepare an I-D-F table (Table 7.2). The values in the Table 7.2 are computed by dividing each point precipitation value in Table 7.1 by the corresponding duration in hours. An I-D-F graph of the computed rainfall intensities versus duration for each storm frequency can then be hand plotted on log-log paper or using a computer spreadsheet such as Microsoft Excel. Blank tables and blank graph paper for tabulating and plotting point precipitation values and computed rainfall intensities are contained in Appendix B.7. An example of an I-D-F graph based on the

data in Table 7.2 is shown on Figure 7.3. An exercise that will help the user understand the process is to use the maps in Appendix B.1 through Appendix B.6 to obtain rainfall values for Kingman and then compare the values read from the figures against the values in Table 7.2, which were taken from the more detailed NOAA Atlas 14 GIS data.

7.2.2.3 Temporal Rainfall Distribution

The temporal rainfall distributions for use with the Unit Hydrograph Method are to be the hypothetical distribution generated in HEC-1 (USACE, 1988) using the PH record option. This method provides a site-specific rainfall distribution for each storm frequency modeled as described in U.S. Army Corps of Engineers, Training Document No. 15 (USACE, 1982). An example of a site-specific 100-year 24-hour hypothetical distribution for Mohave County rainfall gage 1570 at Hualapai Mountain is shown graphically in Figure 7.4 compared with the NRCS Type 2 24-hour rainfall distribution. The two distributions are very similar.

The hypothetical distribution is implemented in HEC-1 using the PH record. The PH record field structure and a sample PH record are shown in Figure 7.5. Refer to the HEC-1 User Manual (USACE, 1988). Field 0 is mandatory and defines the record type. Fields 1 and 2 are for studies using TP-40, which is not the case for Mohave County. Both fields are to be left blank. Fields 3 through 10 define the point precipitation values for the storm frequency being modeled. Each value corresponds to the listed duration from the site-specific D-D-F. Fields 3 through 8 are completed, and field 9 and 10 are left blank, when modeling a 6-hour storm. Fields 3 through 10 are completed when modeling a 24-hour storm. An example PH record is shown using the D-D-F from Table 7.1 for Kingman, AZ.

Table 7.1 Rainfall Depth-Duration-Frequency for Kingman, AZ

Duration	Rainfall Depth, in inches					
	Storm Frequency, in years					
	2	5	10	25	50	100
5-min	0.252	0.355	0.428	0.526	0.600	0.676
10-min	0.384	0.541	0.652	0.801	0.913	1.029
15-min	0.477	0.670	0.808	0.993	1.131	1.275
30-min	0.642	0.903	1.088	1.337	1.523	1.717
1-hour	0.794	1.117	1.347	1.655	1.885	2.125
2-hour	0.873	1.233	1.512	1.896	2.213	2.546
3-hour	0.935	1.302	1.600	2.024	2.381	2.767
6-hour	1.100	1.509	1.835	2.306	2.692	3.116
12-hour	1.284	1.755	2.136	2.658	3.089	3.540
24-hour	1.586	2.175	2.634	3.283	3.801	4.353

Table 7.2 Rainfall Intensity-Duration-Frequency for Kingman, AZ

Duration	Rainfall Intensity, in inches/hour					
	Storm Frequency, in years					
	2	5	10	25	50	100
5-min	3.024	4.260	5.136	6.312	7.200	8.112
10-min	2.304	3.246	3.912	4.806	5.478	6.174
15-min	1.908	2.680	3.232	3.972	4.524	5.100
30-min	1.284	1.806	2.176	2.674	3.046	3.434
1-hour	0.794	1.117	1.347	1.655	1.885	2.125
2-hour	0.437	0.617	0.756	0.948	1.107	1.273
3-hour	0.312	0.434	0.533	0.675	0.794	0.922
6-hour	0.183	0.252	0.306	0.384	0.449	0.519
12-hour	0.107	0.146	0.178	0.222	0.257	0.295
24-hour	0.066	0.091	0.110	0.137	0.158	0.181

Figure 7.3 Site-Specific I-D-F for Kingman, AZ

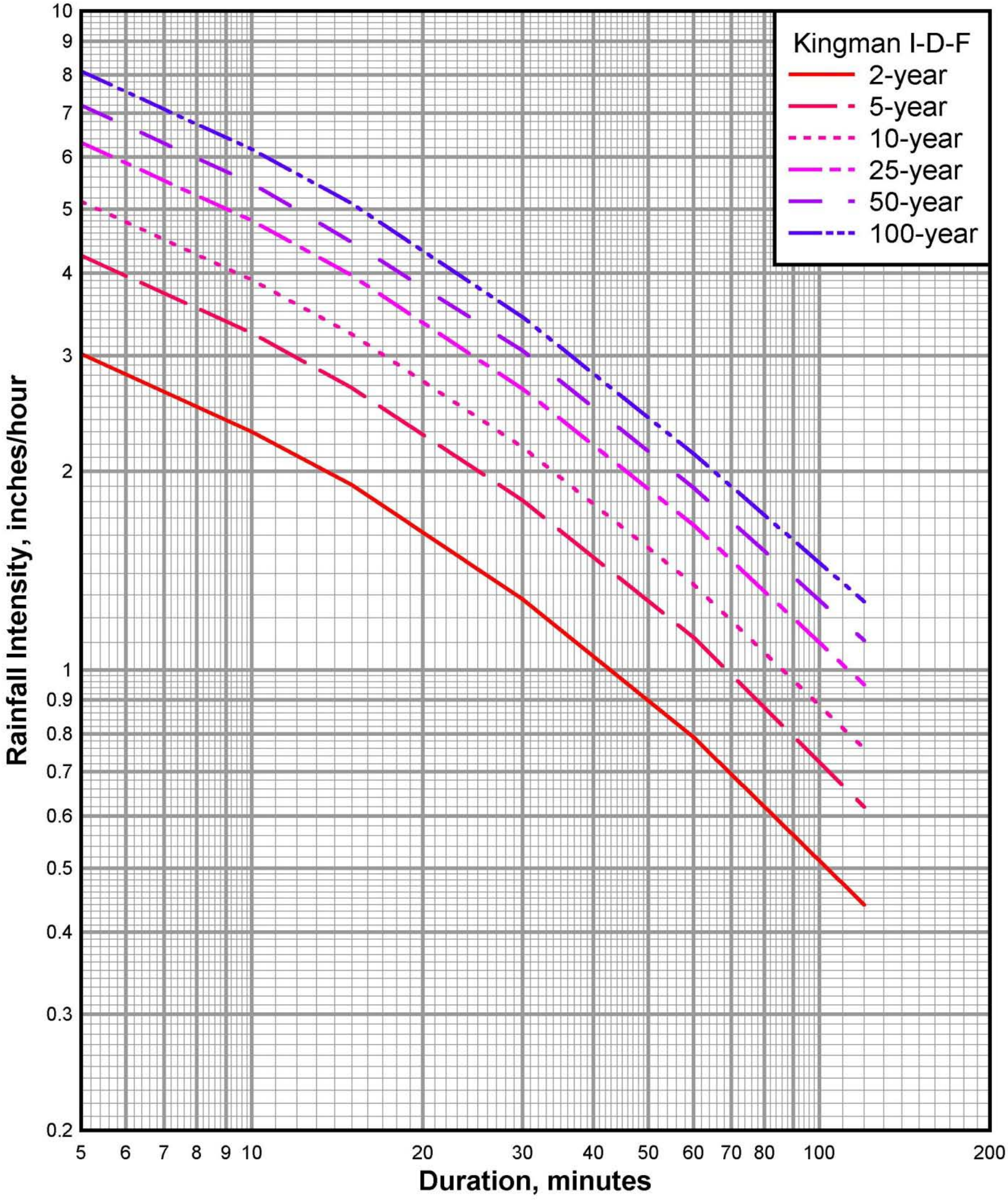


Figure 7.4 Typical hypothetical distribution compared with NRCS Type 2

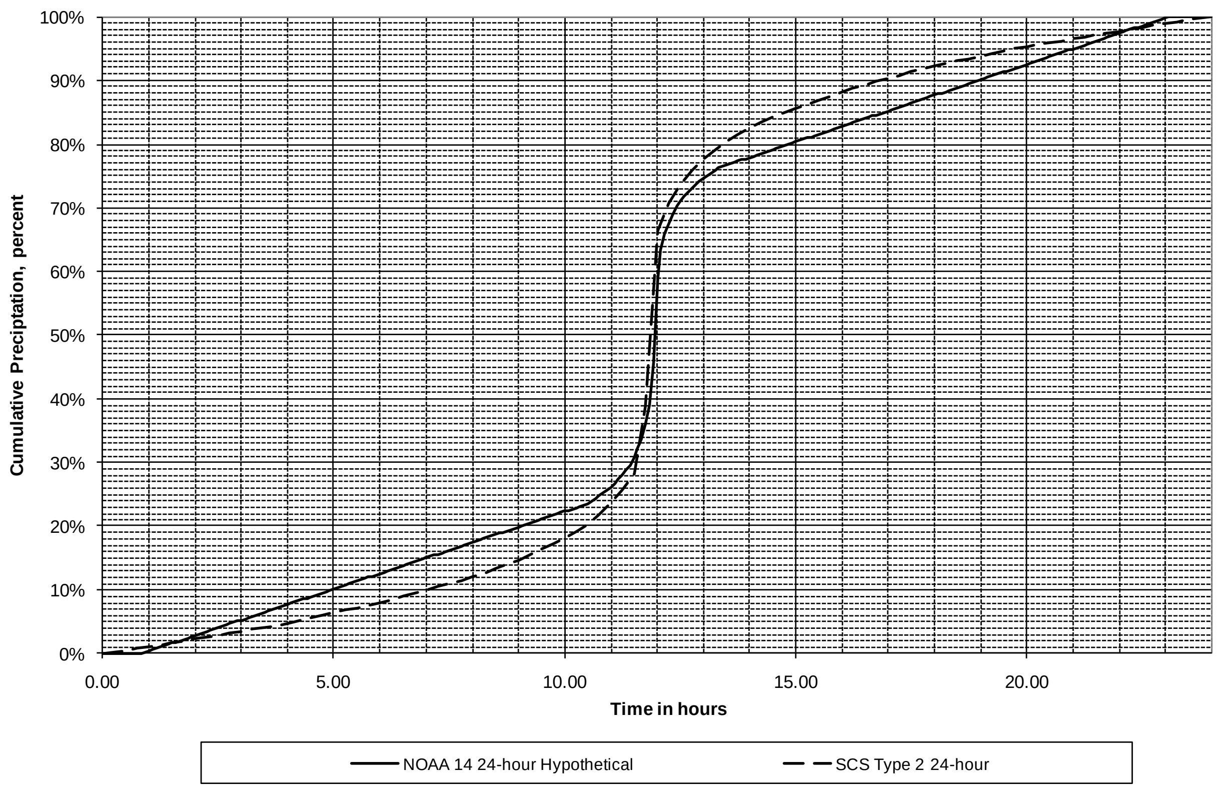


Figure 7.5 PH record field structure and example

HEC-1 PH Record Data Fields										
0	1	2	3	4	5	6	7	8	9	10
PH	PREQ	TRSDA	PNHR(1)	PNHR(2)	PnHR(3)	PNHR(4)	PNHR(5)	PNHR(6)	PNHR(7)	PNHR(8)
			5-min	15-min	1-hour	2-hour	3-hr	6-hour	12-hour	24-hour
Example PH Record for Kingman										
PH			0.66	1.25	2.08	2.49	2.72	3.08	3.5	4.3

7.2.2.4 Depth-Area Relation

The rainfall depths from the isopluvial maps in Figure B.1 through Figure B.60 of Appendix B.1 are point rainfalls for specified frequencies and durations. This is the depth of rainfall that is expected to occur at a point or points in a watershed for the specified frequency and duration. However, this depth is not the areally-averaged rainfall over the basin that would occur during a storm. Estimating the areally-average rainfall requires a two step process.

First, an average point rainfall value for the watershed must be estimated. This can be done by examination of the appropriate isopluvial map and visual estimation of the average value. If there is a high degree of variation in point rainfall across the watershed, the user may need to lay a grid over the watershed, assign a point rainfall value to each grid and then average the rainfall amounts. DDMSW has two methods for accomplishing this, as discussed in Appendix A.1.

Second, a reduction factor is used to convert the watershed average point rainfall to an equivalent uniform depth of rainfall over the entire watershed. As the watershed area increases, the reduction factor decreases, reflecting the greater non-homogeneity of rainfall for storms of larger areas.

Regional research by the Agricultural Research Service, U.S. Department of Agriculture, for the Walnut Gulch Experimental Watershed near Tombstone, Arizona, indicated that local storms are characterized by relatively small areas of high intensity rainfall resulting in depth-area reduction curves that decrease rapidly with increasing area. For local storms (6-hour duration), the depth-area reduction curve that is to be used in Mohave County is shown in Table 7.3 and Figure 7.6. For the 24-hour general storm, the depth-area reduction curve that is to be used in Mohave County is shown in Table 7.3 and Figure 7.7. These curves are taken from Figure 15 of the National Weather Service HYDRO-40 (Zehr and Myers, 1984). Use the factors to adjust the point precipitation values by multiplying the factor times the point value.

For design storms other than what is specified in this manual, the depth-area reduction and temporal distribution will need to be developed on a case-by-case basis depending on the purpose of the study, location of the watershed, and other meteorological and hydrological factors.

7.2.2.5 Examples

Refer to Appendix A.1 for an example of development of a site-specific D-D-F and I-D-F and an example of preparation of rainfall data for use with the Unit Hydrograph Method. The results of the manual method are compared with those prepared using DDMSW.

Table 7.3 Depth-Area curves for Mohave County

6-hour storm		24-hour storm	
Area, sq-mi	**Depth-Area Factor**	**Area, sq-mi**	**Depth-Area Factor**
0	1.000	0	1.000
5	0.909	10	0.950
10	0.851	20	0.919
15	0.811	30	0.900
20	0.785	40	0.887
25	0.766	50	0.877
50	0.715	60	0.870
75	0.676	70	0.863
100	0.649	80	0.857
---	---	90	0.852
---	---	100	0.848
---	---	110	0.844
---	---	120	0.841
---	---	130	0.838
---	---	140	0.835
---	---	150	0.833
---	---	200	0.820
---	---	250	0.820
---	---	300	0.812
---	---	400	0.806
---	---	500	0.796

Figure 7.6 6-hour storm depth-area reduction curve

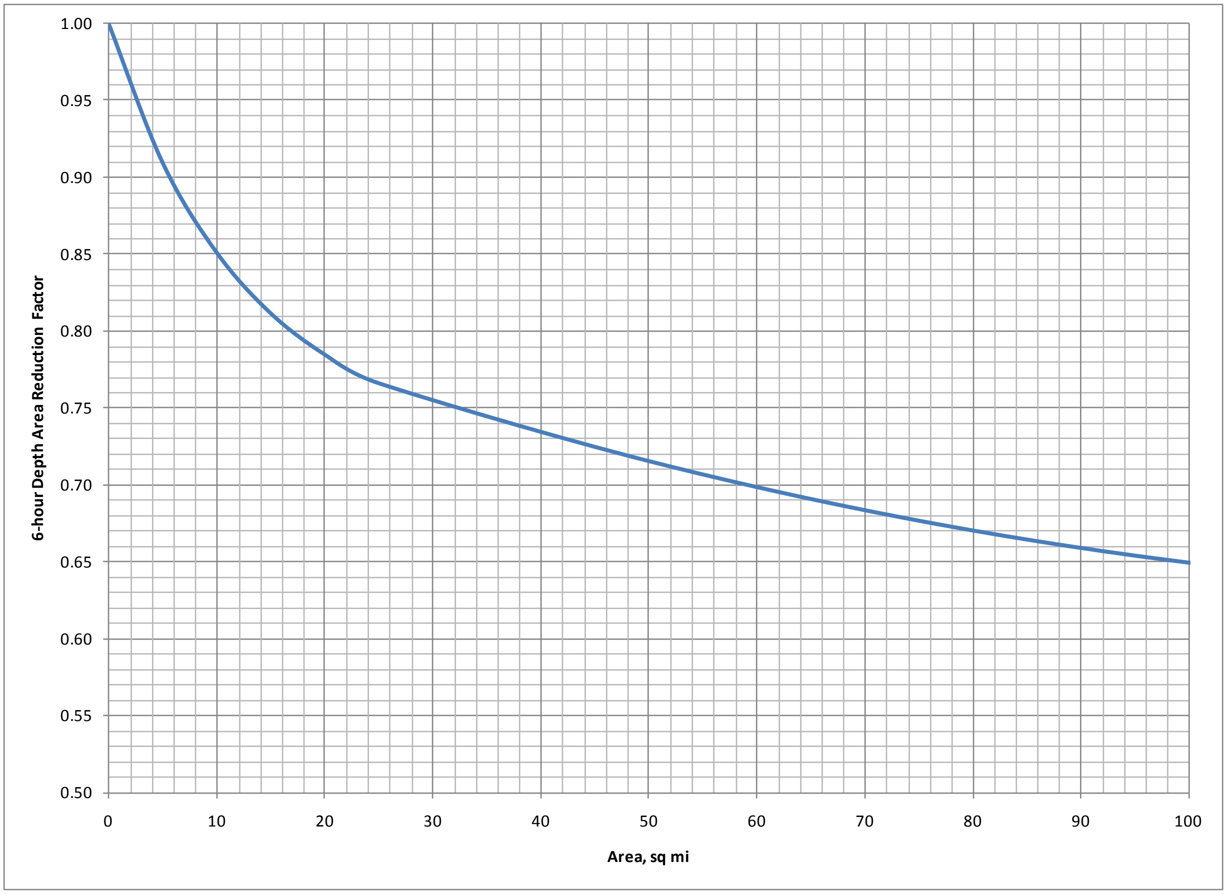
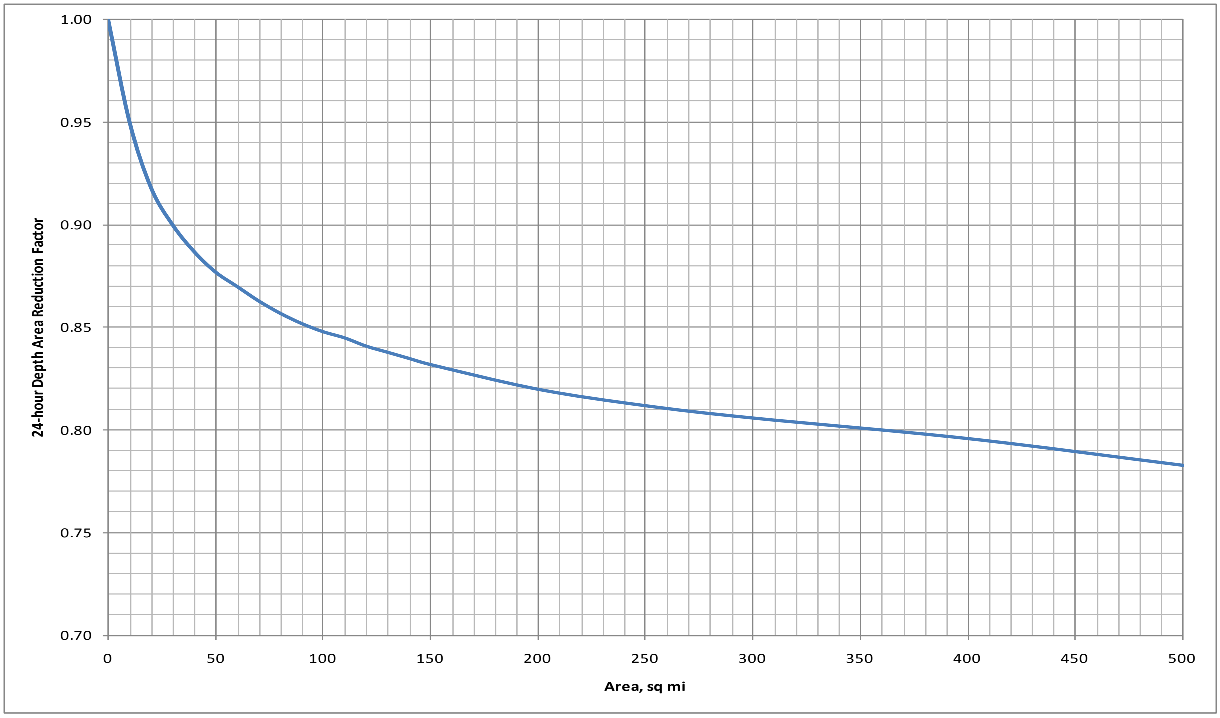


Figure 7.7 24-hour storm depth-area reduction curve



7.2.3 APPLICATION

7.2.3.1 Instructions for Preparing a Site-Specific I-D-F

1. Determine the location of the center of the subject watershed. Locate by section, township and range, or using world coordinates (latitude and longitude).
2. Assemble copies of Figures B.1 through B.60 and make copies of the D-D-F and I-D-F forms contained in Appendix B, Section B.8.
3. Locate centroid of the watershed on Figure B.1. If the watershed is large (> 5 sm) and there is significant variation in point rainfall across the watershed, it may be necessary to draw the watershed at scale on a copy of the isopluvial map
4. Interpolate a point rainfall value for the 2-year 5-minute storm using the rainfall isopluvials shown on the figure. Write the 2-year 5-minute point rainfall value on the D-D-F form.
5. Repeat this process for Figures B.2 through B.60. Tabulate the results on the D-D-F form.
6. Compute the I-D-F values for each duration and frequency and tabulate the results on the I-D-F form.

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7.3 RATIONAL METHOD

7.3.1 INTRODUCTION

The Rational Method relates rainfall intensity, a runoff coefficient and a drainage area size to the direct runoff from the drainage basin.

Basic assumptions of the Mohave County implementation of the Rational Method are:

1. The frequency of the storm runoff is the same as the frequency of the rainfall producing the runoff (i.e., a 25-year runoff event results from a 25-year rainfall event).
2. The peak runoff occurs when all parts of the drainage basin are contributing to the runoff.
3. Rainfall is uniform over the watershed.
4. The peak discharge rate corresponding to a given intensity would occur only if the rainfall duration is at least equal to the time of concentration.
5. The calculated runoff is directly proportional to the rainfall intensity.
6. The runoff coefficient increases as storm frequency decreases.

The Rational Method, as presented herein, can be used to estimate peak discharges, the runoff hydrograph shape, and runoff volume for small, uniform drainage areas that are not larger than 160 acres in size. The method is usually used to size drainage structures for the peak discharge of a selected return period.

The Rational Method is based on the equation:

$$Q = C\mathrm{i}\mathrm{A} \tag{7.1}$$

where:
- Q = the peak discharge, in cfs, of the selected return period,
- C = the runoff coefficient,
- i = the average rainfall intensity, in inches/hr, of calculated rainfall duration for the selected rainfall return period, and
- A = the contributing drainage area, in acres.

7.3.2 PROCEDURE

7.3.2.1 Overview

Applicability

- The total drainage area must be less than or equal to 160 acres.
- T_c shall not exceed 60 minutes.

- The land-use of the contributing area should be fairly consistent over the entire area; that is, the area should not consist of a large percentage of two or more land-uses, such as 50 percent commercial and 50 percent undeveloped. This will lead to inconsistent estimates of T_c (and therefore i) and errors in selecting the most appropriate C coefficient.

- The contributing drainage area cannot have drainage structures or other facilities in the area that would require flood routing to correctly estimate the discharge at the point of interest.

- Drainage areas that do not meet the above conditions will require the use of the unit hydrograph method implemented through the HEC-1 Program to estimate flood discharges.

Key Analysis Parameters

A	Watershed area, in acres.
C	The Rational Method runoff coefficient.
i	The average rainfall intensity, in inches/hour.
K_b	The watershed resistance coefficient.
L	The length of the longest hydraulic flow path, in miles.
P	The rainfall depth, in inches.
Q	The estimated peak discharge, in cubic feet per second.
S	The slope of the longest hydraulic flow path, in ft/mile.
T_c	Time of Concentration, in hours.
V	The estimated runoff volume, in acre-feet.

7.3.2.2 General Considerations

1. Depending on the intended application, the runoff coefficient (C) should be selected based on the character of the existing land surface or the projected character of the land surface under future development conditions. For development projects, it will be necessary to estimate C for both existing and future conditions.
2. Land-use must be carefully considered because the evaluation of land-use will affect both the estimation of C and also the estimation of the watershed time of concentration (T_c).
3. The peak discharge (Q) is generally quite sensitive to the calculation of T_c and care must be exercised in obtaining the most appropriate estimate of T_c.
4. Both C and the rainfall intensity (i) will vary if peak discharges for different flood return periods are desired.

5. The T_c equation is a function of rainfall intensity (i); therefore, T_c will also vary for different flood return periods.

7.3.2.3 Estimation of Area (A)

An adequate topographic map of the drainage area and surrounding land is needed to define the drainage boundary and to estimate the watershed area (A), in acres. The map should be supplemented with aerial photographs, if available, especially if the area is developed. If the area is presently undeveloped but is to undergo development, the land development plan and design condition topographic maps should be used to determine the developed condition watershed boundaries.

The delineation of the drainage boundary needs to be carefully determined. The contributing drainage area for a lower intensity storm does not always coincide with the drainage area for more intense storms. This is particularly true for urban areas where roads can form a drainage boundary for small storms but more intense storm runoff can cross roadway crowns, curbs, etc. resulting in a larger contributing area. Roads on alluvial fans (active and inactive) and in distributary flow systems can result in increased contributing drainage areas during larger and more intense storms. it is generally prudent to consider the largest reasonable drainage area in such situations.

7.3.2.4 Estimation of Rainfall Intensity (i)

The intensity (i) in Equation 7.1 is the average rainfall intensity in inches/hour for the period of maximum rainfall of a specified return period (frequency) having a duration equal to the time of concentration (T_c) for the drainage area. The frequency is usually specified according to a design criteria or standard for the intended application. The rainfall intensity (i) is obtained from an intensity-duration-frequency (I-D-F) table or graph. Three methods can be used for obtaining I-D-F information as described in Section 7.2.2.2. Procedures for developing a site-specific I-D-F graph are also described in Section 7.2.2.2.

The intensity (i) in Equation 7.1 is the average rainfall intensity for rainfall of a selected return period from an I-D-F graph tor a rainfall duration that is equal to the time of concentration (T_c) as calculated according to the procedure described below. A minimum rainfall duration of 5-minutes is to be used if the calculated T_c is less than 5 minutes. The Rational Method should not be used if the calculated T_c is greater than 60 minutes.

7.3.2.5 Estimation of Time of Concentration (T_c)

Time of concentration (T_c) is to be calculated by Equation 7.2:

$$T_c = 11.4 L^{0.5} K_b^{0.52} S^{-0.31} i^{-0.38} \tag{7.2}$$

Note: Reference Papadakis and Kazan, 1987

where:
- T_c = the time of concentration, in hours,
- L = the length of the longest hydraulic flow path, in miles,
- K_b = watershed resistance coefficient,
- S = the slope of the longest hydraulic flow path, in ft/mile, and
- i = the average rainfall intensity, in inches/hr, for a duration of rainfall equal to T_c (the same (i) as Equation 7.1) unless T_c is less than 5-minutes, in which case the (i) of Equation 7.1 is for a 5-minute duration).

The longest hydraulic flow path will be estimated from the best available map and the length (L) measured from the map. This is the flow path with the longest travel time, which is not necessarily the longest in length.

The slope (S), in ft/mile, will be calculated by one of two methods:

1. If the longest flow path has a uniform gradient with no appreciable grade breaks, then the slope is calculated by Equation 7.3;

$$S = \frac{H}{L} \tag{7.3}$$

where:
- H = the change in elevation, in feet, along L, and
- L = as defined in Equation 7.2.

2. If the longest flow path does not have a uniform gradient or has distinct grade breaks, then the slope is calculated by Equation 7.4:

$$S = 5{,}280 \left(\frac{d}{j}\right)^2 \tag{7.4}$$

where:
- d = 5,280 x L, and

$$j = \sum \left(\frac{d_i^3}{H_i}\right)^{1/2} \tag{7.5}$$

Source: Pima County (1979).

and
- d_i = an incremental change in length, in feet, along the longest hydraulic flow path, and
- H_i = an incremental change in elevation, in feet, for each length segment, d_i.

General values of resistance coefficient (K_b) may be selected from Table 7.4. Refer to Section 7.13.2, Table 7.19 for specific K_b values for use in unincorporated Mohave County. Use of Table 7.4 requires a landform classification and a determination of the nature of runoff; whether in a defined drainage network of rills, gullies, channels, etc., or predominantly as overland flow. The values in Table 7.19 are specific to Mohave County zoning and land use classifications.

The solution of Equation 7.2 is an iterative process since the determination of (i) requires the knowledge of the value of T_c. Therefore, Equation 7.2 will be solved by a trial-and-error procedure. After L, K_b, and S are estimated and after the appropriate I-D-F graph is selected or prepared, a value for T_c is estimated (a trial value) and (i) is then read from the I-D-F graph for the corresponding value of duration = T_c. That (i) is then used in Equation 7.2 and T_c is calculated. If the calculated value of T_c does not equal the trial value of T_c, the process is repeated until the calculated and trial values of T_c are acceptably close (a difference of less than 10 percent is normally acceptable).

7.3.2.6 Selection of Runoff Coefficient (C)

The runoff coefficient (C) is dependent on many variables, including soil infiltration characteristics, vegetation cover, slope, land use and the percentage of impervious area of that land use, and storm frequency. The Mohave County implementation of the Rational Method is focused on two key characteristics: land use and storm frequency. The effects of the other characteristics of C are lumped into the values specified for use in Mohave County. The range of values of C provided should produce conservative results when estimating peak discharges for design of drainage facilities, which is important for public safety considerations.

General land use descriptions for Mohave County are listed in Table 7.5. These descriptions are intended to provide the necessary information to allow correlation with agencies zoning code categories. The land use codes correspond with the general land use categories listed in Table 7.6. Table 7.6 contains the range of (C) values for each general land use category by storm frequency. Refer to Section 7.13.2 for specific values for use in unincorporated Mohave County. If the area of concern is within a municipality in Mohave County, refer to that jurisdiction for guidance on (C) values specific to the jurisdictions zoning code.

Table 7.4 Resistance coefficient (K_b) for use in the Rational Method T_c Equation

Derived from: ADOT, 1994

Description of Landform	K_b	
	Defined Drainage Network	Shallow Overland Flow Only
Mountain, with forest and dense ground cover (overland slopes – 50% or greater)	0.15	0.30
Mountain, with rough rock and boulder cover, and sparse vegetation (overland slopes – 50% or greater)	0.12	0.25
Foothills (overland slopes – 10% to 50%)	0.10	0.20
Alluvial fans, Pediments and Rangeland (overland slopes – 10% or less)	0.05	0.10
Irrigated Pasture [a]	---	0.20
Tilled Agricultural Fields [a]	---	0.08
Urban		
Residential/ Commercial/Industrial, L < 1,000 ft [b]	0.04	---
Residential/ Commercial/Industrial, L > 1,000 ft [b]	0.025	---
Grass; parks, cemeteries, etc. [a]	---	0.20
Bare Ground; playgrounds, etc. [a]	---	0.08
Paved; parking lots, etc. [a]	---	0.02

Notes:

a – No defined drainage network.

b – L is the length in the T_c equation (eqn 7.2). Roadways serve as drainage network.

The values in Section 7.13.2 are the default values used in the DDMSW computer program. The user may select a non-default value provided the selection is within the range of values provided in Table 7.6, and engineering justification is provided and approved by Mohave County or the controlling jurisdiction.

The user should delineate sub-basins around areas with a single land use, where possible. When that is not practical, then an area-averaged (C) value should be computed for the sub-basin using Equation 7.6.

$$C_{comp} = \left(\sum_{i=1}^{n} C_i A_i \right) \Big/ A_t \qquad 7.6$$

where:

C_{comp}	=	the area-averaged value of (C),
n	=	the number of different land use polygons within the sub-basin,
C_i	=	the value of (C) corresponding to each land use in the sub-basin,
A_i	=	the area in acres of the corresponding land use within the sub-basin, and
A_T	=	the total area of the sub-basin in acres.

Table 7.5 General land use category descriptions for Mohave County

Land Use Code	Land Use Category Description
VLDR	Residential: 40,000 square feet and greater lot size
LDR	Residential: 12,000 - 40,000 square feet lot size
MDR	Residential: 6,000 - 12,000 square feet lot size
MFR	Residential: 1,000 - 6,000 square feet lot size
C1	Commercial: Light, Neighborhood, Residential
C2	Commercial: Central, General, Office, Intermediate
I1	Industrial: Light to General
I2	Industrial: General to Heavy
P	Paved Areas: asphalt and concrete, sloped rooftops
GR	Graded Areas: graded and compacted, treated and untreated
AG	Agricultural: tilled fields, irrigated pastures, slopes < 1%
LP1	Landscaped Park 1: *RTIMP* = 0-10%, irrigated vegetation = 0-20%
LP2	Landscaped Park 2: *RTIMP* = 0-10%, irrigated vegetation = 20-80%
LP3	Landscaped Park 3: *RTIMP* = 0-10%, irrigated vegetation >80%
L1	Landscaping 1: with impervious under treatment
L2	Landscaping 2: without impervious under treatment
NDR	Natural Desert Rangeland: little topographic relief, slopes < 5%
NHS	Natural Hillslope: moderate topographic relief, slopes > 5%
NMT	Natural Mountain: high topographic relief slopes, slopes > 20%

Table 7.6 Rational Method general runoff coefficients for Mohave County

Derived from ADOT (1993) and Maricopa County (2008)

Land Use Code	Land Use Category	Runoff Coefficients by Storm Frequency							
		2-10 Year		25-year		50-year		100-year	
		min	max	min	max	min	max	min	max
VLDR	Very Low Density Residential[1]	0.33	0.45	0.36	0.50	0.40	0.60	0.45	0.65
LDR	Low Density Residential[1]	0.42	0.48	0.46	0.55	0.50	0.64	0.53	0.70
MDR	Medium Density Residential[1]	0.48	0.65	0.53	0.72	0.58	0.78	0.60	0.80
MFR	Multiple Family Residential	0.65	0.75	0.72	0.83	0.78	0.90	0.82	0.94
C1	Commercial 1	0.55	0.65	0.61	0.72	0.66	0.78	0.69	0.81
C2	Commercial 2	0.75	0.85	0.83	0.94	0.90	0.95	0.94	0.95
I1	Industrial 1	0.60	0.70	0.66	0.77	0.72	0.84	0.75	0.88
I2	Industrial 2	0.70	0.80	0.77	0.88	0.84	0.95	0.88	0.95
P	Pavement and Rooftops	0.75	0.85	0.83	0.94	0.90	0.95	0.94	0.95
GR	Gravel Roadways & Shoulders	0.60	0.70	0.66	0.77	0.72	0.84	0.75	0.88
AG	Agricultural	0.10	0.20	0.11	0.22	0.12	0.24	0.13	0.25
LP1	Landscaped Park 1	0.30	0.45	0.35	0.48	0.36	0.50	0.38	0.55
LP2	Landscaped Park 2	0.20	0.35	0.25	0.40	0.30	0.45	0.35	0.50
LP3	Landscaped Park 3	0.10	0.25	0.11	0.28	0.12	0.30	0.15	0.35
L1	Desert Landscaping 1	0.55	0.85	0.61	0.94	0.66	0.95	0.69	0.95
L2	Desert Landscaping 2	0.30	0.40	0.35	0.45	0.36	0.48	0.38	0.50
NDR	Undeveloped Desert Rangeland	0.30	0.40	0.35	0.45	0.36	0.48	0.38	0.50
NHS	Hillslopes, Sonoran Desert	0.40	0.55	0.45	0.60	0.48	0.66	0.50	0.70
NMT	Mountain Terrain	0.50	0.70	0.65	0.80	0.70	0.90	0.75	0.90
1	Based on NDR (undeveloped desert rangeland) terrain class. Values should be increased for NHS and NMT terrain classes by the difference between NHS or NMT and the NDR C values, up to a maximum of 0.95. Engineering judgment should be used.								

7.3.2.7 Volume Calculations

Rational method runoff volume estimations should be computed using Equation 7.7. In the case of volume calculations for stormwater storage facility design, *P* in Equation 7.7 equals the 100-year, 2-hour depth, in inches. Refer to Section 7.2.

$$V = C\left(\frac{P}{12}\right)A \qquad 7.7$$

Source: Maricopa County (2008)

where: V = runoff volume, in acre-feet,

C = runoff coefficient (or C_{comp}),

P = rainfall depth, in inches, and

A = drainage area, in acres.

7.3.2.8 Runoff Hydrographs

If a runoff hydrograph is needed for design purposes, the Rational Method should not be used. The Unit Hydrograph Method (Section 7.5) should be used for that case.

7.3.3 Application

The Rational Method can be used to calculate the generated peak discharge from drainage areas less than 160 acres. Procedures for calculating peak discharge for single and multiple sub-basins are provided in the following sections.

7.3.3.1 Instructions for Single Basin Approach

1. Area: Determine the area of the watershed in question using a suitable topographic map. Define the land uses present in each sub-basin and compute the area of each.
2. C: Select the Runoff Coefficient (C) from Table 7.19. If the drainage area contains subareas of different runoff characteristics, and thus different C coefficients, arithmetically area-weight the values of C using Equation 7.6.
3. T_c Parameters.
   a. L: Determine the T_c flow path and measure the value of L.
   b. S: Inspect the T_c path on the topographic map of the watershed and assess if there are significant changes in slope along the T_c path. If there are, plot a profile of the T_c path and determine the break locations. Compute an adjusted slope using Equation 7.4. If the profile is uniform, use Equation 7.3. The slope cannot be adjusted by calculation within DDMSW. If DDMSW is being used, and the slope needs to be adjusted, compute the adjusted slope manually using Equation 7.4 then enter the adjusted slope directly into DDMSW.
   c. K_b: Determine the K_b parameter from Table 7.19. If the drainage area contains subareas of different K_b values, arithmetically area-weight the values of K_b using Equation 7.6 and substituting K_b for C.
   d. I-D-F: Tabulate the depth-duration-frequency (D-D-F) statistics for the project site using the figures in Appendix B.1 through B.6 on the form in Appendix B.7. Compute

the Intensity-Duration-Frequency (I-D-F) data and tabulate and plot the results using the form and graph paper in Appendix B.7.

4. T_c: Calculate the time of concentration. This is to be done as an iterative process.

   a. Make an initial estimate of the duration and read the corresponding intensity i from the I-D-F curve plot for the desired frequency. The initial estimate of duration can be made by assuming an average velocity in feet per second and dividing it into L in feet, and converting the result to minutes.

   b. Compute an estimated T_c using Equation 7.2. If the computed T_c is reasonably close to the estimated duration (i.e. within 10%), then proceed to Step 5; otherwise, repeat this step with a new estimate of the duration. T_c should not be less than 5-minutes or more than 60-minutes. If the final T_c is greater than 60-minutes, re-delineate the watershed into smaller sub-basins or use the Unit Hydrograph Method. Use the final T_c as the duration and read a final value of i from the I-D-F plot.

5. Q: Determine the peak discharge Q by using the above value of i in Equation 7.1.

6. As an alternative to the above procedure, the DDMSW program may be used to calculate peak discharge.

7.3.3.2 Multiple Basin Approaches

The Rational Method can be used to compute peak discharges at intermediate locations within a drainage area less than 160 acres in size. A typical application of this approach is a local storm drain system where multiple sub-basins are necessary to compute a peak discharge at each proposed inlet location. Consider the schematic example watershed shown in Figure 7.8. A peak discharge is needed for all three individual sub-basins, sub-basins A and B combined at Concentration Point 1 and sub-basins A, B and C combined at Concentration Point 2.

There are two accepted methods in Mohave County for computing peak discharges for multiple basins using the Rational Method. The first method is the traditional approach that relies upon combining the sub-basin areas into a single watershed, computing a new T_c, an arithmetically area-weighted value of C for the combined sub-basins, and then computing the peak discharge. This approach is referred to as the “Combined Watershed Approach.” The second method is the “Triangular Hydrograph Approach.” A triangular hydrograph is created for each sub-basin where the time-to-peak is assumed equal to T_c and the hydrograph time base is equal to $2.67T_c$, as shown on Figure 7.9. Referring to Figure 7.8 for example, the ordinates of hydrographs for sub-basins A and B at CP 1 are added to obtain the total flow hydrograph at CP 1. That hydrograph is then lagged downstream to CP 2 by the estimated travel time in the roadway, pipe, or channel. The lagged hydrograph is then added to the sub-basin C hydrograph to obtain the peak discharge at CP 2. Only the Triangular Hydrograph Method is implemented in the DDMSW computer program. The Combined Hydrograph Method may be used when the engineer/hydrologist does not have access to a computer running DDMSW.

Figure 7.8 Example watershed

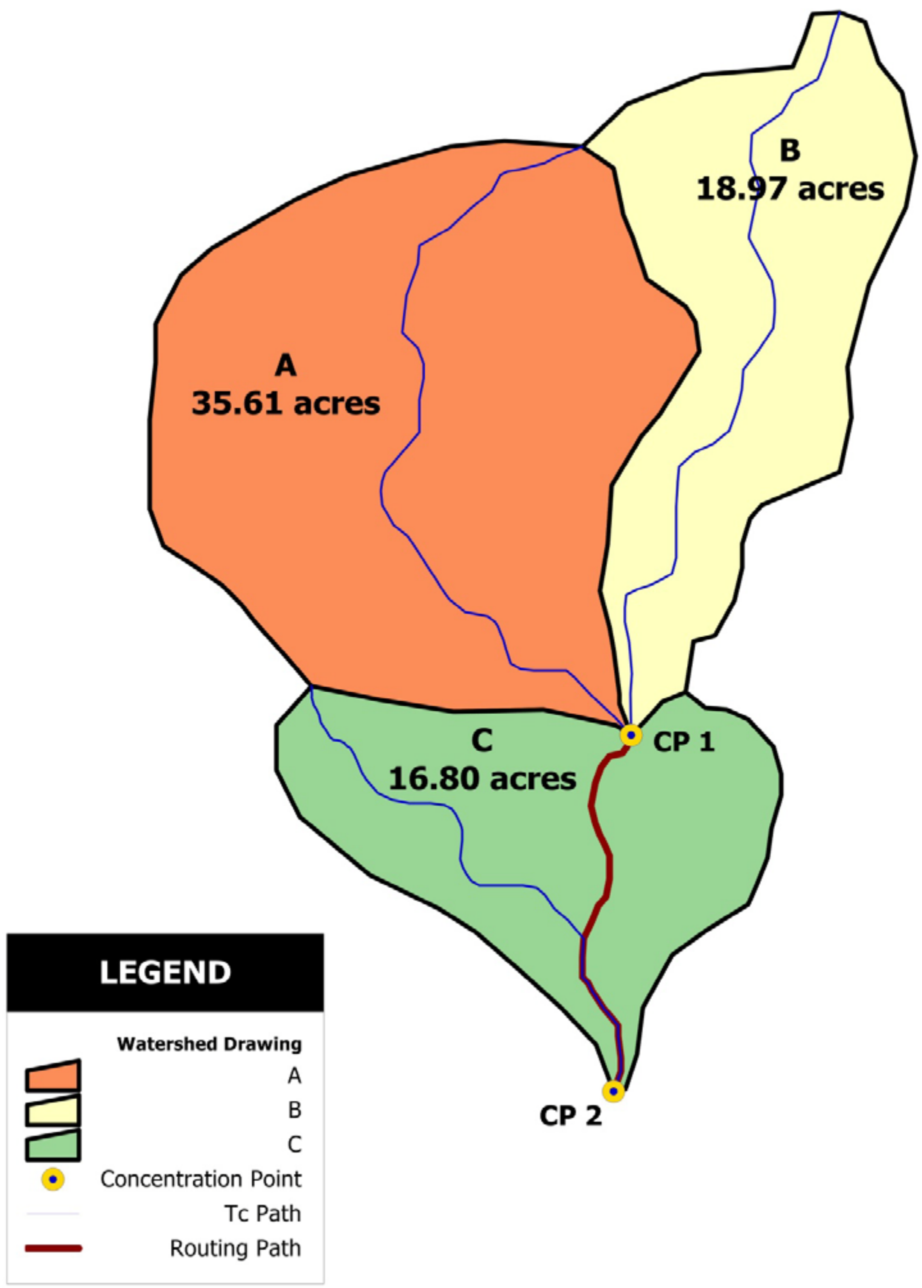
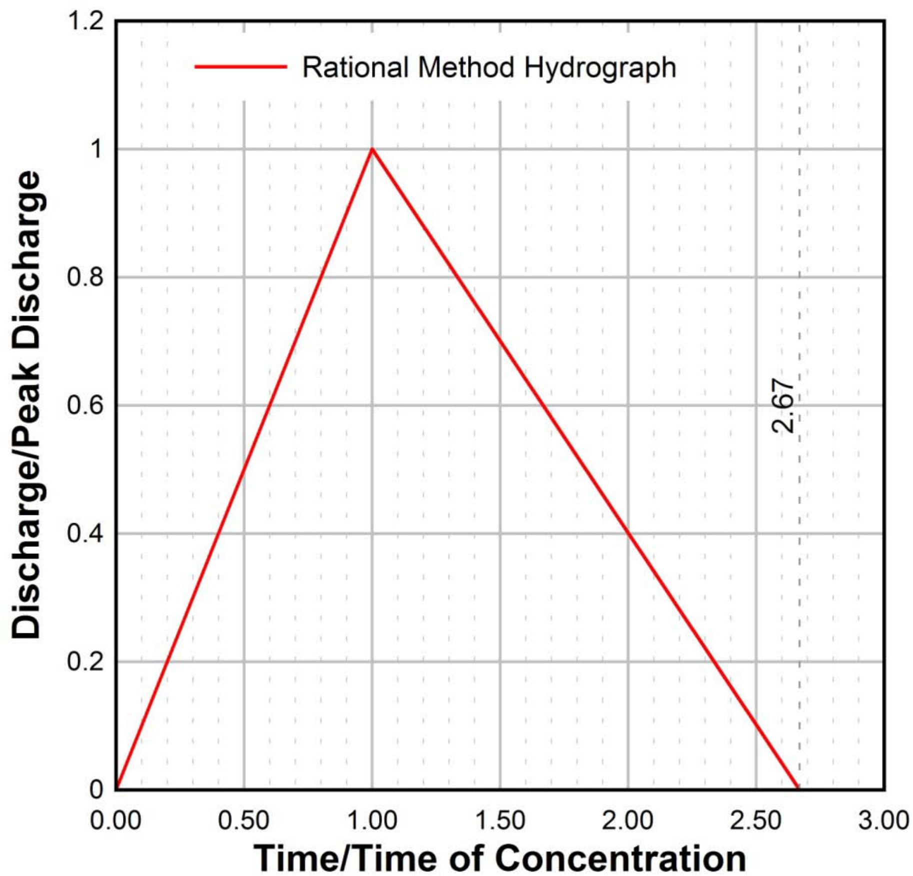


Figure 7.9 Triangular hydrograph for use with the Rational Method

Derived from: *Highway Hydrology* (FHWA, 2002)

(modified based on analysis of urban flow gage hydrograph data for short duration high intensity storms on watersheds less than 160 acres in Maricopa County)



Instructions for Combined Watershed Approach

1. Compute the peak discharge for each individual sub-basin using steps 1 through 5 from Section 7.3.3.1.
2. Compute the arithmetically area-weighted value of *C* for combined sub-basins A and B.
3. Follow step 4 from Section 7.3.3.1 to calculate the T_c for the combined area of sub-basins A and B at CP 1.

4. Compare the T_c values from sub-basins A and B to the T_c value for the combined area at CP 1. Compute the peak discharge at CP 1 using the i for the longest T_c from step 3. If the combined peak discharge is less than the discharges for the individual sub-basins, use the largest discharge as the peak discharge at CP 1. The design peak discharge SHOULD NOT DECREASE going downstream in a conveyance system unless storage facilities are used to attenuate peak flows.

   NOTE: If there are more than two watersheds being combined, and the combined peak discharge is less than any of the individual sub-basin peak discharges, another check needs to be made. A long narrow watershed having a long T_c may not be representative of the majority of the combined watershed and could be the reason the combined sub-basin peak discharge is too low. A combination of the other sub-basins may be more appropriate, using a computed T_c for the new combination. The T_c cannot be greater than 60 minutes.

5. Compute the arithmetically area-weighted value of C for combined sub-basins A, B and C.

6. Calculate the T_c for the combined area at CP 2 using the following two methods (T_c cannot be greater than 60 minutes):

   a. Method 1 - Follow step 4 from Section 7.3.3.1 to calculate the T_c for the single basin composed of all three sub-basins.

   b. Method 2 - Compute the travel time from CP 1 to CP 2 using the Manning equation or other appropriate technique and hydraulic parameters for the conveyance path. Add the computed travel time for the conveyance path to the T_c from CP 1.

7. Using the T_c values from Methods 1 and 2 as well as the T_c from sub-basin C, calculate the peak discharge at CP 2 as follows:

   a. If the T_c value from Method 1 is the longest, compute the total peak discharge using the Method 1 intensity, the arithmetically area-weighted value of C for all three subareas and the total contributing drainage area at CP 2.

   b. If the T_c value from Method 2 is the longest, determine i directly from the D-D-F statistics from step 3 of Section 7.3.3.1. Compute the total peak discharge at CP 2 using the arithmetically area-weighted value of C for all three sub-basins and the total contributing drainage area at CP 2.

   c. If the T_c from subarea C is the longest, compute the total peak discharge using the i for sub-basin C, the arithmetically area-weighted value of C for all three sub-basins and the total contributing drainage area at CP 2.

Instructions for Triangular Hydrograph Approach

1. Compute the peak discharge for each individual sub-basin using steps 1 through 5 from Section 7.3.3.1.

2. Plot triangular hydrographs for sub-basins A and B on a single sheet of graph paper using the dimensionless triangular hydrograph shown in Figure 7.9 as the model. The peak discharge occurs at time T_c and the hydrograph time base is 2.67T_c.

3. Add the hydrograph ordinates from sub-basins A and B to produce and plot a combined hydrograph at CP 1.

4. Compute the travel time from CP 1 to CP 2 using the Manning equation or other appropriate technique and hydraulic parameters for the conveyance path.
5. Plot the hydrograph for sub-basin C on a new piece of graph paper, starting at time = 0.0. Plot the hydrograph for CP 1 starting at time = travel time from CP 1 to CP 2.
6. Add the hydrograph ordinates from CP 1 and sub-basin C to produce and plot a combined hydrograph at CP 2.
7. As an alternative to the above procedure, the DDMSW program may be used to calculate the peak discharge at intermediate locations.

7.3.3.3 Example

Refer to Appendix A.2 for an example of application of the Rational Method.

7.4 RAINFALL LOSSES

7.4.1 INTRODUCTION

Rainfall excess is that portion of the total rainfall depth that drains directly from the land surface by overland flow. By a mass balance, rainfall excess plus rainfall losses equals total rainfall.

This chapter is only applicable when performing rainfall runoff modeling with the HEC1 program. The design rainfall is determined from the procedures in Section 7.2, and this chapter provides procedures to estimate the runoff from the applied rainfall. When using the Rational Method, it is not necessary to estimate rainfall losses by the procedures in this chapter because the C factor accounts for the effect of rainfall loss on the peak discharge and runoff volume.

One of two methods shall be used to estimate rainfall losses; the primary method is to be used for the majority of cases, and the secondary method is to be used only for special cases when it is determined that the primary method is inappropriate. The primary method requires the estimation of the rainfall infiltration loss by the Green and Ampt equation. This model, first developed in 1911 by W.H. Green and G.A. Ampt, has since the early 1970's received increased interest for estimating rainfall infiltration losses. A sound and concise explanation of the Green and Ampt equation is provided by Bedient and Huber (1988) and Chow, Maidment and Mays (1988).

The secondary method requires the estimation of the initial loss and a uniform loss rate (IL+ULR method). The secondary method is to be used for watersheds or sub-basins where rainfall losses are known to be controlled by factors other than soil texture and vegetation cover, or for watersheds that are predominantly composed of sand or volcanic cinder overlain by forest duff where the Green and Ampt equation is not appropriate. Infiltration is not controlled by soil texture in such watersheds and infiltration rates may be as high as 5 inches per hour or more. Use of the secondary method requires adequate data or appropriate studies to verify the IL+ULR parameters or to calibrate the model of the watershed.

Both methods are described in detail in the following sections.

7.4.2 GREEN AND AMPT METHOD

7.4.2.1 General

Use of the Green and Ampt equation as coded in HEC-1 involves the simulation of rainfall loss as a two phase process, as illustrated in Figure 7.10. The first phase is the simulation of the surface retention loss. This loss is called the initial loss (IA) in HEC-1. During this first phase, all rainfall is lost (zero rainfall excess generated) during the period from the start of rainfall up to the time that the accumulated rainfall equals the value of IA. It is assumed, for modeling purposes that no infiltration of rainfall occurs during the first phase. The second phase of the rainfall loss process is the infiltration of rainfall into the soil matrix. For modeling purposes, the infiltration begins immediately after the surface retention loss (IA) is completely satisfied, as illustrated in Figure 7.10.

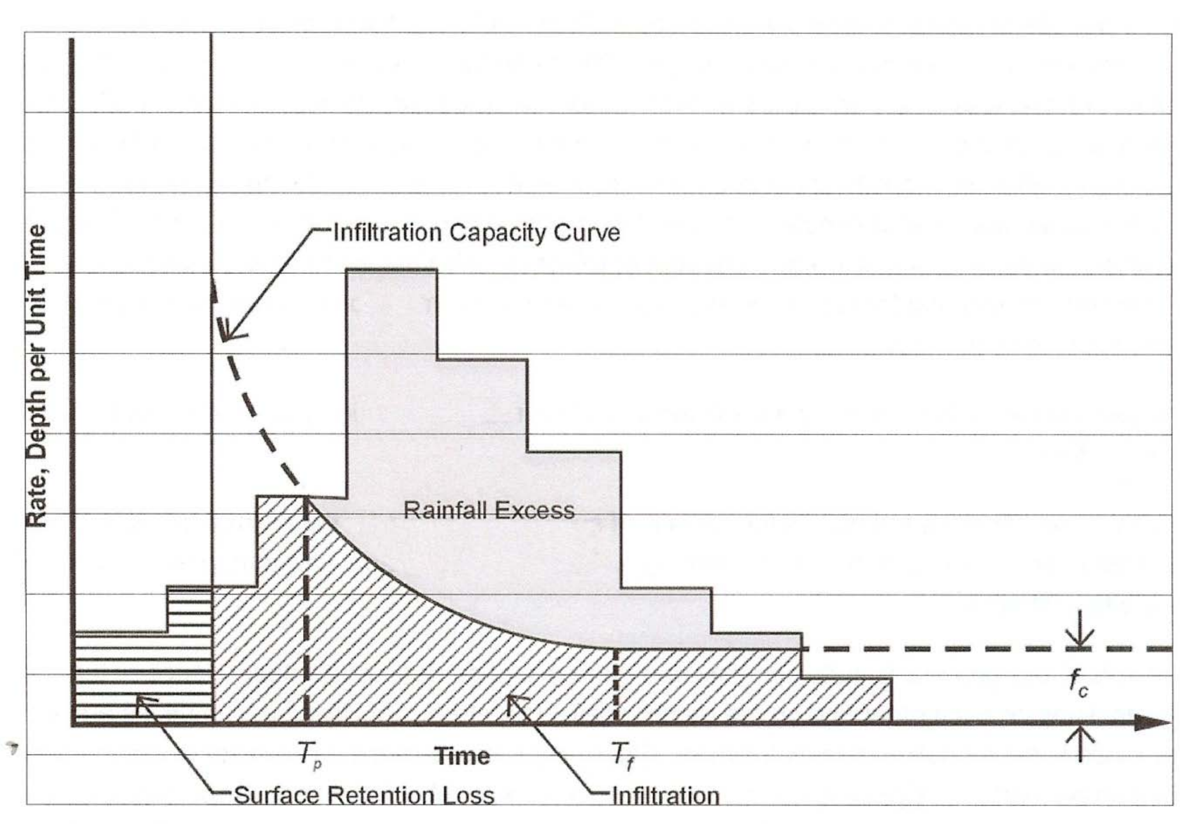
7.4.2.2 Applicability

The Green and Ampt infiltration equation, along with an estimate of the surface retention loss can be used to estimate rainfall losses for most areas of Mohave County with confidence. Most soils in Mohave County are loamy sand, sandy loam, loam, or silt loam for which the Green and Ampt infiltration equation parameters should apply. Silt, as a soil texture, is relatively rare and it is not expected that significant areas will be encountered. The finer soil textures (those with 'clay" in the classification name) occur in Mohave County but not usually over large areas; however, these soils have relatively low infiltration rates (*XKSAT*). Use of the Green and Amp! Infiltration equation parameters for the finer soil textures may be somewhat conservative, and therefore their use should be appropriate for most design flood estimation purposes. Sand, as a soil texture, is also relatively rare and it has a very high infiltration rate (*XKSAT*). Therefore, when encountering large areas that have soils that are classified as sand, it is possible that estimates of rainfall losses with the Green and Ampt Equation would be too large and the IL+ULR method should be used.

In general, the Green and Ampt Infiltration equation with an estimate of the surface retention loss should be used for most drainage areas in Mohave County. The IL+ULR method should be used for drainage areas where soil texture does not control the infiltration rate (such as volcanic cinder) or where the soil texture of the drainage area is predominantly sand. Calibration data or results of regional studies are necessary to justify the selection of parameters tor the IL+ULR method.

The Green and Ampt equation should be applied for the one percent storm (100-year) and more frequent storms. This method is also suitable for use with less frequent storms but the parameters IA and volumetric storage of infiltrated rainfall may not be significant for large rainfall amounts associated with storms less frequent than the one percent storm.

Figure 7.10 Simplified representation of rainfall losses (a function of surface retention losses plus infiltration)



7.4.2.3 Method Description

The first phase of the rainfall loss process is simulated with the Surface retention loss (IA) parameter. The second phase is simulated with the Green and Ampt equation. Implementation of both phases of the rainfall loss process for rainfall runoff modeling in Mohave County is discussed in the following sections.

Surface Retention Loss, IA

Surface retention loss, as used herein, is the summation of all rainfall losses other than infiltration. The major component of the surface retention loss is depression storage; relatively minor components of surface retention loss are due to interception and evaporation. Depression storage is considered to occur in two forms. First, in-place depression storage occurs at, and in the near vicinity of, the raindrop impact. The mechanism for this depression storage is the micro relief of the soil and soil cover. The second form of depression storage is the retention of surface runoff that occurs away from the point of the raindrop impact in surface depressions such as puddles, roadway gutters and swales, roofs, irrigation bordered fields and lawns, and so forth. A relatively minor contribution by interception is also considered as a part of the total surface retention loss.

Estimates of surface retention loss are difficult to obtain and are a function of the physiography and land-use of the area. The surface retention loss on an impervious surface has been estimated to be in the range 0.0625 inch to 0.125 inches by Tholin and Keefer (1960), 0.11 inches for 1 percent slopes to 0.06 inches for 2.5 percent slopes by Viessman (1967), and 0.04 inches based on rainfall-runoff data for an urban watershed in Albuquerque by Sabol (1983). Hicks (1944) provides estimates of surface retention losses during intense storms as 0.20 inches for sand, 0.15 inches for loam, and 0.10 inches for clay. Tholin and Keefer (1960) estimated the surface retention loss for turf to be between 0.25 and 0.50 inches. Based on rainfall simulator studies on undeveloped alluvial plains in the Albuquerque area, the surface retention loss was estimated at 0.1 to 0.2 inches (Sabol and others, 1982a). Rainfall simulator studies in New Mexico result in estimates of 0.39 inches for eastern plains rangelands and 0.09 inches for pinon-juniper hillslopes (Sabol and others, 1982b). Chow (1964) quotes Horton (1935) as stating that initial detention (IA) “commonly ranges from 1/8 to 3/4 inch for flat areas and 1/2 to 1.5 inches for cultivated fields and for natural grass lands or forests.” Further research for estimating values of IA for various land uses and land surfaces is needed. All known reference sources for values of IA for use with the Green and Ampt equation are listed above and have been used in developing the data listed in Table 7.7.

IA is primarily a function of land-use and surface cover, and recommended values of IA for use with the Green and Ampt equation are presented in Table 7.7. For example, about 0.25 inches of rainfall will be lost to runoff due to surface retention for mountain terrain where there is high topographic relief (slopes > 10%) in Mohave County. For unincorporated Mohave County, standard values for the natural and developed condition IA for each zoning category is provided in Table 7.20. The engineer or hydrologist should, in general, use the standard default values from Table 7.20, which are also built into the DDMSW computer program. The engineer/hydrologist should first examine the watershed characteristics to be sure the default values are appropriate.

<table>
<tr><th colspan="5">Table 7.7 IA and RTIMP estimates for various land uses
(Source: Derived from ADOT, 1993; FCDMC, 2009a)</th></tr>
<tr><th colspan="2" rowspan="2">Land-use and/or Surface Cover
(1)</th><th rowspan="2">Surface Retention Loss (IA), inches
(2)</th><th colspan="2">RTIMP, percent</th></tr>
<tr><th>Mean
(3)</th><th>Range
(4)</th></tr>
<tr><td colspan="5">Natural</td></tr>
<tr><td colspan="2">Natural grasslands (flat slope)</td><td>0.50</td><td></td><td></td></tr>
<tr><td colspan="2">Rangeland, flat slope
(moderate vegetation)</td><td>0.35</td><td>varies</td><td>varies</td></tr>
<tr><td colspan="2">Rangeland, hill slopes
(moderate vegetation)</td><td>0.15</td><td>varies</td><td>varies</td></tr>
<tr><td colspan="2">Mountain, flat slope (vegetated)</td><td>0.50</td><td>varies</td><td>varies</td></tr>
<tr><td colspan="2">Mountain, steep slopes (vegetated)</td><td>0.25</td><td>varies</td><td>Varies</td></tr>
<tr><td colspan="5">Developed (Residential and Commercial)</td></tr>
<tr><td rowspan="5">Single Family Residential</td><td>1/4 acre</td><td>0.25</td><td>40</td><td>25-55</td></tr>
<tr><td>1/3 acre</td><td>0.25</td><td>30</td><td>20-40</td></tr>
<tr><td>1/2 acre</td><td>0.25</td><td>23</td><td>15-30</td></tr>
<tr><td>1 acre</td><td>0.30</td><td>18</td><td>10-25</td></tr>
<tr><td>>=2 acres</td><td>0.30</td><td>15</td><td>5-25</td></tr>
<tr><td colspan="2">Multi-Family Residential</td><td>0.25</td><td>50</td><td>40-60</td></tr>
<tr><td colspan="2">Commercial</td><td>0.10</td><td>75</td><td>50-95</td></tr>
<tr><td colspan="2">Industrial</td><td>0.20</td><td>70</td><td>50-90</td></tr>
<tr><td colspan="2">Non-irrigated Landscape</td><td>0.10</td><td>varies</td><td>varies</td></tr>
<tr><td colspan="2">Lawn and Turf</td><td>0.20</td><td>0</td><td>0</td></tr>
<tr><td colspan="2">Pavement and Roof Tops</td><td>0.05</td><td>95</td><td>95</td></tr>
<tr><td colspan="5">Agricultural</td></tr>
<tr><td colspan="2">Tilled fields irrigated pasture</td><td>0.50</td><td>0</td><td>0</td></tr>
</table>

Green and Ampt Parameters

The second phase of the rainfall loss process is simulated with the Green and Ampt equation and an estimate of watershed impervious area (*RTIMP*). The three Green and Ampt equation infiltration parameters as coded in HEC-1 are:

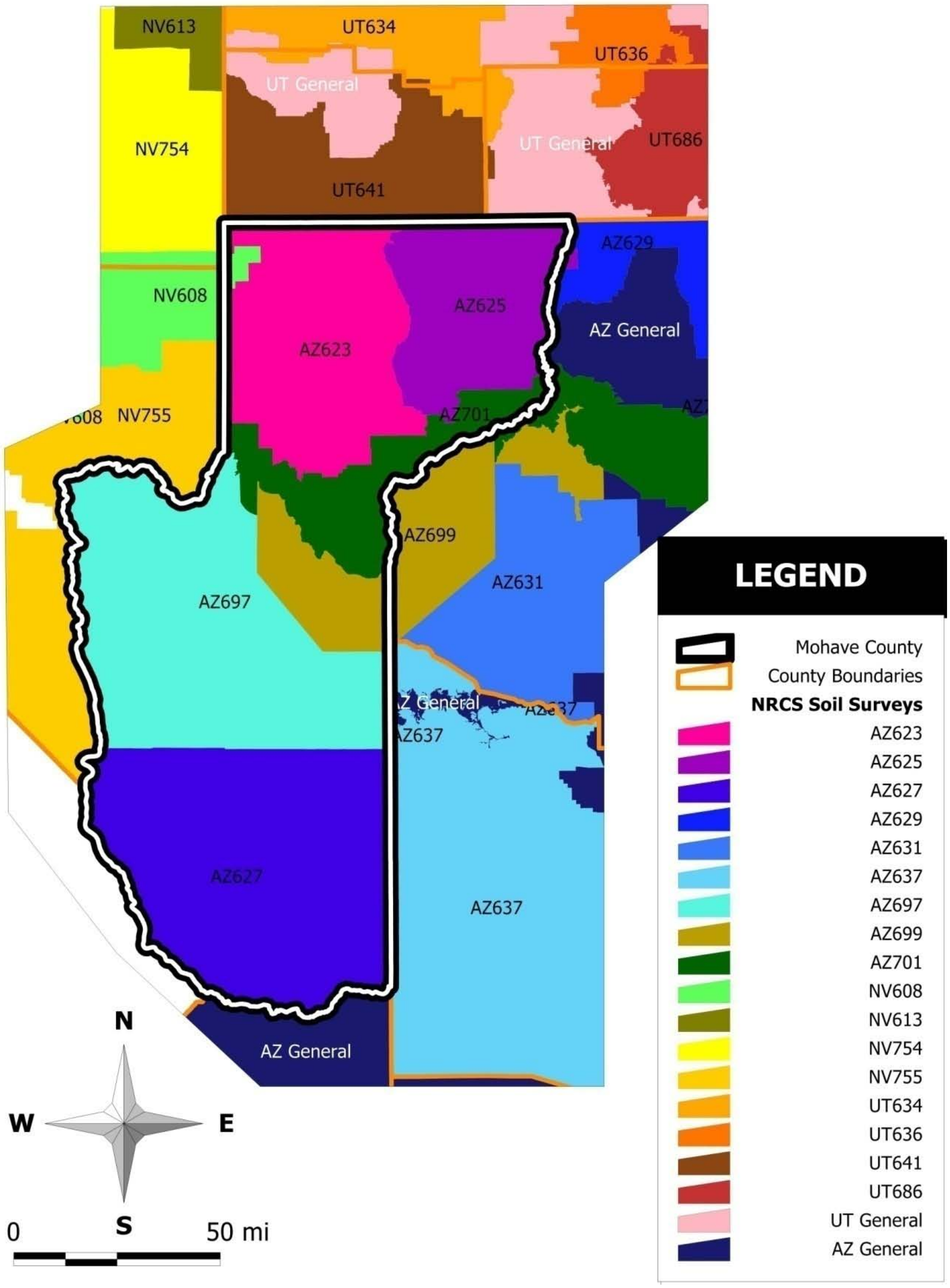
1. hydraulic conductivity at natural saturation (*XKSAT*);
2. wetting front capillary suction (*PSIF*); and
3. volumetric soil moisture deficit at the start of rainfall (*DTHETA*).

The three infiltration parameters are functions of soil characteristics, ground surface characteristics, and land management practices.

The soil characteristics of interest are particle size distribution including the percentage of sand and clay, gravel fractions, organic matter, and porosity. The primary soil surface characteristics are vegetation canopy cover, ground cover, and soil crusting.

Values of Green and Ampt equation parameters as a function of soil characteristics alone (bare ground condition) have been computed for soil map units from soil surveys prepared by the Natural Resources Conservation Service (NRCS) (http://soildatamart.nrcs.usda.gov/Default.aspx). Refer to Figure 7.11 for the spatial location of the soil surveys used. A soil map unit, as defined in the Soil Taxonomy handbook (NRCS, 1999) is an individual polygon identified on a soil map by a map unit symbol and/or name that defines a three-dimensional soil body of a specified area, shape, and location on the landscape. The three-dimensional soil body is an aggregate of all soil delineations in a soil survey area that have a defined set of similar soil characteristics.

Figure 7.11 NRCS soil surveys used to estimate Green and Ampt parameters



GIS coverages of soil survey maps that define the geospatial location, limits and characteristics of soil map units for Mohave County and watersheds that drain into Mohave County, were obtained. Physical characteristics and soil properties available in database format as a part of the GIS coverages were used to compute average Green and Ampt parameters for each soil map unit. The computations were done using the equations and methods set forth in Saxton and Rawls (2005). These equations are based on values of percent sand, clay, gravel, organic matter, and porosity rather than a general texture classification as used in ADOT (1993) and the FCDMC (2009a).

The Saxton and Rawls (2005) equations produce a true saturated hydraulic conductivity, K_s. The Green and Ampt equation uses *XKSAT*, which is hydraulic conductivity at natural saturation. K_s must be adjusted to compute an estimate of *XKSAT* using Equation 7.8. The correction factor is necessary to adjust for entrapped air.

$$XKSAT = \mathrm{CF} * \mathrm{K_s} \tag{7.8}$$

Two sources are available for estimating the correction factor *CF*. The first is Bouwer (1966), which recommends a value of 0.5. The second is Morel-Seytoux and Khanji (1974), who furthered Bouwer's work and introduced a correction factor that varies with soil type and ponding depth, ranging from 0.6 to 0.9, with an average of 0.7. The average correction factor by Morel-Seytoux and Khanji (1974) is accepted for use in Mohave County.

The Saxton and Rawls (2005) equations include a compaction adjustment. For Mohave County, soils in an undisturbed natural condition are assumed to be at a compaction rating of "normal", which corresponds to a density factor (*DF*) of 1.0. Soils within developed areas that have been graded and re-compacted, or are subject to compaction from human activities, are assumed to be at a compaction rating of "dense", which corresponds to a *DF* of 1.1. Two separate values for K_s and *XKSAT* were computed for each SMU using the density factors. $XKSAT_N$ refers to the natural condition *XKSAT*, and $XKSAT_D$ is the developed condition *XKSAT*. $XKSAT_N$ and $XKSAT_D$ are assigned by land use as listed in Table 7.20.

Refer to Appendix D for a description of the data sources and procedures used to compute the soil map unit Green and Ampt parameters for use in Mohave County. A single GIS polygon soils coverage for Mohave County was prepared and is available for download from the Mohave County web site at: http://gis.co.mohave.az.us/. This GIS coverage can be used as a stand-alone source for the Green and Ampt parameter *XKSAT*, or the Mohave County version of the

DDMSW computer program, which has this coverage built-in, can be used. Refer to Figure 7.12 for a map depicting the spatial variation of *XKSAT* for Mohave County watersheds. The *XKSAT* for all loamy sands and sands is set at a maximum of 2.0 inches/hour.

There are a number of soil map unit component soil types within the NRCS soil surveys for which values of the percentage of sand and clay are not provided. A soil texture was assigned using methods described in Appendix D.2.5 and then the value of *XKSAT* for the assigned texture was taken from Table 7.8. Where insufficient information is available for assigning a soil texture, conservative values of *XKSAT* were assigned. The engineer/hydrologist should check for the presence of any miscellaneous component soil types as the conservative assumption for *XKSAT* could significantly affect model results. For these cases, the areas where the miscellaneous soil types occur should be physically inspected and the default value of *XKSAT* adjusted to match observed conditions if necessary. A geotechnical investigation may need to be performed to obtain representative soil physical characteristics, including the percentages of sand, clay, gravel and organic matter. The Saxton and Rawls (2005) *Soil Water Characteristics* computer program may then be used to compute the value of K_s, which must then be corrected as described above to estimate XKSAT. The miscellaneous component soils requiring field verification are listed in Appendix D, Table D.4 with the NRCS soil surveys they occur in. The assumptions made for each miscellaneous component soil are listed in Appendix D, Table D.5. The general location of miscellaneous component soils is shown on Figure 7.13. Rock outcrop areas are also treated as miscellaneous component soils since soil parameters are also not provided for rock out crop component soil types. Rock outcrop is not shown on Figure 7.13. Refer to Figure 7.17. SMU's with rock outcrop should be carefully evaluated for reasonableness for the watershed being studied and estimates of rock outcrop field verified.

PSIF and *DTHETA* are assumed to be a function of *XKSAT*, as discussed in Appendix D. A value of *PSIF*, $DTHETA_{dry}$ and $DTHETA_{normal}$ were computed for every component soil and every component soil horizon used in the preparation of the Mohave County GIS soil coverage. A nonlinear regression analysis was then performed to provide equations for computing *PSIF* and *DTHETA* as a function of *XKSAT*. The equations computed are:

$$PSIF = 1/(0.06149 - 0.03544 * XKSAT + 0.37264 * XKSAT^2) \qquad 7.9$$

$$DTHETA_{Dry} = 0.35174 + 0.03787 * \log_e XKSAT \qquad 7.10$$

$$DTHETA_{Normal} = 0.26309 * XKSAT^{0.31813} \qquad 7.11$$

These equations are plotted on Figure 7.14 and Figure 7.15. The average values of *XKSAT* and *PSIF* for each of the twelve general soil texture classes are shown in columns (2) and (3) of Table 7.8. Equations 7.9, 7.10, and 7.11 or Figure 7.14 and Figure 7.15 should be used for computation or selection of values of *PSIF* and *DTHETA* based on bare ground *XKSAT*. The values of *XKSAT* and *PSIF* from Table 7.8, Figure 7.14 and Figure 7.15 could be used for smaller studies being done by hand without the benefit of DDMSW or GIS; however, the use of DDMSW is recommended.

The *XKSAT* and *PSIF* parameter values that are shown in Table 7.8 for loamy sand and sand are very high. Using those parameters values for drainage areas can result in the generation of no rainfall excess which may or may not be correct. Incorrect results could cause serious consequences for flood control planning and design. Therefore, it is recommended to use a XKSAT equal to 2.0 inches/hour for watersheds consisting of relatively small subareas of sand. PSIF and DTHETA should be determined using Figure 7.14 and Figure 7.15, respectively. If the area contains a large portion of sand, either the Green and Ampt method should be used with the stated parameter values or the Initial and Uniform Loss method should be used with the appropriately determined values for the parameters.

The methods presented herein can be accomplished using hand computation techniques including hard copy maps and use of a planimeter or other suitable method for estimating the areas of irregular polygons. The DDMSW computer program is the preferred method. DDMSW can be used for detailed studies where GIS polygons for the watershed sub-basins are available, and for relatively small areas by selecting the watershed location using maps provided within DDMSW and letting it estimate average Green and Ampt parameters. More complex watersheds with detailed soils mapping including numerous polygons defining multiple soil textures may, from a practical standpoint, require analysis using DDMSW in combination with ESRI ArcMap GIS software.

Figure 7.12 Spatial variation of *XKSAT* for Mohave County watersheds

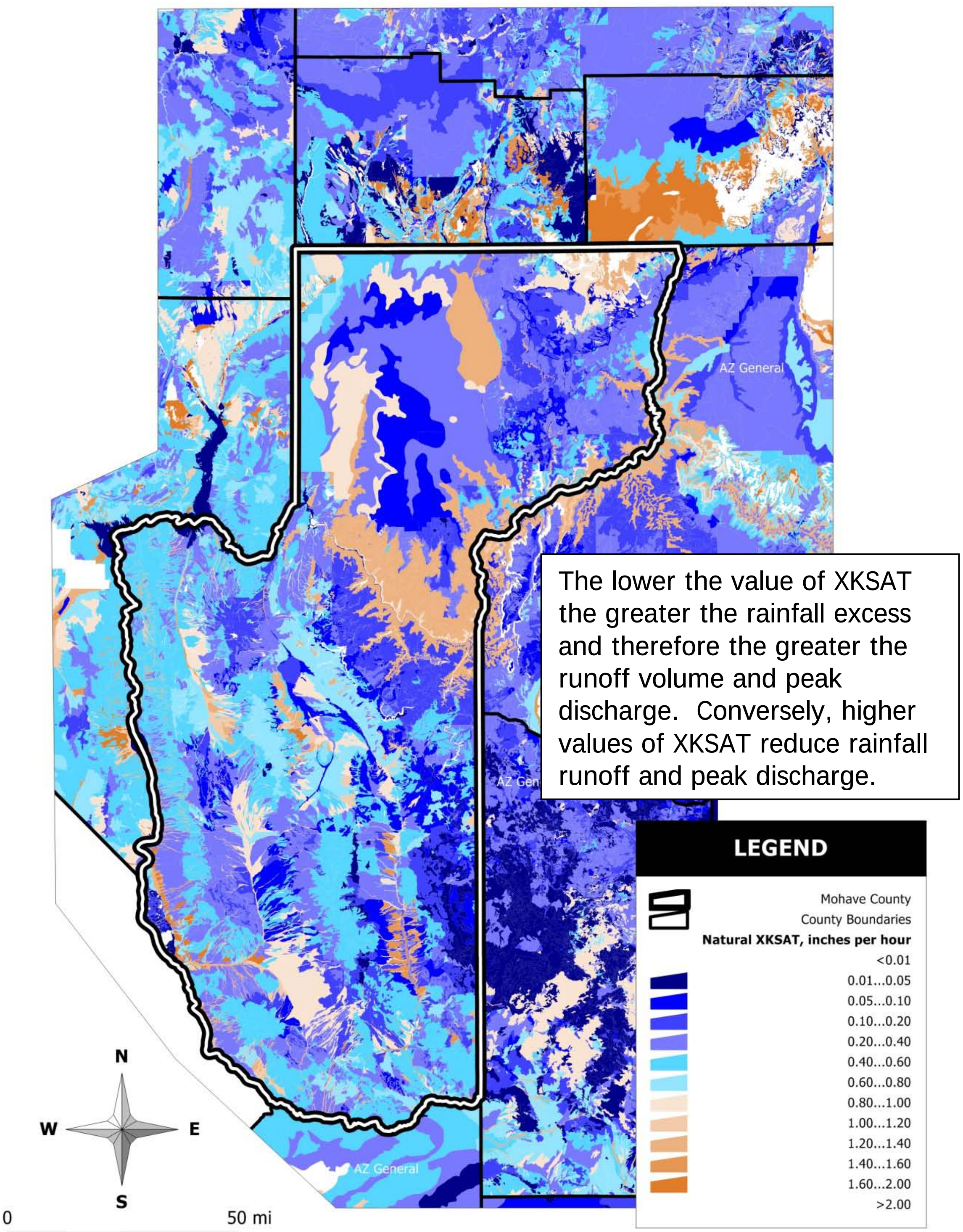


Figure 7.13 Miscellaneous component soils for Mohave County watersheds

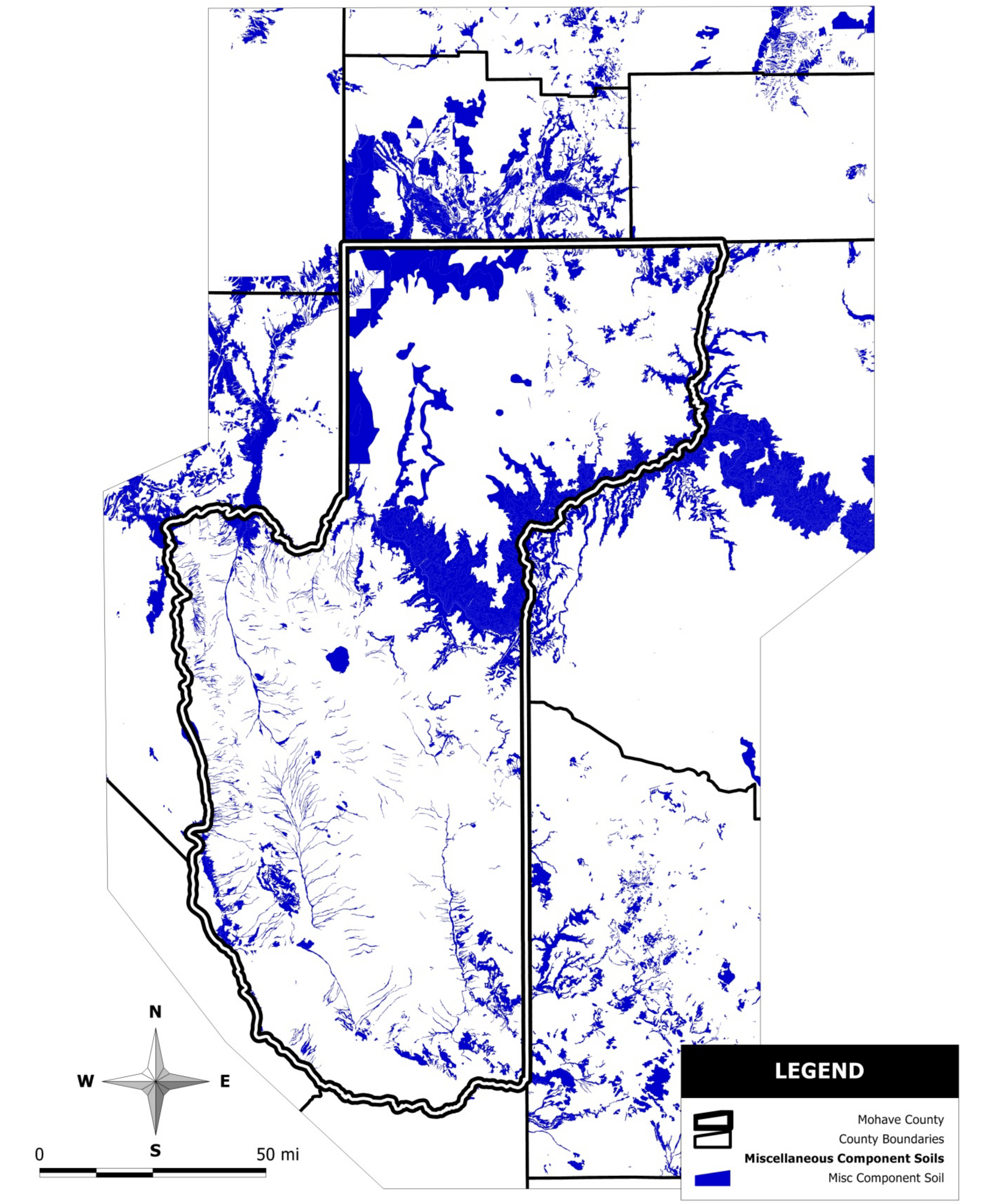


Figure 7.14 *PSIF* as a function of *XKSAT*

(computed from Mohave County watersheds NRCS soils data using information in Saxton and Rawls, 2005)

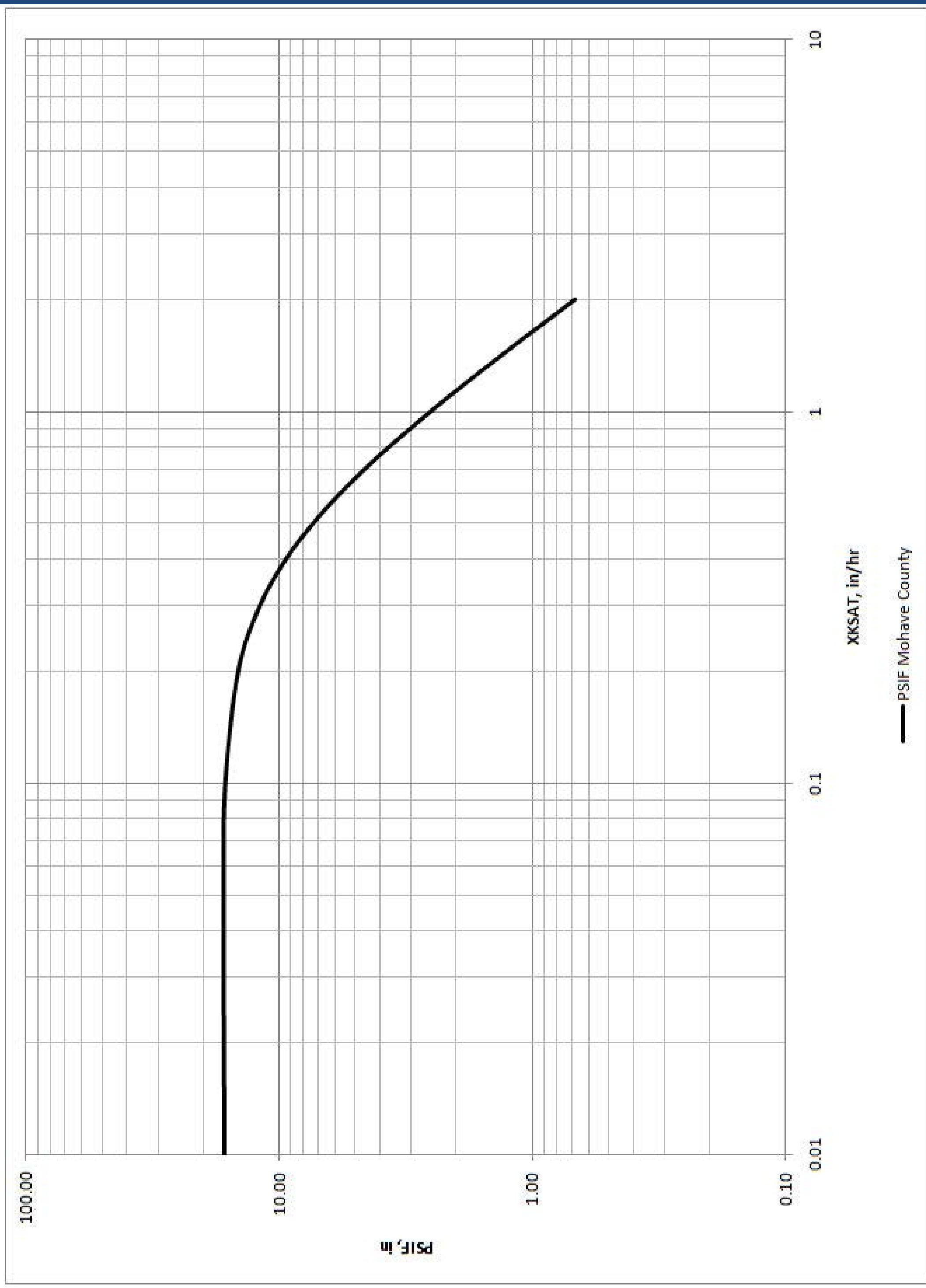


Figure 7.15 *DTHETA* **as a function of** *XKSAT*

(computed from Mohave County watersheds NRCS soils data using information in Saxton and Rawls, 2005)

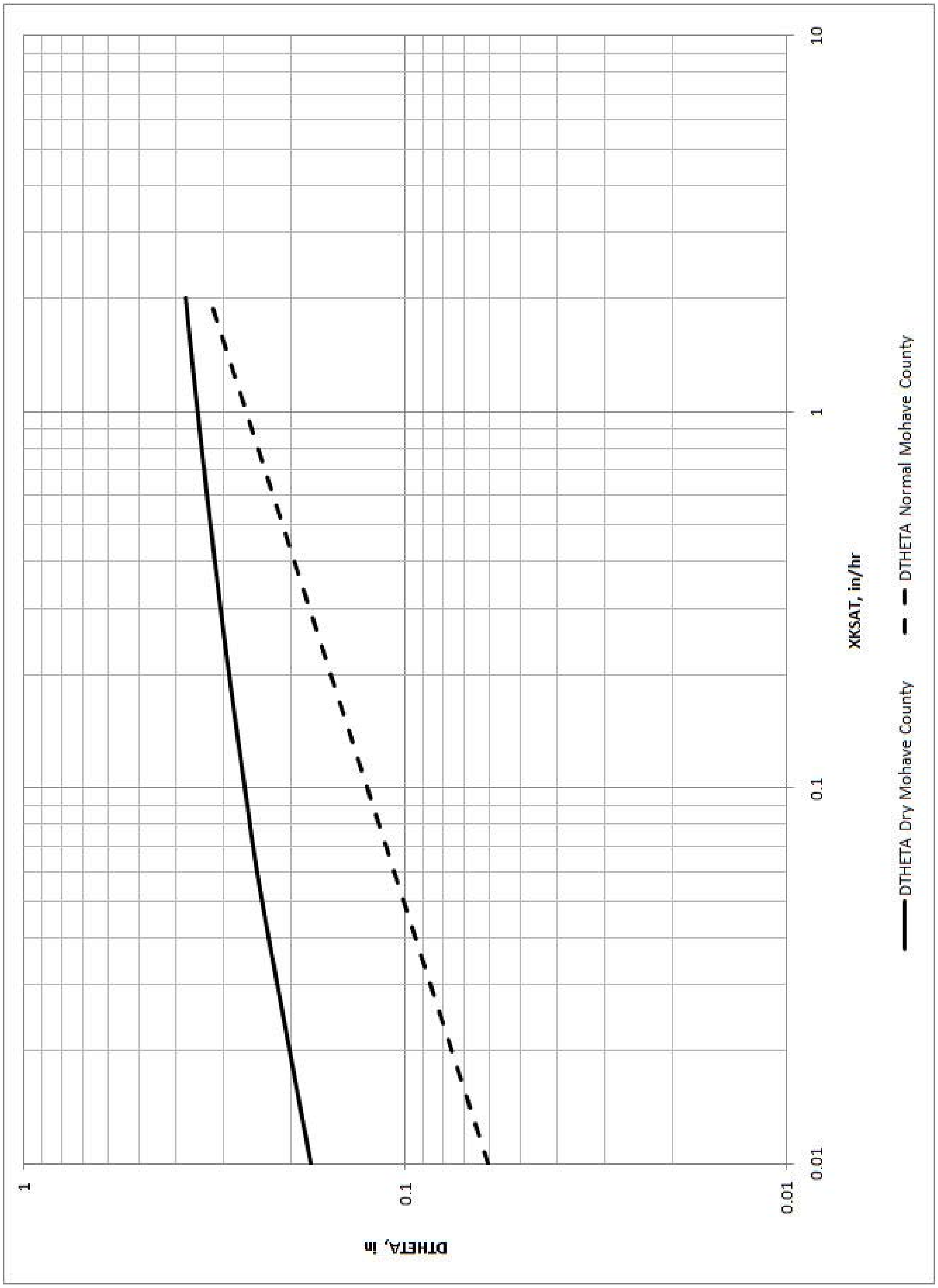


Table 7.8 Green and Ampt equation bare ground loss rate parameter values

(Source: derived from Saxton and Rawls, 2005)

Soil Texture Classification	K_s in/hr	$XKSAT_N$ in/hr	*PSIF* inches	*DTHETA*[a] Dry	Normal	Saturated
(1)	(2)	(3)	(4)	(5)	(6)	(7)
sand[b]	4.03 (4.64)[c]	2.82	2.1 (0.38-9.98)	0.37 (0.35-0.48)	0.29	0.00
loamy sand	3.60 (1.18)	2.52	2.2 (0.53-11.00)	0.37 (0.33-0.48)	0.29	0.00
sandy loam	1.92 (0.43)	1.34	2.7 (1.05-17.90)	0.35 (0.28-0.54)	0.28	0.00
loam	0.61 (0.13)	0.43	8.4 (0.52-23.38)	0.32 (0.33-0.53)	0.21	0.00
silt loam	0.63 (0.26)	0.44	8.2 (1.15-37.56)	0.32 (0.39-0.58)	0.21	0.00
silt	0.80	0.56	6.5	0.33	0.22	0.00
sandy clay loam	0.44 (0.06)	0.31	10.9 (1.74-42.52)	0.31 (0.24-0.43)	0.19	0.00
clay loam	0.17 (0.04)	0.12	17.1 (1.89-35.87)	0.29 (0.28-0.50)	0.15	0.00
silty clay loam	0.23 (0.04)	0.16	15.4 (2.23-51.77)	0.30 (0.35-0.52)	0.16	0.00
silty clay	0.15 (0.02)	0.11	17.7 (2.41-54.88)	0.29 (0.33-0.51)	0.15	0.00
sandy clay	0.06 (0.02)	0.04	20.9 (1.61-55.20)	0.26 (0.21-0.44)	0.13	0.00
clay	0.05 (0.01)	0.04	21.3 (2.52-61.61)	0.26 (0.27-0.50)	0.13	0.00

[a] Selection of *DTHETA*:

Dry:	for non-irrigated lands such as desert and rangeland
Normal:	for irrigated lawn, turf, and permanent pasture
Saturated:	for irrigated agricultural lands

[b] The use of the Green and Ampt Infiltration Equation for drainage areas or sub-basins that are predominately sand or loamy sand should be avoided and the IL+ULR method should be used.

[c] Values in parentheses are from Rawls, Brakensiek and Miller (1983)

For sand and Loamy sand texture classes, use the following:

$XKSAT_N$ = 2.00 in/hr

PSIF = 0.68 in

$DTHETA_{dry}$ = 0.38 in

$DTHETA_{normal}$ = 0.33 in

Values of K_s can vary significantly within a given texture class as shown in the following table. These values were generated using Saxton and Rawls (2005) *Soil Water Characteristics* computer program.

Soil Texture Classification	XKSAT (in/hr)
sand[b]	3.36-9.61
loamy sand	1.54-6.71
sandy loam	0.43-4.56
loam	0.12-1.35
silt loam	0.07-1.81
silt	0.09-0.55
sandy clay loam	0.09-0.76
clay loam	0.05-0.17
silty clay loam	0.06-0.10
silty clay	0.05-0.07
sandy clay	0.00-0.09
clay	0.00-0.05

Adjusting Bare Ground *XKSAT* for Vegetation Cover

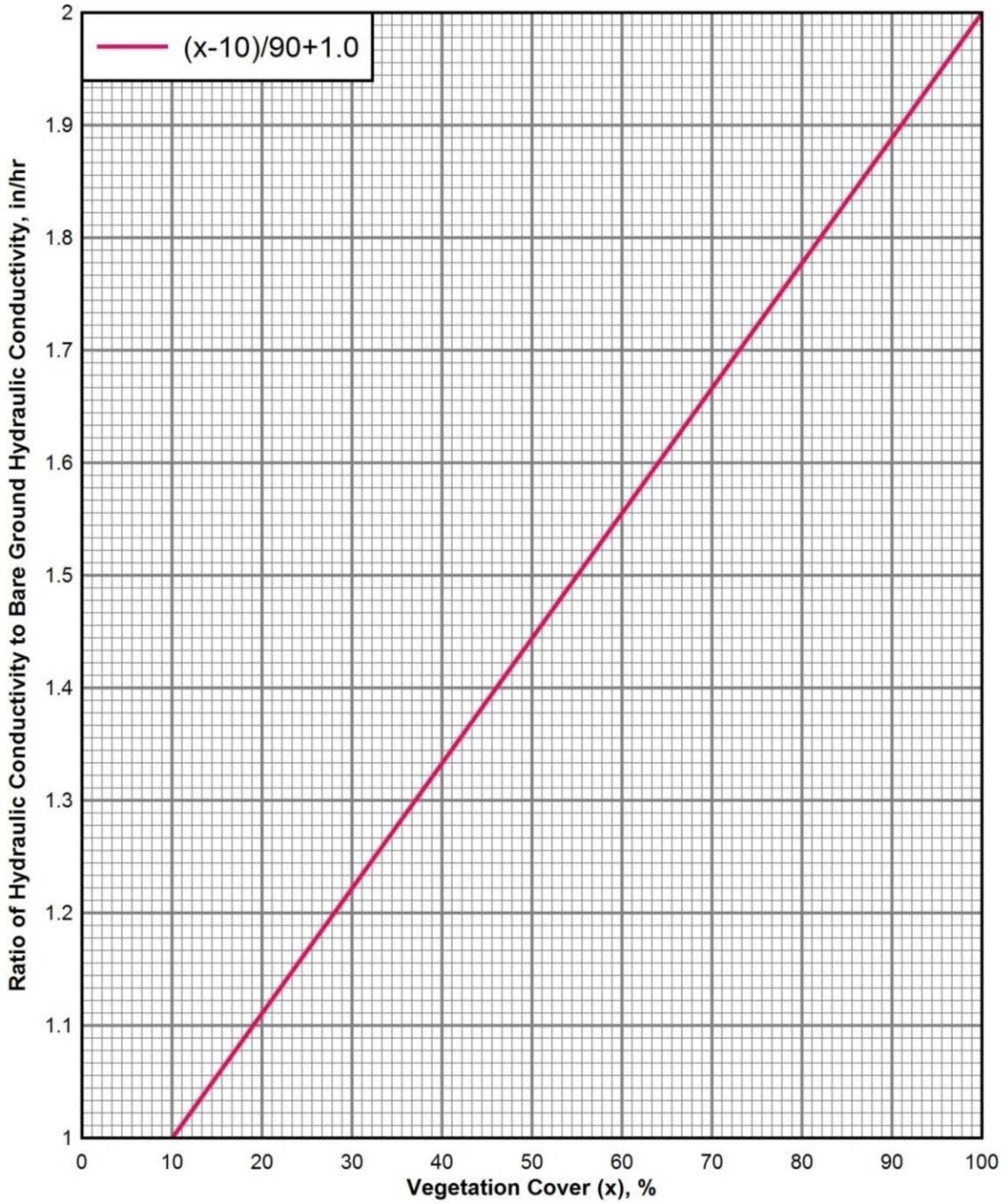
The bare ground value of *XKSAT* can be affected by several factors besides soil texture. For example, hydraulic conductivity is reduced by soil crusting, increased by tillage, and increased by the influence of ground cover and canopy cover. The values of *XKSAT* that are presented for bare ground as a function of soil texture alone should be adjusted under certain soil cover conditions. Ground cover, such as grass, litter, and gravel, will generally increase the infiltration rate over that of bare ground conditions. Similarly, canopy cover such as from trees, brush, and tall grasses can also increase the bare ground infiltration rate. The procedures and data that are presented above for estimating the Green and Ampt parameters are applicable for bare ground conditions. Past research has shown that *PSIF* is relatively insensitive in comparison with *XKSAT*; therefore only the hydraulic conductivity parameter is adjusted for the influences of cover over bare ground.

Procedures have been developed (Rawls and others, 1989) for incorporating the effects of soil crusting, ground cover, and canopy cover into the estimation of hydraulic conductivity for the Green and Ampt equation; however, those procedures are not recommended for use in Mohave County at this time. A simplified procedure to adjust the bare ground hydraulic conductivity for vegetation cover is shown in Figure 7.16. This figure is based on the documented increase in hydraulic conductivity due to various soil covers as reported by investigators using rainfall simulators on native western rangelands (Kincaid and others, 1964; Sabol and others, 1982a; Sabol and others, 1982b; Bach, 1984; Ward, 1986; Lane and others, 1987; Ward and Bolin, 1989). This correction factor can be used based on an estimate of vegetation cover as provided by the NRCS in soil surveys; that is, vegetation cover is evaluated on basal area for grass and forbs, and is evaluated on canopy cover for trees and shrubs. Note that this correction can be applied only to soils other than sand and loamy sand.

The influence of tillage results in a change in total porosity and therefore a need to modify the three Green and Ampt equation infiltration parameters. The effect of tillage systems on soil porosity and the corresponding changes to hydraulic conductivity, wetting front capillary suction, and water retention is available (Rawls and Brakensiek, 1983). Although this information is available, it is not presented in these guidelines, nor is it recommended that these adjustments be made to the infiltration parameters for design purposes in Mohave County. For most flood estimation purposes it cannot be assumed that the soil will be in any particular state of tillage at the time of storm occurrence and therefore the base condition infiltration parameters, as presented, should be used for flood estimation purposes. However, appropriate adjustment to the infiltration parameters can be made, as necessary, for special flood studies such as reconstitution of storm events.

Figure 7.16 Effect of vegetation cover on hydraulic conductivity

Source: FCDMC (2009a)



Note: For all soil textures other than Sand and Loamy Sand. Do not use for *XKSAT*>2.00 in/hr.

Correction of *XKSAT* for vegetation cover using Figure 7.16 is made after the composite value of bare ground *XKSAT* is estimated for a basin or sub-basin. A procedure for estimating vegetation canopy cover density for natural watersheds is presented in Appendix C.

For unincorporated Mohave County, standard values for the natural and developed condition vegetation cover density for each terrain class and zoning category are provided in Table 7.20. The engineer or hydrologist may use the standard default values from Table 7.20, which are also built into the DDMSW computer program. If values other than the standard defaults are more appropriate, the engineer/hydrologist should estimate new values using the procedure in Appendix C or other method approved by Mohave County.

The rainfall loss estimation process also includes the parameter "effective impervious area." In HEC-1, this parameter is coded as the variable *RTIMP*. An estimate of *RTIMP* is needed for each sub-basin. HEC-1 computes no rainfall losses for the percentage of sub-basin area input for *RTIMP*.

Effective Impervious Area, *RTIMP*

Impervious area (or nearly impervious area) is composed of rock outcrop, paved roads, parking lots, roof tops, and so forth. When performing watershed modeling with the HEC-1 program, the impervious area is to be the effective (directly connected) impervious area. For urbanized areas, the effective impervious area should be estimated from aerial photographs with guidance as provided in Table 7.7. For unincorporated Mohave County, standard values for the natural and developed condition *RTIMP* for each zoning category are provided in Table 7.20. The engineer or hydrologist may use the standard default values from Table 7.20, which are also built into the DDMSW computer program. If values other than the standard defaults are more appropriate, the engineer/hydrologist should estimate new values based on estimates of actual *RTIMP* measured by field survey or from aerial photographs. For areas that are presently undeveloped but for which flood estimates are desired for future urbanized conditions, estimates of effective impervious area should be obtained based on regional planning and land-use zoning as determined by the local jurisdiction. Estimates of the effective impervious area for urbanizing areas should be selected from local guidance, if available, along with the general guidance that is provided in Table 7.7, and the more detailed guidance in Table 7.20.

For undeveloped areas, the effective impervious area is often 0 percent. However, in some watersheds there could be extensive rock outcrop or areas of water such as reservoirs that would greatly increase the imperviousness of the watershed. Refer to Figure 7.17 for an overview of the percentage of rock outcrop present in each SMU. Care must be exercised when estimating effective impervious area for rock outcrop. Often the rock outcrop is relatively small (in terms of the total drainage area) and is of isolated units surrounded by soils of relatively high infiltration capacities. Relatively small, isolated rock outcrop may not be effective impervious area because runoff must pass over pervious surfaces before reaching the point of discharge concentration. However, impervious areas that are not hydraulically directly connected may still be included in the estimate of sub-basin *RTIMP* if the intended result includes a conservative estimate of rainfall runoff volume. Often, the *RTIMP* value for such areas is reduced by a factor determined using engineering judgment.

For watersheds that have significant, contiguous rock outcrop, it may be necessary to establish those areas as separate sub-basins so that the direct runoff can be estimated and then routed (with channel transmission losses, if appropriate) to the point of interest. Paved roads through undeveloped watersheds will not normally contribute to effective impervious area unless the road serves as a conveyance to the watershed outlet.

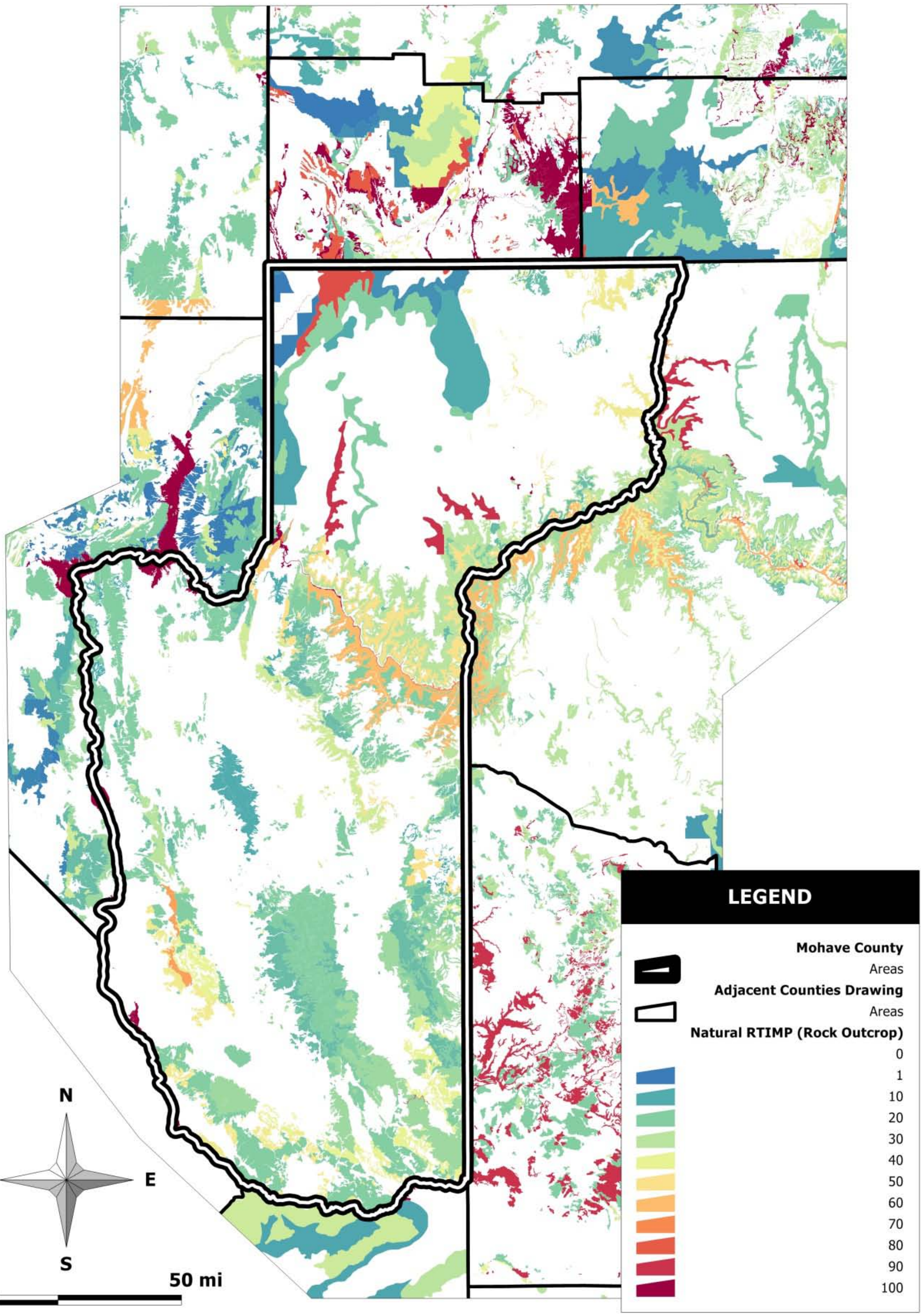
Sensitivity of Green and Ampt Equation Parameters

It is important for the modeler to be aware of the sensitivity of the Unit Hydrograph Method to the various input parameters. More time and effort is warranted for the sensitive parameters than for the less sensitive parameters. The possible effects of each of the parameters discussed above on computation of rainfall excess and peak discharge is shown in Table 7.9 relative to the one percent chance storm.

Table 7.9 Sensitivity of rainfall runoff computations to G&A parameters

Parameter	Storm Frequency		
	More Frequent	One Percent Storm	Less Frequent
IA	Moderate	Moderate-Low	Low
Bare Ground *XKSAT*	High-Very High	Moderate-High	Low-Moderate
XKSAT Adjustment for Vegetation	High-Very High	Moderate-High	Low-Moderate
PSIF	Directly related to *XKSAT*		
DTHETA	Low-Moderate	Low	Very Low
RTIMP	High-Very High	High	Moderate-High

Figure 7.17 Rock outcrop (natural RTIMP) for Mohave County



7.4.3 INITIAL LOSS PLUS UNIFORM LOSS RATE METHOD (IL+ULR)

7.4.3.1 General

This is a simplified rainfall loss estimation method that is often used, and generally accepted, for flood hydrology. It is assumed that the rainfall loss process can be simulated as a two-step procedure, as illustrated in Figure 7.18. The two steps are:

Step 1: All rainfall is lost to runoff until the accumulated rainfall is equal to the initial loss (STRTL).

Step 2: After the initial loss is satisfied, a portion of all future rainfall is lost at a uniform rate (CNSTL). All of the rainfall is lost (runoff does not occur) if the rainfall intensity is less than the uniform loss rate.

The HEC-1 implementation of this method requires input of the three parameters: STRTL, CNSTL, and *RTIMP*.

7.4.3.2 Applicability

This method is acceptable for use when modeling very infrequent storms with high amounts of precipitation. It is also an acceptable method for more frequent storms when the dominate soils in the watershed are sand and/or loamy sand, or drainage areas where soil texture does not control the infiltration rate, such as areas of volcanic cinders. This method should not generally be used for the one percent and more frequent storms.

7.4.3.3 Method Description

Initial Loss

The initial loss, STRTL, can be assumed to consist of two components, the surface retention loss, IA, from the Green and Ampt method, and the initial infiltration, II. After the IA is satisfied, II includes all other losses that occur until the soil profile is saturated and a stabilized, uniform infiltration condition occurs. Therefore, STRTL is the sum of IA and II. IA can be estimated using Table 7.7. II can be estimated using Table 7.10.

Uniform Loss

The uniform loss parameter, CNSTL, is equivalent to the Green and Ampt method bare ground *XKSAT* parameter adjusted for vegetation cover and can be estimated using the procedures for adjusted *XKSAT*.

Table 7.10 IL+ULR Parameter values for bare ground

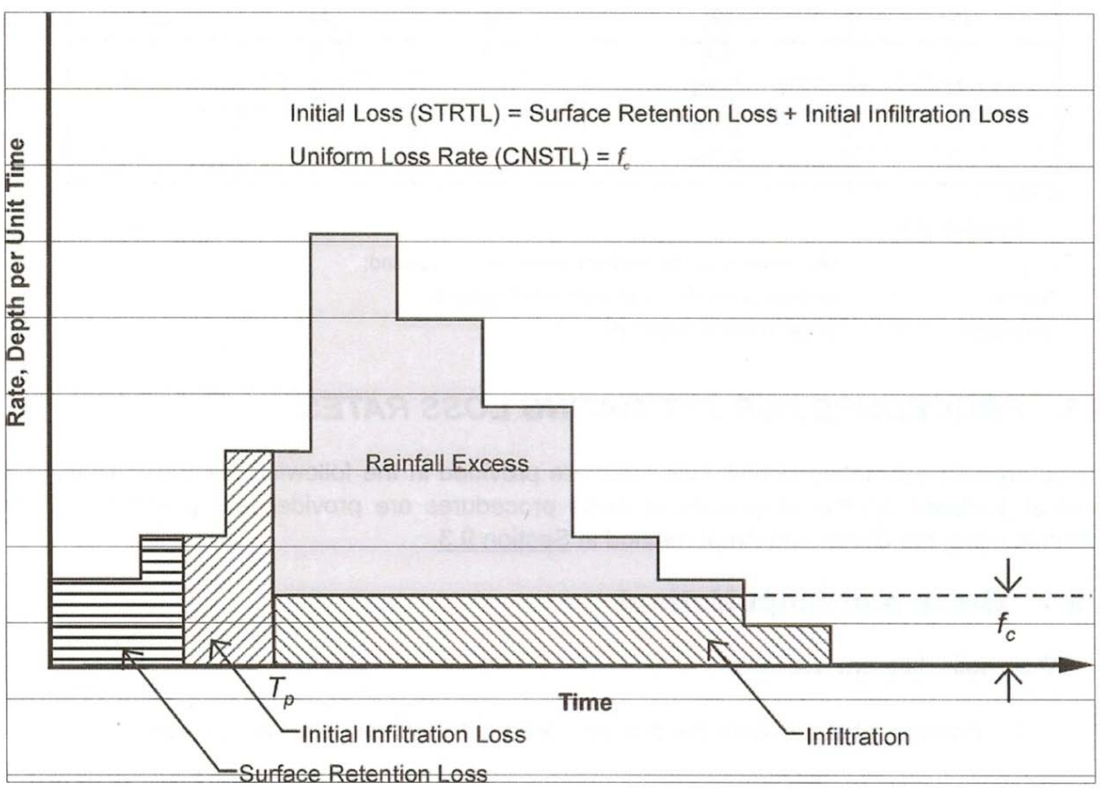
Source: FCDMC (2009a)

Uniform Loss Rate (CNSTL), in/hr	Initial Infiltration (II), inches[1]		
	Dry	Normal	Saturated
(1)	(2)	(3)	(4)
0.30 – 1.20	0.6	0.5	0
0.15 – 0.30	0.5	0.3	0
0.05 – 0.15	0.5	0.3	0
0.00 – 0.05	0.4	0.2	0

Notes:
[1] Selection of II:
Dry = Non-irrigated lands, such as mountain, hillslope and rangeland.
Normal = Irrigated lawn, turf, and permanent pasture.
Saturated = Irrigated agricultural land.

Figure 7.18 Simplified representation of rainfall losses for the IL+ULR method

Source: FCDMC (2009a)



Effective Impervious Area, *RTIMP*

RTIMP for the Initial and Uniform Loss method is identical to the parameter used with the Green and Ampt method. The procedures defined for estimating *RTIMP* for the Green and Ampt method should be used for the Initial and Uniform Loss Rate method.

7.4.4 PROCEDURE FOR GREEN AND AMPT METHOD

7.4.4.1 Overview

Applicability

- For use with the Unit Hydrograph Method.
- All areas where soil texture controls the infiltration rate (*XKSAT* <= 2 inches/hour).

Key Analysis Parameters

$DTHETA_{dry}$ Volumetric soil moisture deficit (soil moisture at wilting point) at start of rainfall, in inches.

$DTHETA_{normal}$ Volumetric soil moisture deficit (soil moisture normal) at start of rainfall, in inches.

$DTHETA_{sat}$ Volumetric soil moisture deficit (soil moisture saturated) at start of rainfall, in inches.

IA Initial abstraction, in inches.

$XKSAT_{BG}$ Bare ground hydraulic conductivity at natural saturation, in inches/hour.

$XKSAT_{adj}$ Hydraulic conductivity at natural saturation, adjusted for the effects of vegetation canopy cover, in inches/hour.

PSIF Wetting front capillary suction, in inches.

RTIMP Effective impervious area, in percent.

VCD Estimated vegetation canopy cover, in percent.

7.4.4.2 General

In general the following steps are used to compute rainfall loss parameters for the Green and Ampt method. The sets of instructions following these general steps are specific to computing parameter values for each sub-basin. The descriptions below use GIS procedures to describe the process. Whether or not GIS is used to perform the data sorting and computations, the basic processes are the same for hand computations and use of CADD or other software applications. The DDMSW computer program automates most of the computations. The GIS process was selected for these descriptions because the NRCS detailed soil data are mostly available in only a GIS or PDF format and the NRCS is only publishing new studies in these formats. To perform the computations by hand or using other software, the GIS data must first be converted to a scaled paper map or converted to another digital format more convenient to the user. Additional descriptions are provided where the hand computation process differs from the GIS procedure.

1. <u>Sub-basin Delineation.</u> Prepare a base map of the drainage area and delineate modeling basins for the concentration points of interest. Delineate sub-basins for each basin so that the sub-basins are reasonably homogeneous in terms of area and/or time of concentration characteristics, and surface characteristics and/or soil type. Delineate large impervious areas as separate sub-basins. Create GIS polygon coverages for each basin and sub-basin and calculate the area of each basin and sub-basin.
2. <u>Subarea Delineation.</u> Delineate subareas for each sub-basin for the purpose of assigning IA and *RTIMP* estimates. The polygons from NRCS soil surveys delineating soil map units also are subareas, and often are used as subareas for estimation of IA and *RTIMP*, based on soil characteristics. Create GIS polygon coverages for each subarea and calculate the area of each subarea within each sub-basin.
3. <u>Subarea Parameters.</u> Assign estimates of IA and *RTIMP* for each subarea.
4. <u>Estimate Composite IA for each Sub-basin.</u>
5. <u>Estimate Composite *RTIMP* for each Sub-basin.</u>
6. <u>Estimate Bare Ground *XKSAT* for each Soil Map Unit (subarea).</u>
7. <u>Estimate Composite Bare Ground *XKSAT* for each Sub-basin.</u>
8. <u>Estimate *PSIF* and *DTHETA* for each Sub-basin based on Composite Bare Ground *XKSAT*</u>
9. <u>Estimate Adjusted Composite *XKSAT* for each Sub-basin.</u>
10. <u>HEC-1 Loss Rate Record.</u> Enter the composite values of IA, *DTHETA*, *PSIF*, adjusted *XKSAT*, and *RTIMP* for the drainage area or each sub-basin on the LG record of the HEC-1 input file.

7.4.4.3 Instructions for Sub-basin Composite IA

1. Assign an IA Estimate to Sub-basin Subareas: Sub-basins may have to be divided into subareas based on land use and/or surface characteristics. The NRCS soil map units may also be used. An estimate of IA can be made for each soil map unit, entered into the GIS table for each soil map unit, and then area averaged as described in step 2. NRCS soil

map units are further described under the Saturated Hydraulic Conductivity, *XKSAT* section.

2. Compute a Composite Value of IA: If there are multiple subareas within a sub-basin, calculate an area-weighted value of IA using Equation 7.12.

$$\overline{IA} = \left(\frac{\sum A_i IA_i}{A_T} \right) \qquad 7.12$$

where:

$\overline{IA}$	=	composite value of IA, inches
IA_i	=	IA of each subarea, inches
A_i	=	size of IA subarea
A_T	=	size of the watershed or sub-basin

7.4.4.4 Instructions for Sub-basin Composite *XKSAT*

1. Intersect the GIS Soils Coverage: Use the ArcMap intersect tool to divide the soil map unit polygon coverage so that the soil map unit polygon boundaries are divided by the watershed sub-basin boundaries. This is done by using the soil map unit polygon coverage as the input feature and the watershed sub-basin GIS coverage as the clip feature. The results are soil map unit polygons completely contained within each sub-basin polygon.

2. Simplify the GIS Soils Coverage: Use the ArcMap dissolve tool to simplify the soil map unit polygons within each watershed sub-basin, based on the soil map unit identifier field. When completed, there will only be one polygon for each soil map unit within each sub-basin polygon. When performing this step by hand, identify all polygons that have the same *XKSAT* value and then color code the *XKSAT* polygons.

3. Compute a Composite Bare Ground *XKSAT* Value for Each Sub-basin: Standard values of *XKSAT* for each soil map unit are listed in Appendix D. Use ArcMap to compute the area of each soil map unit within each sub-basin. Then either use ArcMap to apply Equation 7.13, or export the soil map unit number, *XKSAT* value and area information for each sub-basin into a Microsoft Excel spreadsheet and apply Equation 7.13 within the spreadsheet. When performing these computations by hand, planimeter each color-shaded polygon to obtain the total area of each *XKSAT* value within the sub-basin. Then apply Equation 7.13 by hand or within a spreadsheet.

$$\overline{XKSAT} = 10^{\left(\frac{\sum A_i \log_{10} XKSAT_i}{A_T} \right)} \qquad 7.13$$

where:

$\overline{XKSAT}$	=	composite bare ground hydraulic conductivity for the watershed sub-basin, inches/hour

$XKSAT_i$ = bare ground hydraulic conductivity of the soil map unit within a sub-basin, inches/hour

A_i = area of soil map unit subarea within a sub-basin

A_T = total area of the watershed or sub-basin

7.4.4.5 Instructions for *PSIF* and *DTHETA*

1. Read Values of *PSIF* and *DTHETA* from Figure 7.14 and Figure 7.15: Enter the x-axis of Figure 7.14 with the composite bare ground value of *XKSAT* for each sub-basin. Read the corresponding value of *PSIF* on the y-axis. Enter the x-axis of Figure 7.15 with the composite bare ground value of *XKSAT* for each sub-basin. Read the corresponding value of *DTHETA* dry or normal, as appropriate, on the y-axis. As an alternative, Equations 7.9, 7.10, and 7.11 may be used to compute values of *PSIF* and *DTHETA*. Where subareas of differing *DTHETA* are present in a sub-basin, when natural and developed land uses are both present for example, an area-weighted value of *DTHETA* should be computed using Equation 7.14.

$$\overline{DTHETA} = \left(\frac{\sum A_i DTHETA_i}{A_T} \right) \tag{7.14}$$

where:

$\overline{DTHETA}$ = composite value of *DTHETA*, inches

$DTHETA_i$ = *DTHETA* of each subarea, inches

A_i = size of *DTHETA* subarea

A_T = size of the watershed or sub-basin

7.4.4.6 Instructions for Adjusting *XKSAT* for Vegetation Canopy Cover

1. Estimate Vegetation Cover Density (VCD) for Each Sub-basin: Determine an estimate of average vegetation cover density (canopy cover) for each land use classification using aerial photographs supplemented by field verifications as described in Appendix C. Compute an area-averaged value of VCD for each sub-basin using Equation 7.15.

$$\overline{VCD} = \left(\frac{\sum A_i VCD_i}{A_T} \right) \tag{7.15}$$

where:

$\overline{VCD}$ = composite value of VCD, inches

VCD_i = VCD of each subarea, inches

A_i = size of VCD subarea

A_T = size of the watershed or sub-basin

2. Obtain the Bare Ground *XKSAT* Adjustment Factor from Figure 7.16: Enter Figure 7.16 on the x-axis with the estimated vegetation cover density for each sub-basin. Read the ratio of adjusted *XKSAT* to bare ground *XKSAT* on the y-axis.

3. Compute Adjusted *XKSAT*: Multiply the sub-basin bare ground *XKSAT* estimate by the factor from the y-axis of Figure 7.16 to obtain the adjusted *XKSAT* value.

Equation 7.16 may be used in lieu of Figure 7.16.

$$XKSAT_{adj} = \overline{XKSAT}_{BG}\left(\frac{\overline{VCD}-10}{90}+1\right) \qquad 7.16$$

where:

$XKSAT_{adj}$ = $\overline{XKSAT}_{BG}$ adjusted for effects of vegetation canopy cover, inches/hour

$\overline{XKSAT}_{BG}$ = Sub-basin composite bare ground *XKSAT*, inches/hour

$\overline{VCD}$ = Sub-basin composite value of vegetation canopy cover, percent

7.4.4.7 Instructions for Sub-basin Composite *RTIMP*

1. Assign an *RTIMP* Estimate to Sub-basin Subareas: Sub-basins may have to be divided into subareas based on land use and/or surface characteristics. *RTIMP* consists of any impervious surface that is hydraulically connected to the watershed outlet, including large areas of natural rock, large bodies of pooled water, asphalt and concrete pavement, rooftops, etc. Normally, naturally occurring RTIMP ($RTIMP_N$) is defined separately from developed condition RTIMP ($RTIMP_D$). A composite RTIMP is computed for each sub-basin for natural and developed conditions. Then the two are added to obtain a total composite RTIMP for the sub-basin. Aerial photographs can be used to aid in the process of defining subareas, particularly for developed watersheds. Planning and zoning maps may also be used for developed or developing areas. The NRCS soil map units may be used to aid in estimating *RTIMP* for natural areas. Standard values of natural *RTIMP* are provided for each soil map unit in Appendix D, but should be field verified if significant.

2. Compute a Composite Value of *RTIMP*: If there are multiple subareas within a sub-basin, calculate an area-weighted value of *RTIMP* using Equation 7.17. Use this equation for computing both $RTIMP_N$ and $RTIMP_D$.

$$\overline{RTIMP} = \left(\frac{\sum A_i RTIMP_i}{A_T}\right) \qquad 7.17$$

where:

$\overline{RTIMP}$ = composite value of *RTIMP*, inches

$RTIMP_i$ = *RTIMP* of each subarea, inches

A_i = area of *RTIMP* subarea

A_T = area of the watershed or sub-basin

7.4.4.8 Green and Ampt Method Example

Refer to Appendix A.3.1 for an example of application of the Green and Ampt Method.

7.4.5 PROCEDURE FOR INITIAL LOSS AND UNIFORM LOSS RATE METHOD

7.4.5.1 Overview

Applicability

- This method is acceptable for use when modeling very infrequent storms with high amounts of precipitation.
- This method is an acceptable method for more frequent storms when the dominate soils in the watershed are sand and/or loamy sand, or drainage areas where soil texture does not control the infiltration rate, such as areas of volcanic cinders.
- This method should not generally be used for the one percent and more frequent storms.

Key Analysis Parameters

IA	Initial abstraction, in inches.
II	Initial infiltration, in inches.
STRTL	The total of IA and II, in inches.
CNSTL	Uniform loss rate, equivalent to bare ground hydraulic conductivity at natural saturation, in inches/hour.
RTIMP	Effective impervious area, in percent.

7.4.5.2 General

In general the following steps are used to compute rainfall loss parameters for the Initial Loss and Uniform Loss Rate Method. The sets of instructions following these general steps are specific to computing parameter values for each sub-basin.

1. Sub-basin Delineation. Prepare a base map of the drainage area and delineate modeling basins for the concentration points of interest. Delineate sub-basins from each basin so that the sub-basins are as homogeneous as possible in terms of area and/or time of concentration characteristics, and surface characteristics and/or soil type. Delineate large areas of impervious area as separate sub-basins. Create GIS polygon coverages for each basin and sub-basin and calculate the area of each basin and sub-basin.

2. Subarea Delineation. Delineate subareas for each sub-basin for the purpose of assigning IA and *RTIMP* estimates. The polygons from NRCS soil surveys delineating soil map units also are subareas, and often are used as subareas for estimation of IA and *RTIMP*. Create GIS polygon coverages for each subarea and calculate the area of each subarea.
3. Subarea Parameters. Assign estimates of IA, II and *RTIMP* for each subarea.
4. Estimate Bare Ground *XKSAT* for each soil map unit (subarea). Follow the procedure in Section 7.4.4.4.
5. Estimate CNSTL for Each Sub-basin. Compute composite bare ground *XKSAT* for each sub-basin, adjust for vegetation cover, and assign as CNSTL.
6. Estimate STRTL for Each Sub-basin. Compute STRTL by summing composite IA and an estimate of II.
7. Estimate Composite *RTIMP* for each Sub-basin. Follow the procedures in Section 7.4.4.7.
8. HEC-1 Loss Rate Record. Enter the composite values of STRTL, CNSTL, and *RTIMP* for the drainage area or each sub-basin on the LU record of the HEC-1 input file.

7.4.5.3 Instructions for STRTL

1. Compute a Composite Value of IA: Use the procedures defined for the Green and Ampt method to compute a composite value of IA for each sub-basin.
2. Compute a Composite Value of II: Use the sub-basin composite estimate of CNSTL (see below) to estimate a value of II from Table 4.
3. Compute an Estimate of STRTL for each Sub-basin: Add IA and II to obtain an estimate of STRTL.

7.4.5.4 Instructions for CNSTL

1. Compute a Composite Value of CNSTL: Use the procedures defined for the Green and Ampt method to compute a composite value of *XKSAT* adjusted for vegetation cover for each sub-basin, and use those values for CNSTL.

7.4.5.5 Initial Loss and Uniform Loss Method Example

Refer to Appendix A.3.2 for an example of application of the Initial Loss and Uniform Loss Method.

7.5 UNIT HYDROGRAPHS

7.5.1 INTRODUCTION

7.5.1.1 General Discussion

A unit hydrograph is defined as the hydrograph of one inch of direct runoff from a storm of a specified duration for a particular watershed. Every watershed will have a different unit hydrograph that reflects the physiography, topography, land-use, and other unique characteristics of the individual watershed. Different unit hydrographs will be produced for the same watershed for different durations of rainfall excess. For example, a unit hydrograph for a particular watershed can be developed for a rainfall excess duration of 5-minutes, or 15-minutes, or 1-hour, or 6-hours, etc. Any duration can be selected for unit hydrograph development as long as an upper limit for the unit hydrograph duration is not exceeded. Guidelines for the determination of the upper limit of unit hydrograph duration are provided in Section 7.5.2.5.

Only a few watersheds in Arizona will have an adequate data base (rainfall and runoff records) from which to develop unit hydrographs. Therefore, indirect methods usually will be used to develop unit hydrographs. Such unit hydrographs are called synthetic unit hydrographs. Several procedures are available to develop synthetic unit hydrographs, and virtually all of these procedures are empirical. The selection of a synthetic unit hydrograph procedure should be made such that the data base for the empirical development is representative of the study watershed.

The unit hydrograph itself is a lumped parameter in that it represents the composite effects of all of the watershed and storm characteristics that dictate the rate of rainfall excess runoff from the watershed. Although there are numerous watershed and storm characteristics that determine the shape of a unit hydrograph, only a limited number of those characteristics can be quantified and used to calculate a unit hydrograph. One or more unit hydrograph parameters (depending on the selection of a synthetic unit hydrograph procedure) are needed to calculate a unit hydrograph.

The concept of the unit hydrograph is used to route the time increments of rainfall excess from the watershed (or modeling sub-basin) to the watershed outlet (or modeling concentration point). The synthetic unit hydrograph procedure that is recommended for use in Mohave County is the Clark unit hydrograph. Procedures are provided herein to estimate the three Clark unit hydrograph parameters and these are entered on the UC and UA records of HEC-1. Unit hydrograph procedures other than the Clark procedure can be used for specific applications; however, this will require justification and approval by DISTRICT for such use.

7.5.2 PROCEDURE

The Clark unit hydrograph requires the estimation of three parameters; the time of concentration (T_c), the storage coefficient (R), and a time-area relation. Sections 7.5.2.1 through 7.5.2.6 describe the procedures that are to be used to calculate these parameters, and the guidelines that are to be used to select the unit hydrograph duration and computation interval (NMIN).

7.5.2.1 Overview

Applicability

- The Clark unit hydrograph, as described herein, can be used for virtually any watershed that will be encountered in Mohave County.
- The Unit Hydrograph Method is normally applied for watersheds greater than 160 acres in area and when a runoff hydrograph is needed.

Key Analysis Parameters

A	Watershed area, in square miles.
L	The length of the longest hydraulic flow path, in miles.
L_{ca}	Length measured from the concentration point along L to a point on L that is perpendicular to the watershed centroid, in miles.
Q	The estimated peak discharge, in cubic feet per second.
R	Storage coefficient, in hours.
$RTIMP$	Effective impervious area, in percent.
S	The slope of the longest hydraulic flow path, in ft/mile.
T_c	Time of Concentration, in hours.

7.5.2.2 Time of Concentration

Time of concentration is the travel time, during the corresponding period of most intense rainfall excess, for a flood wave to travel from the hydraulically most distant point in the watershed to the point of interest (concentration point). Three T_c equations are to be used depending on the type of watershed; desert/mountain, agricultural fields, or urban. The recommended T_c equations are:

desert/mountain

$$T_c = 2.4A^{0.1}L^{0.25}{L_{ca}}^{0.25}S^{-0.2} \qquad 7.18$$

agricultural fields

$$T_c = 7.2A^{0.1}L^{0.25}{L_{ca}}^{0.25}S^{-0.2} \qquad 7.19$$

urban

$$T_c = 3.2A^{0.1}L^{0.25}{L_{ca}}^{0.25}S^{-0.14}RTIMP^{-0.36} \qquad 7.20$$

where:

T_c	=	time of concentration, in hours,
A	=	area, in square miles,
S	=	watercourse slope, in feet/mile,
L	=	length of watercourse to the hydraulically most distant point, in miles,
L_{ca}	=	length measured from the concentration point along L to a point on L that is perpendicular to the watershed centroid, in miles, and
$RTIMP$	=	effective impervious area, in percent.

In using Equations 7.18 through 7.20, the following points should be noted and observed:

1. The area (A) will be determined from the best available map. The delineation of the drainage boundary needs to be carefully performed, and special care must be taken where there is little topographic relief. In urban areas, land grading and road construction can produce drainage boundaries that separate runoff from contributing areas during small and lower intensity storms. However, larger and more intense storms, such as the design storm from this manual, can produce runoff depths that can cross these intermediate drainage boundaries resulting in a larger total contributing area. Similarly, floods on alluvial fans (active and inactive) and in distributary flow systems can result in increased contributing areas during larger and more intense storms. For such areas, it is generally prudent to consider the largest reasonable drainage area in these situations.

2. Determination of the hydraulically most distant point will define both *L* and *S*. Often, the hydraulically most distant point is determined as the point along the watershed boundary that has the longest flow path to the watershed outlet (or sub-basin concentration point). This is generally true where the topography is relatively uniform throughout the watershed. However, there are situations where the longest flow path (L) does not define the hydraulically most distant point. Occasionally, especially in mountainous areas, a point with a shorter flow path may have an appreciably flatter slope (*S*) such that the shorter flow path defines the hydraulically most distant point. For watersheds with multiple choices for the hydraulically most distant point, the T_c should be calculated for each point and the largest T_c should be used.

3. Slope (*S*) is the average slope calculated by dividing the difference in elevation between the hydraulically most distant point and the watershed outlet by the watercourse length (*L*). This method will usually be used to calculate *S*. However, there are situations where special consideration should be given to calculating *S* and to dividing the watershed into sub-basins. For example, if there is dramatic change in watercourse slope throughout the watershed, then the use of a multiple sub-basin model should be considered with change in watercourse slope used as criteria in delineating the sub-basins. There will also be situations where the watercourse contains vertical or nearly vertical drops (mountain rims, headcuts, rock outcrop, and so forth). In these situations, plotting of the watercourse profile will usually identify nearly vertical changes in the channel bed. When calculating the average slope, subtract the accumulative elevation differential that occurs in nearly vertical drops from the overall elevation differential prior to calculating S.

4. L_{ca} is measured along L to a point on L that is essentially perpendicular to the watershed centroid. This is a shape factor in the T_c equation. Occasionally, the shape of agricultural fields or urban sub-basins is nearly rectangular and this may result in two different dimensions for L_{ca}. In the case of such nearly rectangular (and therefore, nearly symmetrical) watersheds or sub-basins, L_{ca} can usually be satisfactorily estimated as 0.5*L*.

5. *RTIMP* is the effective impervious area. This is the same value that was determined for the watershed by the procedures in Section 7.4.2.3. *RTIMP* is used to estimate T_c for urban watersheds only (Equation 7.20).

6. Ideally, the selection of the watershed or sub-basin boundaries can be made so that the area represents a hydrologically uniform region that is essentially all desert/mountain, or agricultural fields, or urban, and for those situations, the T_c equations (Equations 7.18 through 7.20) can be applied directly. However, there will be situations where the watershed or modeling sub-basin is a mixture of two or three of those types. In those cases, the T_c equation (Equations 7.18 through 7.20) is selected based on the watershed type that contains the greatest portion of *L*. The effects of a mixture of watershed types are accounted for by the selection of the time-area relation (to be discussed in Section 7.5.2.4).

7.5.2.3 Storage Coefficient

The storage coefficient is a Clark unit hydrograph parameter that relates the effects of direct runoff storage in the watershed to unit hydrograph shape. The equation for estimating the storage coefficient (*R*) is:

$$R = 0.37{T_c}^{1.11} L^{0.80} A^{-0.57} \quad (7.21)$$

where: R is in hours and the variables are as defined for the T_c equations.

7.5.2.4 Time-Area Relation

The time-area relation is a graphical parameter that specifies the accumulated area of the watershed that is contributing runoff to the outlet of the watershed at any time. Two methods can be used to develop a time-area relation:

1. by analysis of the watershed to define incremental runoff producing areas that have equal incremental travel times to the outflow location, or
2. by use of synthetic time-area relations.

The development of a time-area relation by analysis of the watershed is a difficult task and well-defined and reliable procedures for this task are not available. Unless the watershed has an extremely unusual shape, or has several distinct areas of dramatically different land-use, this analysis should not be undertaken. In general, synthetic time-area relations can be used in Mohave County. If it is necessary to develop a site-specific time-area relationship for a watershed, refer to FCDMC (2009a), Section 5.5.3, for a description of the procedure

The dimensionless, synthetic time-area relations that can be used in Mohave County are listed in Table 7.11 and shown graphically in Figure 7.19. Curve A should be used if the land-use in the watershed or sub-basin is urban or predominantly urban. Curve C should be used if the land-use in the watershed or sub-basin is desert/rangeland or is mostly desert/rangeland with some mountains in the watershed and/or some irrigated agricultural fields interspersed in the lowlands. Curve B should be used for all other situations.

Curve B is the default time-area relation in HEC-1 and will be used with the Clark unit hydrograph if a time-area relation (UA record) is not supplied. Curves A and C are dimensionless and these curves are input to HEC-1 by inserting the percent of total area values from Table 7.11 in the UA record.

Table 7.11 Values of the dimensionless synthetic time-area relations

Source: ADOT (1993)

Travel Time as a percent of T_c	Contributing Area, as a Percent of Total Area[a]		
	A	B[b]	C
(1)	(2)	(3)	(4)
0	0	0.0	0
10	5	4.5	3
20	16	12.6	5
30	30	23.2	8
40	65	35.8	12
50	77	50.0	20
60	84	64.2	43
70	90	76.8	75
80	94	87.4	90
90	97	95.5	96
100	100	100.0	100

[a] The dimensionless Synthetic Time-Area relations should be selected as follows:

A - The land-use in the watershed or sub-basin is urban or predominantly urban.

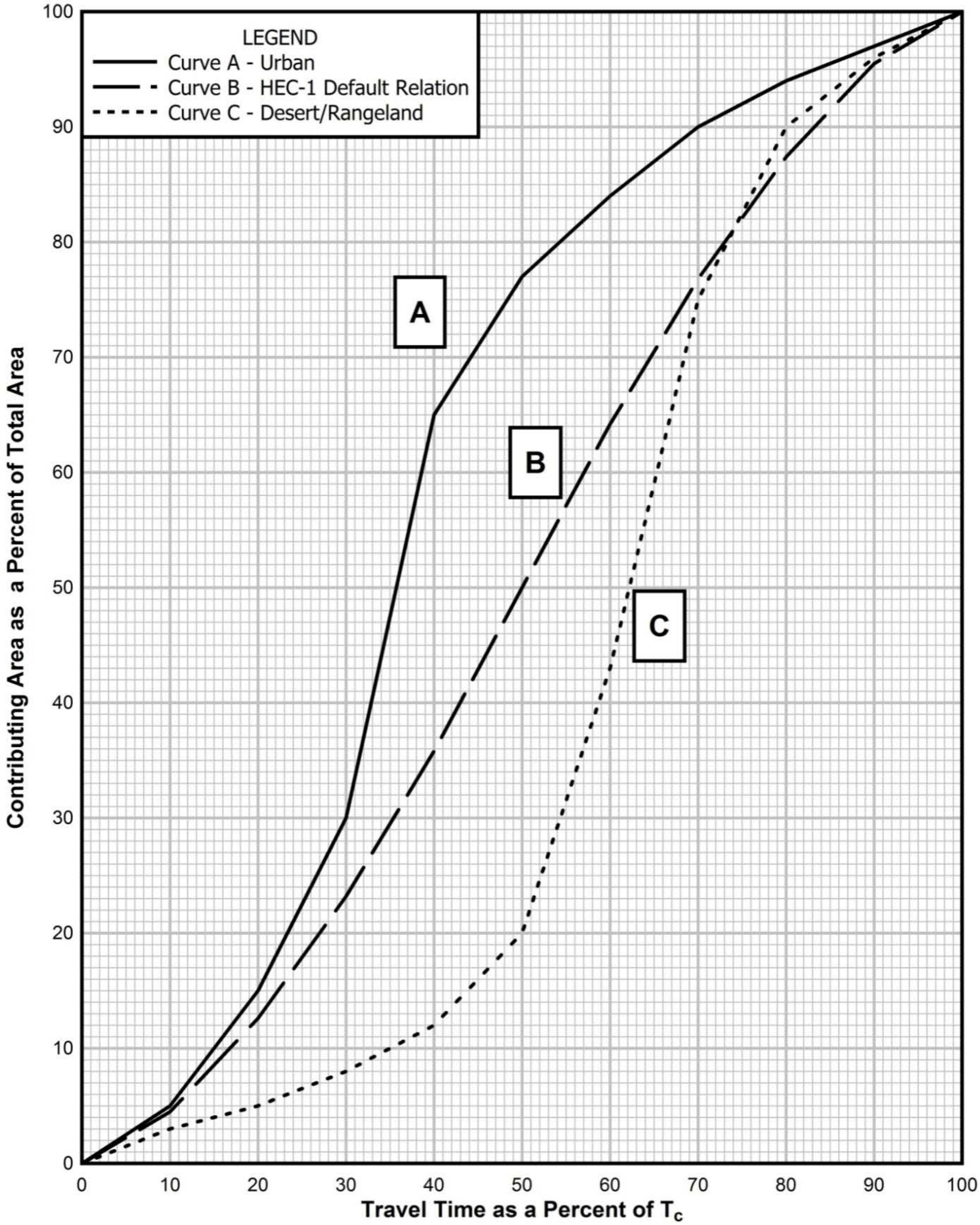
B - All watersheds or sub-basins other than those defined for use of curves A or C.

C - The land-use in the watershed or sub-basin is desert/rangeland or is mostly desert/rangeland with some mountains in the watershed and/or some irrigated agricultural fields interspersed in the lowlands.

[b] Curve B is the HEC-1 default Time-Area relation and the UA record is not needed as input to the HEC-1 model.

Figure 7.19 Synthetic Time-Area Relation

Source: ADOT (1993)



7.5.2.5 Duration

The duration of the unit hydrograph (or all unit hydrographs in a multiple sub-basin model) is specified in HEC-1 on the IT record as NMIN. In general, NMIN will be selected according to the following criteria:

NMIN = 2 minutes for a 6-hour storm duration (drainage area less than or equal to 1.0 square mile), and

NMIN = 5 minutes for a 24-hour storm duration (drainage area greater than 1.0 square mile).

Note: *NMIN* should not exceed 0.25 T_c for the sub-basin with the shortest T_c.

However, there may be special situations (see Section 7.5.2.2, Time of Concentration, for using HEC-1) where a NMIN, other than as defined above, is to be used. In those situations, the following rules should be considered:

1. *NMIN* = 0.15 T_c provides adequate definition of the hydrograph peak with an optimum number of hydrograph coordinate calculations.
2. *NMIN* = 0.25 T_c is the maximum value for NMIN.
3. *NMIN* for a multiple sub-basin model should be selected based on the smallest T_c value for any of the sub-basins in the model.

7.5.2.6 Applications and Limitations

The Clark unit hydrograph, as described herein, can be used for virtually any watershed that will be encountered in Mohave County. However, there may be situations where use of another unit hydrograph will be warranted. For example, rainfall and runoff data may be available for the watershed or a nearby hydrologically similar watershed to develop a unit hydrograph, and in those cases, the developed unit hydrograph would be input to HEC-1 by use of UI records. In other situations, a unit hydrograph at or near the desired location may have been developed for another project. That unit hydrograph or unit hydrograph procedure may be preferable to the recommended Clark unit hydrograph procedure for that application. If other unit hydrographs or unit hydrograph procedures are determined to be more applicable for a certain situation, they should be used. However, deviations from the procedures in this Manual should be discussed with DISTRICT and approval received for deviations from the recommended procedures before incorporating such deviations into the project hydrology analysis.

Equations 7.18 through 7.20 were derived for use in estimating the time of concentration for floods with design return periods that are typical for roadway drainage structures (25-year to 100-year). Use of these equations may result in time of concentration estimates that are too short for floods of return period less than 25-year and too long for floods of return periods appreciably greater than 100-year. This is because of the effect that runoff magnitude has on the hydraulic efficiency (runoff velocity) of watersheds. Therefore, if Equations 7.18 through 7.20 are used to estimate the time of concentration for floods of return period appreciably greater than the 100-year, then the time of concentration should be reduced (by as much as 25 percent for very large. rare floods); similarly, for estimating the time of concentration for floods of return period less than the 25-year, the time of concentration should be increased (by as much as 100 percent for very frequent flooding, such as the 2-year). Since R (Equation 7.21) is a function of T_c, the R value should be recalculated if T_c is adjusted for a given return period.

7.5.3 INSTRUCTIONS FOR UNIT HYDROGRAPH METHOD

1. Delineate the watershed boundaries on the watershed base map.
2. Trace the paths of the major watercourses in the watershed on the base map.
3. If the watershed has more than one land-use, define the areas of the different land-use types:
   a. Urban,
   b. desert/rangeland,
   c. mountain, and
   d. irrigated agriculture.
4. Determine whether the watershed can be treated as a single, hydrologically homogeneous watershed, or if it must be divided into modeling sub-basins. This decision should consider the following factors:
   a. topography (and channel slope),
   b. land-use,
   c. diversity of soil texture (from Section 7.4.2.3),
   d. occurrence of rock outcrop,
   e. existence of drainage and flow control structures within the watershed such as detention/retention basins, elevated highway cross-drainage structures, channelized and improved watercourses, etc.,
   f. shape of the watershed, and
   g. needs for use of the hydrologic model, such as investigation and planning for future development and downstream drainage structures.

5. If the watershed is to be divided into modeling sub-basins, use the information from Steps 2, 3, and 4 to delineate the sub-basin boundaries.
6. For the watershed or each modeling sub-basin, determine the following:

   A - area, in square miles,

   L - length of the flow path to the hydraulically most distant point, in miles,

   L_{ca} - length along L to a point opposite the centroid, in miles,

   S - average slope of L, in feet/mile, and

   $RTIMP$ - effective impervious area, in percent.

7. Calculate T_c depending on the type of watershed:

   desert/mountain

   $$T_c = 2.4A^{0.1}L^{0.25}{L_{ca}}^{0.25}S^{-0.2} \quad (7.18)$$

   agricultural fields

   $$T_c = 7.2A^{0.1}L^{0.25}{L_{ca}}^{0.25}S^{-0.2} \quad (7.19)$$

   urban

   $$T_c = 3.2A^{0.1}L^{0.25}{L_{ca}}^{0.25}S^{-0.14}RTIMP^{-0.36} \quad (7.20)$$

8. Calculate R:

   $$R = 0.37{T_c}^{1.11}L^{0.80}A^{-0.57} \quad (7.21)$$

9. Enter the values of T_c and R in the UC record for the watershed or each sub-basin.
10. Determine whether the time-area relation will be developed from an analysis of the watershed or whether a dimensionless synthetic time-area relation will be used.
    a. If the time-area relation is to be determined by analytic means, proceed with the analysis and input the incremental areas (or percentages of total area) in the UA record.
    b. If the dimensionless synthetic time-area relations are to be used (Figure 7.19 and Table 7.11):
       i. use the values for Curve A in the UA record if the watershed or sub-basin is urban or predominantly urban,
       ii. use the values for Curve C in the UA record if the watershed or sub-basin is desert/rangeland or is mostly desert/rangeland with some mountains and/or some irrigated agricultural fields interspersed in the lowlands, and

iii. use Curve B for all other applications (Curve B is the HEC-1 default relation and the UA record is not needed).

7.5.3.1 Example: Clark Unit Hydrograph Method Parameters

Refer to Appendix A.4 for an example of development of parameters for application of the Clark Unit Hydrograph Method.

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7.6 CHANNEL ROUTING

7.6.1 INTRODUCTION

Channel routing describes the movement of a flood wave (hydrograph) down a watercourse. As a flood wave passes through a river reach, the peak of the outflow hydrograph is usually attenuated and delayed due to flow resistance in the channel and the storage capacity of the river reach. Channel routing is used in flood hydrology models, such as HEC-1, when the watershed is modeled with multiple sub-basins and runoff from the upper sub-basins must be routed through a channel, or system of channels, to the watershed outlet. Several methods are available for channel routing. The method that is recommended for the majority of channel routing applications for drainage in Mohave County is the Normal Depth method.

7.6.2 PROCEDURE

7.6.2.1 Overview

Applicability

- Channel routing is to be used in multiple sub-basin models when the runoff from the upper sub-basins passes through a relatively long watercourse, or a system of watercourses, to the watershed outlet.

Key Analysis Parameters

ANCH Manning's roughness coefficient for the main channel (dimensionless).

ANL Manning's roughness coefficient for the left overbank (dimensionless).

ANR Manning's roughness coefficient for the right overbank (dimensionless).

ITYP Specifies type of initial starting condition; either storage, discharge, or elevation.

NMIN Integer number of minutes in the hydrograph tabulation interval.

NSTPS Number of steps to be used in the storage routing.

RSVRIC Value of route starting condition; either storage in acre-feet, discharge in cubic feet per second, or elevation in feet.

RLNTH Reach length, in feet.

SEL Estimated slope of the energy gradeline, in ft/ft.

7.6.2.2 Method

The recommended method for routing is the Normal Depth method and that method should be used unless there is good cause for deviation from this recommendation. The following procedure is for the Normal Depth method; however, the information can often be used to assist in defining routing input for other methods. For Normal Depth routing, data must be provided for the number of steps in the routing calculation, the initial condition of the flow in the channel, channel resistance coefficients, and channel geometry. Much of this data is normally obtained from appropriate maps and/or field survey data.

7.6.2.3 Number of Computation Steps (NSTPS)

This is the number of computation steps that will be used in the Normal Depth routing calculation. The Normal Depth route operation in HEC-1 is accomplished by use of a single 8-point cross section which is selected to be typical of the routing reach. Storage routing is accomplished by using wedge-storage for sub-reaches. The sub-reach length is the distance traveled by the flood wave during one computation time interval (*NMIN*). The number of necessary sub-reaches corresponds to NSTPS, which must be an integer. *NSTPS* can be estimated by reach length/average velocity/*NMIN*. Refer to Section 7.9.2 for guidance for using HEC-1, and Section 7.9.2.9, for additional guidance in selecting *NSTPS*.

7.6.2.4 Initial Flow Condition (ITYP and RSVRIC)

These variables define the initial condition of the flow in the channel at the start of the routing computation. Normally the initial condition that is used is the discharge in the channel and this will typically be ‘0’ (dry channel) for channels in Mohave County. If the channel is expected to have flow prior to the modeled storm, or a baseflow, then enter the appropriate discharge data. The channel water surface elevation at the start of the routing computation option can be used, if desired instead of the initial discharge condition option.

7.6.2.5 Routing Reach Length (RLNTH)

This is the length of the channel or major flow path. The length is to be measured on the best available map. The units of *RLNTH* are feet. It is the length of the flow path following the dominate velocity vector, which can vary with storm frequency.

7.6.2.6 Energy Grade Line Slope (*SEL*)

This is the slope of the energy grade line and is not normally known. For normal flow, it is parallel to the channel bed slope. It is usually estimated as the channel bed slope, calculated by dividing the difference in bed elevation between the upper and lower ends of the watercourse by the routing reach length. The units of *SEL* are feet/foot.

7.6.2.7 Manning's Roughness Coefficient (*n*)

The Manning's roughness coefficient, *n*, is a measure of the flow resistance of a channel or overbank flow area. The flow resistance is affected by many factors including size of bed material, bed form, irregularities in the cross section, depth of flow, vegetation, channel alignment, channel shape, obstructions to flow, and quantity of sediment being transported in suspension or as bed load. In general, all factors that retard flow and increase turbulent mixing tend to increase n. Manning's *n* must be assigned for the main channel and the left and right overbanks for application of the normal depth channel route method. The Manning's roughness coefficient for the main channel is designated as *ANCH*, for the left overbank it is *ANL*, and for the right overbank it is *ANR* according to HEC-1 nomenclature. Refer to Section 13 for guidance in assigning n-values to the main channel and overbanks.

7.6.2.8 Channel Geometry

The channel geometry is to be provided by an 8-point cross section. That cross section is to be representative of the hydraulic characteristics throughout the routing reach. Considerable judgment is necessary in defining the representative 8-point cross section. The guidance in the HEC-1 User's Manual should be followed. The coordinates (X and Y) can be to any base datum. Specifically, the vertical dimensions (Y) do not need to correspond to land surface elevation or any elevation for any location along the routing reach.

7.6.3 APPLICATIONS AND LIMITATIONS

Channel routing is to be used in multiple sub-basin models when the runoff from the upper sub-basins passes through a watercourse, or a system of watercourses, to the watershed outlet. Routing should be used in models when a major component of watershed runoff (an inflow hydrograph) enters a relatively long channel and must flow through that channel to the watershed outlet or to a point along the channel where a flood hydrograph is desired. In those situations, the peak of the outflow hydrograph is usually attenuated and delayed compared with that of the inflow hydrograph.

The Normal Depth method, that is available in the HEC-1 program, is usually an appropriate routing method for use iwith watercourses in Mohave County. It should be used where routing effects (peak attenuation and delay) are expected. Other methods may be more appropriate or more practical in certain applications. For example, the Kinematic Wave channel routing method can often be used with comparable accuracy for constructed urban channels, including storm drains, and for short, steep natural channels. The Muskingum method may be appropriate for certain rivers if data are available to determine the two parameters (*K* and *X*) by analysis, or by HEC-1 optimization from recorded hydrographs, or if other information is available to yield reliable estimates of *K* and *X*. The Muskingum-Cunge method is also available and it can be used in certain applications. However, the Muskingum-Cunge method can produce unreliable results, particularly for wide, shallow water courses, especially with steep slopes. The use of the Muskingum-Cunge method must be applied with caution, and results carefully reviewed before acceptance. Also, the Muskingum-Cunge method is not amenable for channel routing if channel transmission losses (by the recommended method, see Section 7.8) are to be included in the watershed model. In general, however, the Normal Depth method is to be used. If other methods are considered, technical guidance can be found in FHWA (2002), Chapter 7, USACE (1998), and Hoggan (1997). Use of other methods requires prior approval by COUNTY/DISTRICT.

One of the most critical aspects of watershed modeling using sub-basins and channel routing is the selection of channel routing lengths (*RLNTH*). The numeric procedure used in routing calculations requires that the travel time through each routing reach be a multiple of the selected computation interval (*NMIN*). For this reason, the selection of too short a *RLNTH* could result in the computation of zero travel time through the routing reach (instantaneous translation of the flood wave through the reach). This could result in erroneously large peak discharges at downstream concentration points in the watershed model. A watershed model of numerous small sub-basins and connecting short routing reaches can result in progressively larger overestimation of peak discharges in a downstream direction producing grossly overestimated peak discharge at the watershed outlet. Section 7.9.2.4, Routing Lengths, should be consulted prior to watershed delineation to avoid problems with channel routing lengths that are too short.

7.6.4 INSTRUCTIONS FOR CHANNEL ROUTING

The following steps should be used with the Normal Depth routing method:

1. From the watershed base map, identify the routing reaches. (See Section 7.9.2.9, Routing Lengths, for additional guidance.)
2. Compile information on the characteristics of those reaches (detailed topographic maps to define channel geometry, photographs of the channels and overbanks, other hydrologic reports for the area, etc.)
3. Conduct a field reconnaissance of the watershed and routing reaches, if practical. Observe and note the characteristics of the routing reaches; variations in the channel cross sections, irregularity of the channel, and degree of meandering of the main channel. Determine the hydraulically representative section of the routing reaches. Make note of and photograph the representative sections paying particular attention to flow resistance characteristics; bed material, obstructions to flow (rock outcrop, boulders, debris, etc.), and vegetation in the channel and overbank floodplains. If adequate maps are not available to define the channel geometry of the representative sections, field surveys or field measurements can be made of the channel and overbank floodplains.
4. Prepare a sketch of the representative section of each routing reach, and prepare the channel geometry input (RX and RY records).
5. Estimate the main channel roughness coefficient, *ANCH*, using the information and procedures in Chapter 13.
6. If an 8-point cross section is used that contains overbank floodplains, select the *n* for each of the overbanks (*ANL* and *ANR*), also using the information and procedures in Section 13.
7. Measure the routing reach length, *RLNTH*, from the base map.
8. Estimate the energy gradient (*SEL*) by calculating the channel bed slope from the base map.
9. Input the routing information into the RS, RC, RX and RY records.

7.6.5 NORMAL DEPTH CHANNEL ROUTING EXAMPLE

Refer to Appendix A.5 for an example of development of parameters for application of the Normal Depth channel routing method.

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7.7 STORAGE ROUTING

7.7.1 INTRODUCTION

Storage routing will be used when inflow to a structure is temporarily detained by the storage capacity and/or outlet characteristics of the structure such that the outflow is significantly different than the inflow in terms of flow rate and time. Storage routing is required when flow is routed through retention/detention basins; where flow passes through drainage facilities such as roadway cross-drainage structures (particularly where the road is elevated on earthen fill); where culverts, railroad drainage facilities, and some bridges restrict flow rates; and pump stations.

Level-pool reservoir routing is typically used for these applications. Information must be provided on various combinations of HEC-1 input records to describe the storage capacity and discharge relations of the structure and its outlet works.

7.7.2 PROCEDURE

For storage routing, topographic, design, and/or as-built information must be available to prepare the necessary input. Because of the diversity of structures for which storage routing can be performed, only general guidance is provided for this method.

7.7.2.1 Overview

Applicability

- Routing of runoff hydrographs through volumetric storage structures.

Key Analysis Parameters

Q	Discharge for each point on a stage versus discharge curve controlling outflow from the storage structure, in cubic feet per second.
ITYP	Specifies type of initial starting condition; storage, discharge, or elevation.
RSVRIC	Value of route starting condition; either storage in acre-feet, discharge in cubic feet per second, or elevation in feet.
NSTPS	Number of steps to be used in the storage routing (normally set to 1).
NMIN	Integer number of minutes in the hydrograph tabulation interval.

V Volume for each point on a stage versus storage curve for the storage structure, in acre-feet.

ELEV Elevation for each point on a stage versus storage or stage versus discharge curve for the storage structure, in acre-feet.

7.7.2.2 Stage-Storage Relation

A relation describing the storage volume that is obtained with a specified water surface elevation must be provided. This is accomplished by one of two methods: 1) water stage (SE record) and corresponding storage volume (SV record) , or 2) water stage (SE record) and corresponding surface area for the water stored to that elevation (SA record). Ether method is acceptable and to some extent the selection depends upon the information that is available. If surface area data (SA records) are provided, the storage volume is calculated during the execution of the HEC-1 program.

7.7.2.3 Stage-Discharge Relation

A relation describing the discharge through the structure as a function of stage of water behind the structure must be provided. Discharges are entered on SQ records that correspond to water stages of the SE records. Stage-discharge relations are established by hydraulic analysis of the hydraulic control structure or from design reports.

7.7.2.4 Structure Overtopping

There are situations where structures can be overtopped due to inflow that exceeds the stage-storage-discharge relations. This can happen in a variety of situations including elevated roadway embankments with cross drainage structures that cannot pass the required inflow. Often in such cases, the excess inflow will overtop the structure, and in those cases the ST record can be used to model the flow that would pass over the structure; however, an overtopping discharge rating curve is the recommended method. The SQ record, in that case, is for the combined discharge through the structure plus overtopping discharge.

7.7.2.5 Pump Stations

A pump station may be included as a part of storage routing to withdraw water from the structure at that point. Pumped water leaves the study area unless it is retrieved and inserted in the model at another point. This can occur at depressed road intersections where the pumped water is released to a drainage structure outside of the intersection drainage boundaries. Pump stations can be modeled with WP and WR records. Pump station operation where multiple pumps and/or variable pump capacity is required to be modeled cannot be adequately modeled with HEC-1. In such cases, more sophisticated pump station models should be used. The HEC-1 model can usually be used successfully to provide the inflow hydrograph for the pump station analysis. The hydrograph diversion operation could also be used to simulate a pump station.

7.7.3 INSTRUCTIONS FOR STORAGE ROUTING

1. Define the stage-storage relation from the most appropriate maps and input the relation in SE and SV records, or in SE and SA records.
2. Define the stage-discharge relation for the outflow through the structure by use of the SQ record. Care must be taken if the structure is subject to emergency spillway flows or overtopping. The use of an SO record will suppress all data entered on an SS record (spillway characteristics). However, flows taken from an SQ record will be added to any flows computed from the ST record (top-of-dam overflow).

   The recommended approach is to use SQ/SE records to define the complete discharge rating curve for all types of discharge through (or over) the structure. These input calculations should be performed manually for each of the different types of discharge that could occur. A composite discharge rating curve should then be developed by adding together all applicable discharges that occur at any given elevation. This discharge rating curve should extend above the maximum reservoir water surface elevation achieved during the routing operation.
3. If pump stations are included, and if the pump station capability of the HEC-1 program is adequate for the analysis, provide pump station information in WP and WR records, or simulate using a diversion operation.

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7.8 TRANSMISSION LOSSES

7.8.1 INTRODUCTION

Storm runoff and floods in Mohave County are usually attenuated through the effects of channel and storage routing, but they are often also diminished due to the percolation of water into the bed, banks, and overbank floodplains of the watercourses. These losses in the watercourses are transmission losses, and are losses that accrue in the watershed in addition to the rainfall losses on the land surface. Transmission losses can, and often do, result in a significant reduction in the runoff volume. Often, transmission losses only result in a relatively small reduction in flood peak discharge; however, there are situations, such as very long, wide channels with high percolation rates, where the flood peak discharges are dramatically reduced.

The magnitude of transmission loss (both volumetric and peak discharge) is dependent upon the antecedent conditions of the watercourse; characteristics of the bed, bank, and overbank materials; channel geometry (wetted perimeter); depth to bedrock; depth to the ground water table; duration of flow; and hydrograph shape. For a watercourse that is initially dry and is composed of coarse, granular material, the initial percolation rate can be very high; however, the percolation rate diminishes during passage of the flood and would eventually reach a steady-state rate if the flow continues long enough.

Although it is recognized that transmission losses can be an important element in performing rainfall-runoff modeling, particularly for ephemeral watercourses in Mohave County, procedures and reliable data for estimating transmission losses are poor. Therefore, except for situations where transmission losses should clearly be incorporated in the analysis, the estimation of these losses will not usually be incorporated in rainfall runoff models. The incorporation of transmission losses in a watershed rainfall-runoff model should be approved in advance by Mohave County and the procedure and assumptions clearly documented.

Two options in the HEC-1 program are available for estimating transmission losses. Both options use the RL record. The recommended option uses an estimated channel percolation rate (*PERCRT*) and must be used with the Normal Depth channel storage routing option (RS/RC record). The second option estimates the transmission loss as a constant loss (*QLOSS*), in cfs, plus a ratio (*CLOSS*) of the remaining flow after subtracting *QLOSS*. The second method can be used with any of the HEC-1 channel routing options; however, that method is not recommended for general use because of the very subjective decisions that will need to be made in selecting *QLOSS* and *CLOSS*. The recommended method is physically-based and should result in better estimates of transmission losses, if adequate estimates can be made of the percolation rate and if the necessary storage routing information can be satisfactorily represented.

7.8.2 PROCEDURE

7.8.2.1 Overview

Applicability

- This approach should not normally be applied for design studies.
- This approach may be applied for flood insurance studies, but only after careful consideration of all the points listed in Section 7.8.2.2. Use of this approach requires prior approval by COUNTY/DISTRICT.

Key Analysis Parameters

PERCRT Percolation rate for the wetted surface of the channel, in cfs/acre.

QLOSS Constant discharge loss for entire routing in cfs. Subtracted from every ordinate of the inflow hydrograph.

7.8.2.2 Method

The following conditions should be met for the consideration of the incorporation of transmission losses into a rainfall-runoff model of a watershed:

1. The bed, banks, and overbank floodplains of the watercourse are composed of coarse, granular material. Materials such as cobble, gravel, sandy gravel, gravelly sand, sand, and sandy loam are all indicators that appreciable transmission losses can occur.
2. There is a relatively long total length of watercourse that is composed of coarse, granular material.
3. The watercourse is ephemeral and it is prudent to assume that the watercourse is dry before the onset of the storm.
4. The bed of the watercourse is not underlain by material, such as bedrock, that would inhibit the sustained percolation of water into the bed of the watercourse.
5. The depth to ground water is great enough to not inhibit the sustained percolation of water into the bed of the watercourse.

If the above conditions are met, then the incorporation of transmission losses into the model should be considered. At this point, two other factors should be considered before proceeding:

1. Incorporation of transmission losses requires a multiple sub-basin model with defined routing reaches. Transmission losses are calculated for the routing reaches. Use of the recommended option for calculating transmission losses with the HEC-1 program requires normal depth storage routing. Transmission losses are to be considered only if a multiple sub-basin model is acceptable.
2. Adequate information must be available to provide input for the storage routing method, and the percolation rate can be satisfactorily estimated.

If the above conditions are met, and if it is determined that modeling of transmission losses are vital and practical to the development of a rainfall-runoff model, then proceed to incorporate transmission losses in the model. This will require input of the necessary normal depth storage routing information on RC, RX, and RY records. The elevation of the channel invert (*ELVINV*) must correspond to the lowest elevation that is used in the 8-point cross section for that routing reach.

The transmission loss will be calculated using information from the RL record (*PERCRT* and *ELVINV*). Very little guidance is available for estimating the percolation rates (*PERCRT*), which can vary from more than 5 inches per hour to less than an inch per hour. Table 7-1 provides some guidance for the percolation rate that can be expected in channels of various bed materials. The *XKSAT* values for the NRCS soil map units covering the wash in question can also be used as an estimate for *PERCT*. However, the engineer/hydrologist should be aware that the values presented in Section 7.4.2.3 and Appendix D are based on a maximum value for *XKSAT* of 2.0 inches per hour. The true value of *XKSAT* for sands and gravels, estimated using the equations in Saxton and Rawls (2005), should be used for estimating transmission losses. These values should correspond well with the general guidelines presented in Table 7.12.

Table 7.12 Percolation rates for various channel bed materials

(from NRCS (2007))

Bed Material	Transmission Loss Class	Percolation Rate, *PERCT* Inches/hour
(1)	(2)	(3)
1. Very clean gravel and large sand.	Very High	>5
2. Clean sand and gravel, field conditions.	High	2.0 - 5.0
3. Sand and gravel mixture with low silt-clay content.	Moderately High	1.0 - 3.0
4. Sand and gravel mixture with high silt-clay content.	Moderate	0.23 -1.0
5. Consolidated bed material; high silt-clay content.	Insignificant to Low	0.001 - 0.10

7.9 MODELING TECHNIQUES

7.9.1 INTRODUCTION

7.9.1.1 General Discussion

Practical application of the rainfall-runoff modeling procedures in this manual can be accomplished through use of the HEC-1 Flood Hydrograph Package (U.S. Army Corps of Engineers, 1998). This computer program, which is available from the U.S. Army Corps of Engineers Hydraulic Engineering Center web site at: http://www.hec.usace.army.mil/, the National Technical Information Service, and several commercial program vendors, provides modeling capability for the hydrologic procedures that are specified in this manual. This program is a working part of the DDMSW computer program, which is the Mohave County standard tool for implementing the hydrologic methodology. This section contains an overview of the major theoretical assumptions upon which the HEC-1 computer program is based, and the resultant limitations. Watershed modeling techniques are presented, and these are related to some of the common coding errors that are often made when using the HEC-1 program. A modeler's/reviewer's HEC-1 checklist is presented in Appendix E, Checklist 2, for use by both Mohave County engineers and consulting engineers in developing and reviewing HEC-1 watershed models.

A user's working knowledge of the following areas is assumed:

1. Surface water hydrology and watershed modeling.
2. Basic input data structure for the HEC-1 program.
3. Procedures presented in this manual.

7.9.1.2 Applicable HEC-1 Versions

There are many versions of the HEC-1 computer program available and in use. Care should be taken by the user to obtain and use a version containing the desired capabilities. The HEC-1 program was originally developed by the U.S. Army Corps of Engineers, Hydrologic Engineering Center (HEC) in 1968. Since that time, there have been five significant updates and numerous error corrections. The program was originally written for main frame computers and has since been ported to a number of different platforms. This discussion is specific to the Microsoft Windows versions. The following is a brief synopsis of the releases made since 1988:

1988 Version -

1. The Green-Ampt infiltration equation was added as an option.
2. The Kinematic Wave runoff computations were improved.
3. All the main-frame computer options were made available in the PC version.
4. A program bug is present in the application of the Green and Ampt equation in combination with the JD record option.

1990 Version -

1. Muskingum-Cunge channel routing was added as an option.
2. Detention basin modeling capabilities were improved.
3. The Green and Ampt error from the 1988 version was corrected.
4. A program bug is present in the Kinematic Wave runoff procedure when using the JR record option. Hydrographs do not combine properly.

1991 Version -

1. This version is specific to the 80386/80486 microprocessors and requires a minimum of 2.5 megabytes of total memory, or 640 kilobytes of memory and 3 megabytes of disk space.
2. The Kinematic Wave error from the 1990 version was fixed.
3. The number of hydrograph ordinates available was increased from 300 to 2,000 (1998 Version).

The functional differences between the 1990 version of HEC-1 and the 1998 version are not significant. Several small errors were corrected.

The 1998 version of the HEC-1 program should be used for Mohave County rainfall-runoff watershed modeling purposes.

7.9.1.3 Assumptions and Limitations of HEC-1

Proficiency in use of the HEC-1 program requires an understanding and appreciation of the basic underlying assumptions and limitations. The key assumptions of the program are as follows:

Deterministic

The rainfall-runoff process is stochastic (non-deterministic); however, the HEC-1 program treats the process as deterministic. Randomness of the process (within both the temporal and spatial domain) is not considered. The effects of natural variability can be investigated by making numerous runs of a HEC-1 model using a systematic approach to varying critical input variables.

Lumped Parameters

Many of the model parameters, for example the Green and Ampt infiltration parameters represent spatial averages. These are "lumped" parameters that are intended to represent average conditions for a watershed subarea, not values at a point in the watershed.

Unsteady Flow

The flow rates forecasted by the model vary with time. The key limitations of the program are as follows:

1. **Single Storm**: A single storm event is modeled. Provisions are not available for soil moisture recovery between independent storms or between bursts of rainfall within a single storm.
2. **Hydrologic Routing**: All routing (channel and storage) is by hydrologic methods. Hydraulic routing (the use of the St. Venant equations) is not performed.
3. **Results**: The results are in terms of discharges and runoff volumes. Accurate water stages are not provided for channel flow. The water stages for reservoir routing do meet the standards of the profession for accuracy (except in the tailwater reach of the reservoir where gradually varied flow would exist).

7.9.2 WATERSHED MODELING

7.9.2.1 Instructions for Watershed Modeling Process

The following general steps are encouraged in performing rainfall-runoff modeling:

1. Collect all pertinent information for the watershed, including:
   a. maps,
   b. aerial photographs,
   c. soil surveys,
   d. land-use maps/reports,
   e. reports of flooding,
   f. streamflow data (if available), and
   g. reports of other flood studies (FEMA, county, etc.).
2. Prepare a watershed base map using the best available topographic map and most practical map scale.
3. Perform a preliminary sub-basin delineation.
4. Conduct a field reconnaissance.
5. Finalize the sub-basin delineation.
6. Prepare the rainfall input.

7. Prepare the rainfall loss input.
8. Prepare the unit hydrograph input.
9. Prepare all routing input.
10. Prepare a preliminary logic diagram.
11. Prepare HEC-1 input file.
12. Debug the model, using the HEC-1 checklist in Appendix E as a guide.
13. Optimize the hydrograph routing operation NSTPS parameter.
14. Calibrate the model, where possible.
15. Execute the HEC-1 model.
16. Check results using indirect methods for discharge verification (Section 7.11).
17. Evaluate the model and results based on available information.
18. Revise the model, as appropriate, to best represent actual watershed conditions. Model sophistication, such as incorporation of transmission losses, is usually added to the model at this point, but could be part of steps 13 and 14.
19. Execute the final HEC-1 model.
20. Make final model verifications and evaluations.
21. Revise the logic diagram.
22. Prepare a report.

7.9.2.2 HEC-1 Logic Diagram

A schematic diagram for multiple sub-basin models should be prepared and included as a part of the final report. This diagram symbolically depicts the order of combining and routing hydrographs. The data to be included are:

1. sub-basin data (sub-basin name, area, T_c,)
2. channel routing data (length, slope, average "n" value, base width and/or other dimensions, average velocity, transmission loss rate, peak discharge), and
3. storage routing data (maximum stage, maximum storage).

7.9.2.3 Model Time Base and Computation Interval

The model time base and computation interval are controlled by the NMIN and NQ variables, which are input in the IT record. These variables are defined as:

NMIN The integer number of minutes in the tabulation interval used to define the temporal spacing of the hydrograph ordinates. This variable sets the definition of the hydrograph. Too large a value will result in inaccuracies in peak discharge and runoff volume estimates. NMIN should be in the range of 0.10 to 0.25 times the shortest sub-basin T_c in the HEC-1 model. NMIN must be evenly divisible into 60 and MUST be an integer.

The following general criteria are recommended for NMIN:

NMIN = 1 or 2 minutes for a 6-hour storm duration (drainage area less than or equal to 20.0 square miles), and

NMIN = 5 minutes for a 24-hour storm duration (drainage area greater than 20.0 square miles).

NQ NQ is the integer number of hydrograph ordinates to be computed. There are a maximum of 2,000 allowed for the 1998 version of HEC-1. The total time base for the model is therefore NQ x NMIN, and this product must be greater than the total storm duration specified on the PH record plus the total routed travel through the watershed.

When using a 6-hour storm duration and NMIN = 2 minutes, NQ can usually be set at 200. If NMIN is larger than 2 minutes, NQ can be less than 200. If NMIN is 1 minute, then NQ must be at least greater 360, up to a maximum of 2,000.

When using a 24-hour storm duration and NMIN = 5 minutes, NQ will normally be 300. If NMIN is larger than 5 minutes, NQ can often be less than 300. If NMIN is less than 5 minutes, then NQ must be greater than 300, up to a maximum of 2,000.

Refer to Appendix E for a HEC-1 checklist that contains guidance on inspection of HEC-1 output for determination of the adequacy of the NMIN and NQ values, and guidance on alterative selections of NMIN and NQ.

7.9.2.4 Sub-basin Delineation

The process of breaking down a watershed into sub-basins should be done with careful consideration given to several critical factors. Defining these factors prior to beginning the delineation will help to ensure that the model remains within the limitations of the methodology used. It will also help avoid extensive revisions after the fact. These factors are as follows:

<u>Concentration Points</u>

Identify locations where peak flow rates or runoff volumes are desired. The following locations, at a minimum, should be considered:

1. confluences of watercourses where a significant change in peak discharge may occur,
2. drainage structures and flood retarding structures,
3. crossing of watercourses with major collector or arterial roadways, and
4. jurisdictional boundaries.

Sub-basin Size

Using the concentration point locations, estimate a target average sub-basin size to strive for, and estimate the smallest expected sub-basin.

Time of Concentration, NMIN, NQ

Estimate the time of concentration (T_c) for the smallest sub-basin. Using this value, determine the integer number of minutes (NMIN) for the computation interval, which should be in the range of $0.1T_c$ to $025T_c$. Then estimate the number of hydrograph ordinates (NQ) required to provide an adequate time base for the HEC-1 model. As a minimum, NQ should be greater than the longest T_c value plus the total reach route travel time for the model. This should capture all the time to peak for each hydrograph operation. Ideally, NQ should be greater than the sum of the storm duration and the total reach route travel time for the model.

Homogeneity

Considerations for sub-basin homogeneity in order to meet the Lumped Parameter assumption are:

1. The sub-basin sizes should be as uniform as possible.
2. Each sub-basin should have nearly homogeneous land-use and surface characteristics. For example, mountain, hillslope, and valley areas should be separated into individual sub-basins wherever possible.
3. Soils and vegetation characteristics for each sub-basin should be as homogeneous as reasonably possible.

The average sub-basin size may need to be adjusted (addition of concentration points) as required, in order to satisfy the key assumptions upon which the HEC-1 model is based.

Routing Lengths

The length of the channel reaches defined as a result of the delineation should be considered while breaking down the watershed. A key parameter used in routing a hydrograph through a channel reach is the number of steps (NSTPS). Although this is most important for channel storage routing using the Normal Depth option, it is also a good check to use when applying the Muskingum Cunge method. The minimum reach length should satisfy the following expression:

$$L = (NSTEPS)(V_{avg})(60)(NMIN) \qquad 7.22$$

where:

L	=	the minimum reach length, in feet.
NSTPS	=	an integer with a minimum value of 1, but preferably more than 1.
V_{avg}	=	an estimate of the average velocity, in feet/second.
NMIN	=	the integer number of minutes for the computation interval.

Equation 7.22 is intended to be used as a guide in estimating the minimum channel routing length ($RLNTH_{min}$) before delineating sub-basins in a multi-basin watershed model. The use of Equation 7.22 to estimate the minimum reach length in the model can improve modeling accuracy and will minimize routing instability warnings in the model output. Section 7.6.3 should be consulted for discussion of problems that may result if this recommendation is not followed.

7.9.2.5 Precipitation and Rainfall Distributions

Precipitation and rainfall distributions for watershed modeling in Mohave County are implemented in HEC-1 using the PH and JD records. The PH record, without JD records, may be used for a single basin model and for multiple basin models with a small total watershed area not requiring areal reduction (less than 5 square miles total). JD records are used in conjunction with the PH record for multiple basin models with a total watershed area greater than 1 to 5 square miles, at the discretion of the modeler. Field 1 of the PH record is left blank for HEC-1 models using NOAA Atlas 14 point precipitation data. Field 2 is to be left blank because the TP-40 depth-area reduction curves are not used for Mohave County. The point precipitation values entered in fields 3 through 10 should be reduced manually for single basin models of a large watershed (greater than 1 to 5 square miles) using the depth-area reduction curves in Table 7.3.

If design discharges are needed at existing internal concentration points in multiple basin models, then the JD record option is to be used. Instructions in USACE (1998) for use of the JD record option in conjunction with the PH record for rainfall should be consulted. Insert the correct unreduced precipitation values in Fields 3 through 8 of the PH record for a 6-hour storm, or use Fields 3 through 10 of the PH record for a 24-hour storm. Implement the depth-area reduction factors from Table 7.3 using JD records.

7.9.2.6 Rainfall Losses

Keep in mind that the rainfall loss parameters are averages, assumed to be evenly distributed, for the sub-basin. The percent impervious value (*RTIMP*) is the percent of the sub-basin area for which one hundred percent runoff will be computed. This means that the impervious area is assumed to be hydraulically connected to the concentration point. This parameter should be used with care. For urban areas, *RTIMP* is the effective impervious area which is usually less than the actual total impervious area. Rock outcrop is not often directly connected to the watershed outlet. Care must be exercised when estimating *RTIMP* for rock outcrop.

7.9.2.7 Time of Concentration

Certain watersheds may require estimation of several T_c's for different hydraulically most distant points. Use the largest T_c value that is calculated for the different flow paths that are considered. Since the Unit Hydrograph Method is extremely sensitive to the T_c parameter, every time of concentration estimate should be checked for reasonableness. Because of the numerous watershed characteristics that influence T_c, verification of this parameter can be difficult. However, an evaluation of average flow velocities through a sub-basin can yield worthwhile information on the validity of the computed T_c value.

Any attempt to verify T_c calculations by using an average flow velocity analysis should be pursued with caution. Due to the large influence that overland flow travel time has on the sub-basin T_c, an average flow velocity that is computed as simply L/T_c, where L is the length of the sub-basin watercourse to the hydraulically most distant point, will normally yield an average velocity that will appear unrealistically low for the open channel flow component of the T_c value. Since overland flow velocities are normally on the order of a few tenths of a foot per second, they can consume a very large proportion of the time of concentration for a sub-basin.

Case studies have shown that it is not unusual for a simple L/T_c calculation to produce average flow velocities that are on the order of 2 to 3 fps for channels with slopes in excess of three percent. Such low velocities would not normally be considered reasonable for such steep-sloped channels.

Accordingly, a velocity analysis approach should consider separating the open channel flow contribution of T_c from the overland flow portion of T_c. Average velocities can be computed for each flow regime and then applied to the flow path length that would be associated with each of these regimes. By dividing the flow path length for each regime by the average velocity for each regime, a travel time can be computed for each flow regime. The total sub-basin travel time computed by such an approach should be similar in magnitude to the estimated T_c value.

The following guidelines are suggested for computing the travel times for each flow regime:

Open Channel Flow

1. Use a 4-point trapezoidal cross-section to approximate the average main channel geometry for the sub-basin. The approximate cross-sectional geometry, depth, and roughness should be based on field inspections whenever possible.
2. Record the channel slope value that was used for the T_c calculation.
3. Apply the data from Steps 1 and 2 to Manning's equation to compute the average channel velocity that is associated with the bankfull discharge of the channel.
4. Record the length (L) of the sub-basin watercourse that was used for the T_c calculation.
5. Compute the open channel travel time by dividing the watercourse length from step 4 by the average velocity from Step 3.

Overland Flow

1. Compute the overland flow travel time with the following equation:

$$T_{OF} = 0.007(nL)^{0.8}/(P_2)^{0.5}S^{0.4} \tag{7.23}$$

where:

T_{OF} = overland flow travel time (hours)

n = overland flow roughness

L = overland flow length (feet)

P_2 = 2-year, 24-hour rainfall (inches]

S = overland flow slope (feet/foot)

Equation 7.23 is taken from NRCS (1986). Guidelines for selecting the overland flow roughness (n) are provided in the NRCS reference, as well as in the HEC-1 User's Manual. Overland flow lengths are generally less than 300 feet.

7.9.2.8 Hydrograph Combine and Divert Operations

The primary hydrograph operations available with the HEC-1 program, other than routing options, are combining and diverting of hydrographs. The combine operation is performed on the number of specified hydrographs starting with the most recent operation and extending sequentially back to previous operations. Key points to remember when using this operation are:

1. The maximum number of hydrograph locations that can be displayed using the DIAGRAM option of HEC-1, maintained in the computation stream, is nine.
2. The maximum number of hydrographs which can be combined at one time is five.
3. The total watershed area of the combined hydrographs may be entered manually in Field 2 of the HC record.

Hydrograph diversions may be used to simulate flow splits such as might occur at roadway intersections, over elevated highways, or at distributary channel apexes. Key points to remember about this operation are:

1. The split is done using a discharge rating table for the diversion with a maximum volume cutoff option.
2. The hydrograph that continues downstream in the model should be the one associated with the primary, dominant, flow channel for the wash. The diverted hydrograph should be the smaller side wash that is diverting away from the main, more dominant, channel.
3. It is very important to check the shape of diversion hydrographs for oscillations and to verify that the expected results are obtained.
4. When a diverted hydrograph is recalled into the stack, the drainage area associated with the hydrograph is zero. The HEC-1 summary tables will reflect incorrect areas unless the area is corrected using the manual area input option (Field 2 of the HC record) for the first combine operation downstream of the recalled hydrograph.

7.9.2.9 Hydrograph Channel Routing Operations

The channel routing option specified for use in this manual is the Normal Depth method. The following are considerations for use of the Normal Depth channel routing option:

Number of Calculation Steps

The NSTPS parameter must be selected with care. Normally, this parameter may be estimated iteratively as follows (this is the same approach used by DDMSW to optimize NSTPS):

1. Make an initial estimate of NSTPS for each reach using an assumed average velocity for the peak discharge. Use a rearrangement of Equation 7.22 as follows:

$$\text{NSTPS} = \frac{L}{(V_{avg})(60)(NMIN)} \quad 7.24$$

where:

L = the minimum reach length, in feet.

$NSTPS$ = an integer with a minimum value of 1, but preferably more than 1.

V_{avg} = an estimate of the average velocity, in feet/second.

$NMIN$ = the integer number of minutes for the computation interval.

2. Run the model and calculate the discharge velocity for each reach. This velocity can be approximated by the following method:

   Estimate the discharge velocity by dividing the routing length on the RC record by the difference between "Time of Peak" at the upstream and downstream routing limits. The "Time of Peak" values are listed in the Runoff Summary of the HEC-1 output file.

   The times to peak are based on multiples of the user selected computation interval (*NMIN*). This method is generally accurate to plus or minus one time step because of program rounding protocol when printing the "Time of Peak".

3. Estimate the new *NSTPS* values for each reach based on the calculated discharge velocity. Update and run the HEC-1 model

4. Perform Steps 2 and 3 until the *NSTPS* values converge. This normally occurs within three iterations.

<u>Channel Geometry</u>

Considerations, which should be checked by field reconnaissance when possible, for the Normal Depth method are:

1. All eight points on the cross section should be meaningful.
2. Be sure there is sufficient hydraulic capacity to convey the peak flow without overtopping the section.
3. Be sure that the cross section is representative of the average characteristics of the reach. If there are significant variations in section geometry, the reach should be broken down into multiple shorter reaches.
4. Verify that the Manning's n-values for the cross section are representative of the average characteristics of the reach. If there are significant variations in roughness, the reach should be broken down into multiple shorter reaches.

HEC-1 Warnings

A common warning message is the following:

WARNING Modified Puls Routing May Be Numerically Unstable For Outflows Between "Q_1" to "Q_2".

When this warning occurs, the following steps should be taken:

1. Examine the outflow hydrograph for oscillations and check the outflow peak against the inflow peak to be sure that the routed peak did not increase in magnitude. If these checks are satisfactory, then the warning can generally be considered to be satisfactorily addressed.
2. The *NMIN* variable can be reduced until the warning message goes away, or the calculated peak lies outside the specified range. However, when changing the *NMIN* value remember that this may affect other input parameters such as *NQ* and *NSTPS*.

7.9.2.10 Reservoir Routing

Modeling of reservoirs and detention basins can be accomplished using the modified Puls storage routing option of HEC-1. It is recommended that low level outlets, spillways, and structure overtopping be modeled using a discharge rating curve (SQ and SE records). The rating curve should be developed using appropriate calculation methods including using an appropriate computer program such as HEC-RAS, HY-8 (FHWA, 2007), or manual methods.

7.10 FLOOD FREQUENCY ANALYSIS

Flood frequency analysis is a procedure for computing flood magnitude frequency relations where systematic stream gaging records of sufficient length are available. The result of such an analysis can include a graph of peak discharge as a function of return period. This graph can be used to estimate the flood magnitude for selected return periods, generally between 2-year and 100-year. The resulting flood magnitude-frequency relation can be used to (1) estimate the design flood peak discharge, (2) provide estimates of flood peak discharges for the calibration or verification of rainfall-runoff models, (3) provide regional estimates of flood magnitudes that can be used to check or substantiate other methods to estimate flood magnitudes or to develop regional flood discharge relations, or (4) perform other hydrologic studies, such as the investigation of flood magnitudes from snowmelt to be used as baseflow to a watershed rainfall-runoff model. Flood gage records are sparse in Mohave County, but the period and quantity of records is increasing as Mohave County and the USGS continue to operate and monitor stream flow gages within the county. The engineer/hydrologist with a need to perform a flood frequency analysis for watersheds in Mohave County is referred to ADOT (1993), Chapter 9, “Flood Frequency Analysis”, for a detailed procedure. The USGS NSS web site at http://water.usgs.gov/software/NSS/ should also be studied.

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7.11 INDIRECT METHODS FOR DISCHARGE VERIFICATION

7.11.1 INTRODUCTION

The estimation of peak discharges by analytic methods (the Rational Method or by rainfall-runoff modeling using the unit hydrograph method is based on various assumptions, and particularly in the case of HEC-1 modeling, requires the correct input of numerous model parameters. Therefore, the resulting peak discharges that are computed by analytic methods should always be verified, to the extent possible, to guard against erroneous design discharges that can result from questionable assumptions and/or faulty model input.

Since the majority of discharge estimates are made for ungaged watersheds, usually only indirect methods can be used to check the reasonableness of discharge estimates obtained from either the Rational Method or rainfall-runoff modeling. When the watershed is gaged, or is near a gaging station, a flood frequency analysis can be performed and the results of that analysis can be used for design or used to check the results from analytic methods. The results of flood frequency analyses, because of variability of flooding in both the time and space regime, and because of uncertainties in the data and the analytic procedures, should also be checked by indirect methods.

True verification of design discharges cannot be made by any of the methods (analytic methods, flood frequency analyses, or indirect methods) because for none of these methods is there "absolute assurance" that the discharges that are obtained are the "true" representations of the flood discharge for a given frequency of flooding. However, the results of the various methods, when compared against each other and when qualitatively evaluated, can provide a basis for either acceptance or rejection of specific estimates of design discharges for watersheds in Mohave County.

In this chapter, three indirect methods are presented for checking the reasonableness of flood discharges that are obtained by either analytic methods or by flood frequency analyses. Results by either analytic methods or flood frequency analysis should always be compared and evaluated by indirect methods. There may be cases, for certain watersheds, where the flood discharges by all three methods (analytic, flood frequency analysis, and indirect) can be obtained and compared prior to making a selection of design discharge.

7.11.2 PROCEDURE

7.11.2.1 General Considerations

Three procedures are provided for obtaining indirect estimates of peak discharges for watersheds in Mohave County:

1. a graph of numerous unit peak discharge versus drainage area curves,
2. five graphs of estimated 100-year discharges and maximum recorded discharges versus drainage area for gaged watersheds in Arizona, and
3. regression equations and data graphs for flood regions in Mohave County.

In general, all three procedures should be used when verifying the results of analytic methods and/or flood frequency analyses.

7.11.2.2 Indirect Method No.1 - Unit Peak Discharge Curves

Figure 7.20 presents 7 unit peak discharge relations and envelope curves. A brief description of each of those curves follows:

A - An envelope curve, based on a compilation of unusual flood discharges in the United States and abroad (data prior to 1941), by Craeger and others (1945).

B - An envelope curve of extreme floods in Arizona and the Rocky Mountain region developed by Matthai and published by Roeske (1978).

C - An envelope curve of peak streamflow data developed for Arizona by Malvick (1980).

D - An envelope curve of peak streamflow data for the Little Colorado River basin in Northern Arizona developed by Crippen (1982).

E - An envelope curve of peak streamflow data for Central and Southern Arizona developed by Crippen (1982).

F - An envelope curve of the largest floods in the semi-arid Western United States developed by Costa (1987).

G - An envelope curve of peak discharges for Arizona, Nevada and New Mexico developed by the U.S. Army Corps of Engineers (1988).

When using Figure 7.20, it must be noted that the curves represent data sets for different hydrologic regions. The curves represent envelopes of maximum observed flood discharges.

The curves of most interest in evaluating 100-year peak discharges for Mohave County are C, D and G.

7.11.2.3 Indirect Method No.2 – USGS Data for Arizona

The U.S. Geological Survey (USGS) provides streamflow and statistical data for 142 continuous-record streamflow-gaging stations and 178 partial-record gaging stations in Arizona (Pope, Rigas and Smith, 1998). The streamflow data were analyzed by the USGS by Log-Pearson Type 3 (LP3) analyses and flood magnitude-frequency statistics are provided in that report along with the maximum recorded discharge for each of the stations. Figure 7.20 is a plot of the 100-year peak discharge (from LP3 analyses) versus drainage area (for stations with drainage areas smaller than 1,000 square miles). A line was fit to the data by regression analysis of the log-transformed data.

Figure 7.20 Peak discharge relations and envelope curves

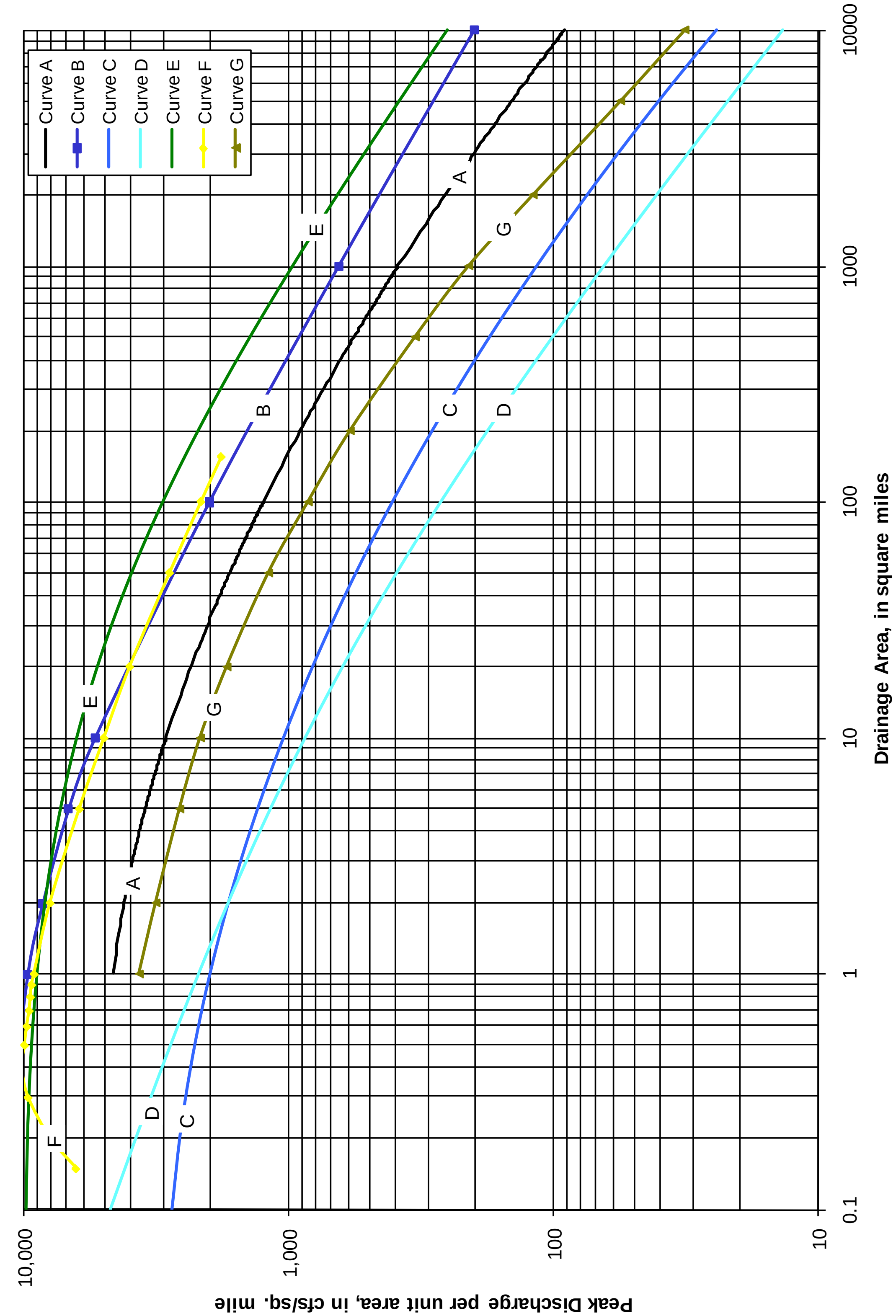


Figure 7.21 100-year peak discharges for Arizona by LP3 analysis

Drainage areas from 0.1 to 1,000 square miles

Source: Pope, Rigas and Smith (1998): Figure adapted from FCDMC (2009a)

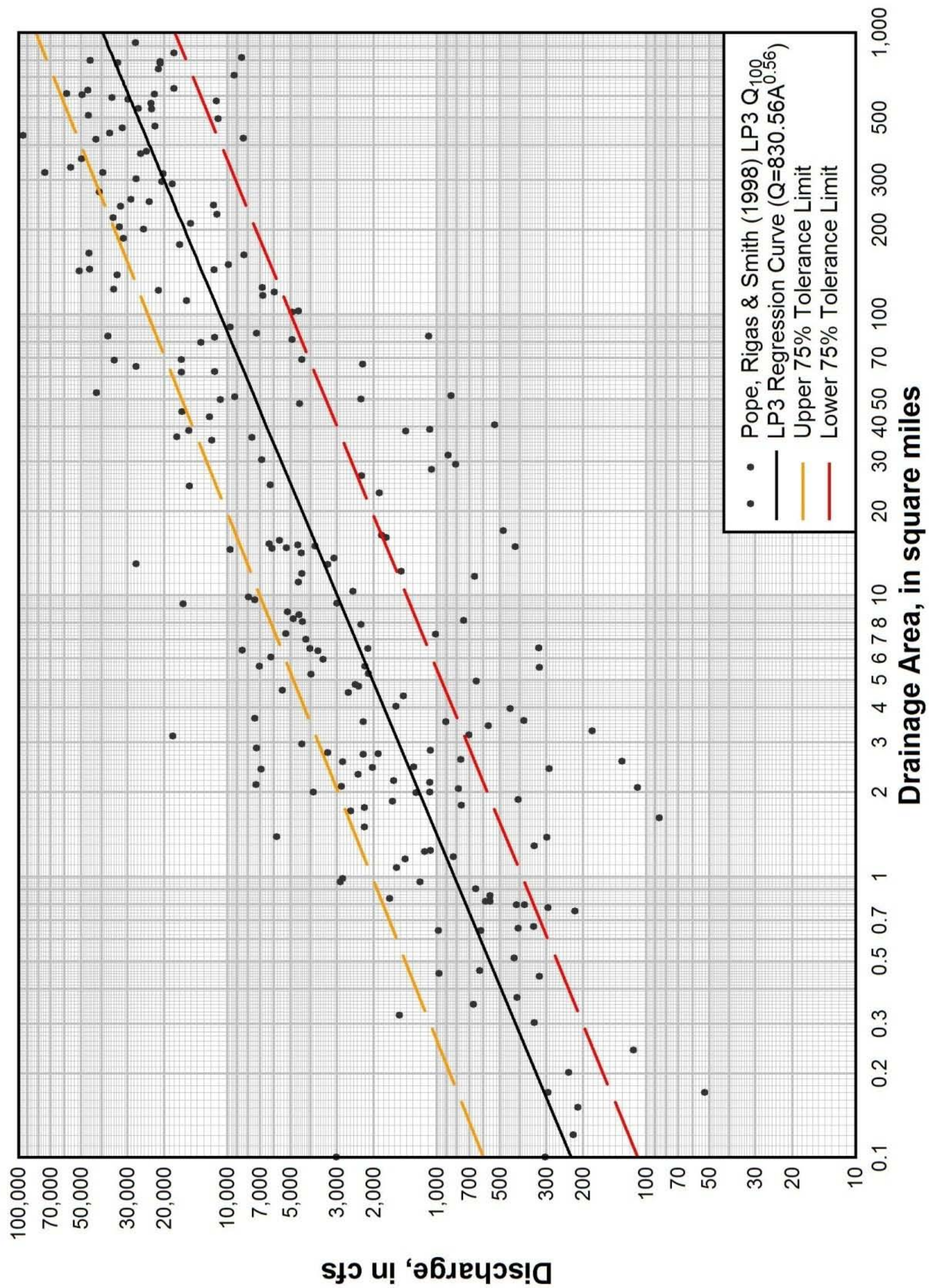


Figure 7.22 100-year peak discharges for Arizona by LP3 analysis

Drainage areas from 0.1 to 10 square miles

Source: Pope, Rigas and Smith (1998): Figure adapted from FCDMC (2009a)

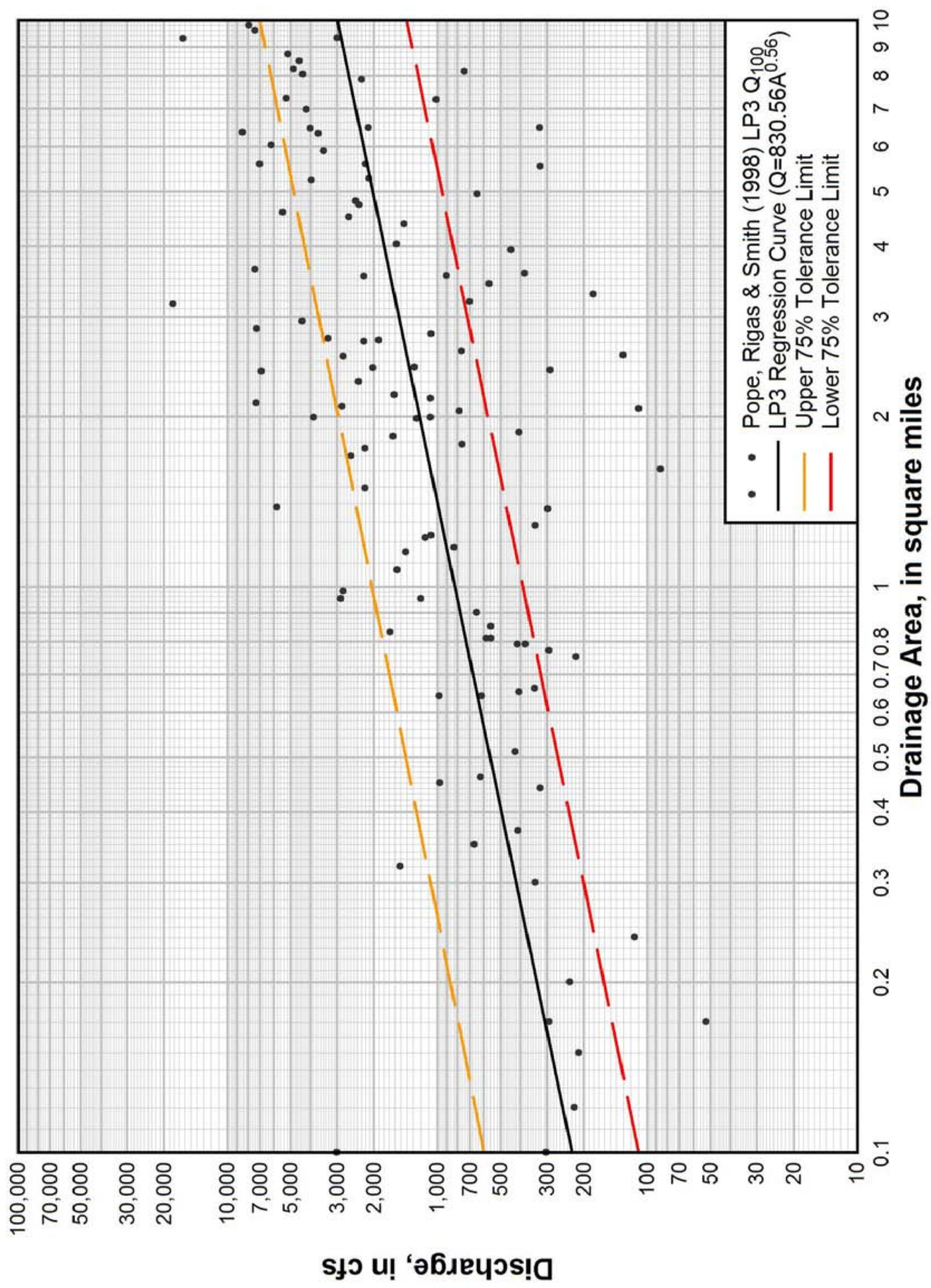
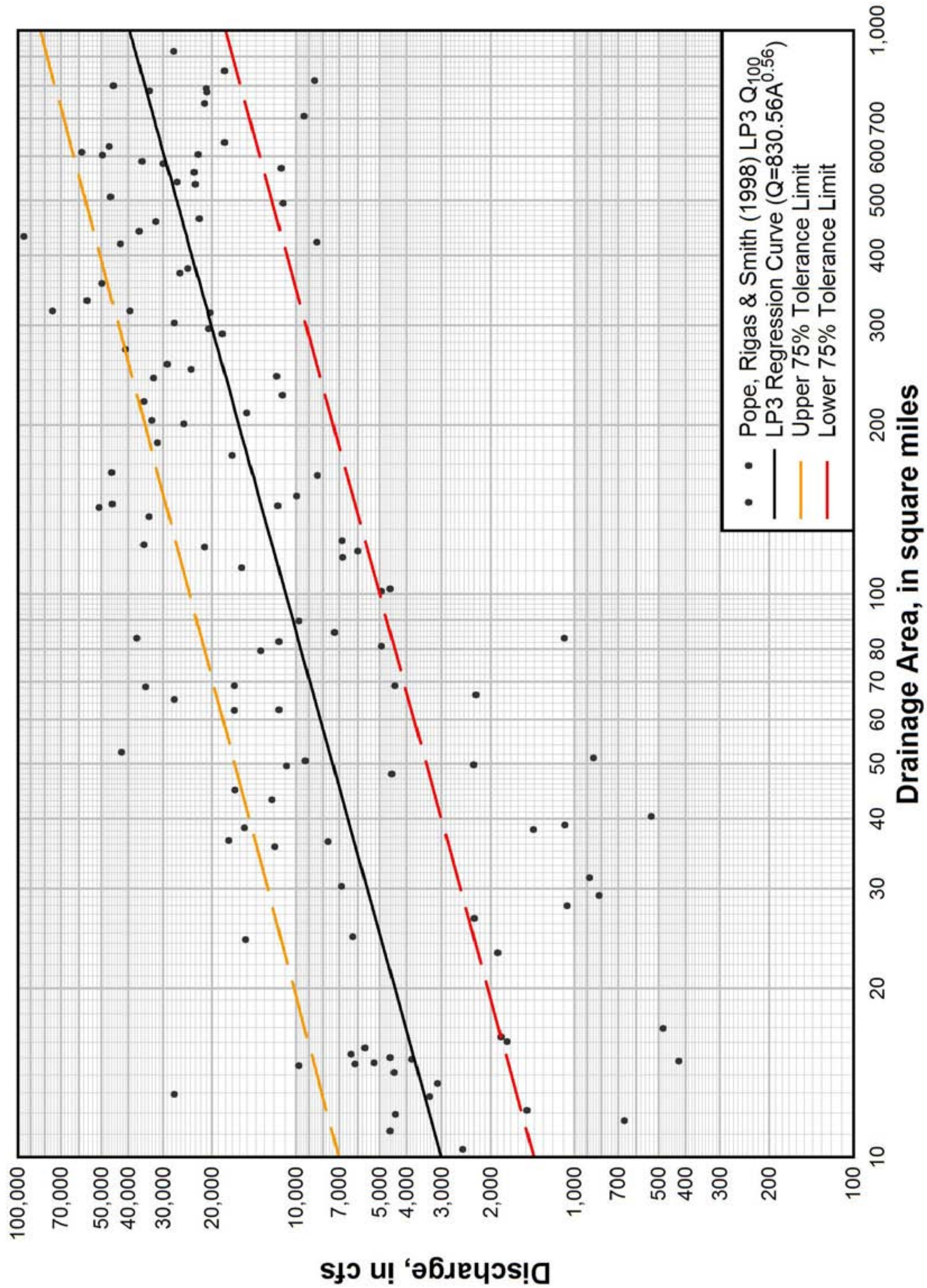


Figure 7.23 100-year peak discharges for Arizona by LP3 analysis

Drainage areas from 10 to 1,000 square miles

Source: Pope, Rigas and Smith (1998): Figure adapted from FCDMC (2009a)



The equation for the 100-year peak discharge (Q_{100}) line is:

$$Q_{100} = 830.56A^{0.56} \tag{7.25}$$

where:

A is the drainage area in square miles.

Figure 7.21 also shows 75 percent tolerance limit lines about the 100-year discharge line from Equation 7.25. A tolerance interval is an interval that one can claim to contain at least a specified proportion of the population with a specified degree of confidence (Hahn and Meeker, 1991). In this case, limits of the tolerance interval are shown for a 75 percent proportion of the expected population with a 99 percent confidence. This is useful when one is concerned with what range an individual measurement can be expected to fall within. If modeled results for a given watershed fall outside these tolerance limit lines, there may be reason to question the result or the model may require further evaluation. As an aid to using Figure 7.21, portions of that figure are reproduced with larger drainage area scales in Figure 7.22 and Figure 7.23. Those larger scale plots of the data also show 75 percent tolerance limit lines about the 100-year discharge line from Equation 7.25. A listing of the data that was used to produce Figure 7.21 through Figure 7.23 is shown in Appendix F, Table F.1. This table includes USGS streamflow-gaging station numbers, the associated drainage areas and the 100-year flood peak discharge estimates by LP3. Watershed characteristics for each of these gaging stations are provided in the USGS report by Pope, Rigas and Smith (1998). Two maps of Arizona showing the locations of the gaging stations for this data compilation are shown in Figure 7.24 and Figure 7.25. Figure 7.24 is a listing of data used from unregulated and partly regulated continuous-record streamflow-gaging stations in Arizona through USGS water year 1996. Figure 7.25 is a listing of data used from unregulated and partly regulated peak-flow partial-record streamflow-gaging stations in Arizona, also through USGS water year 1996.

Figure 7.24 Location of USGS continuous-record gaging stations in Arizona

Unregulated and partly regulated continuous-record gaging stations through water year 1996

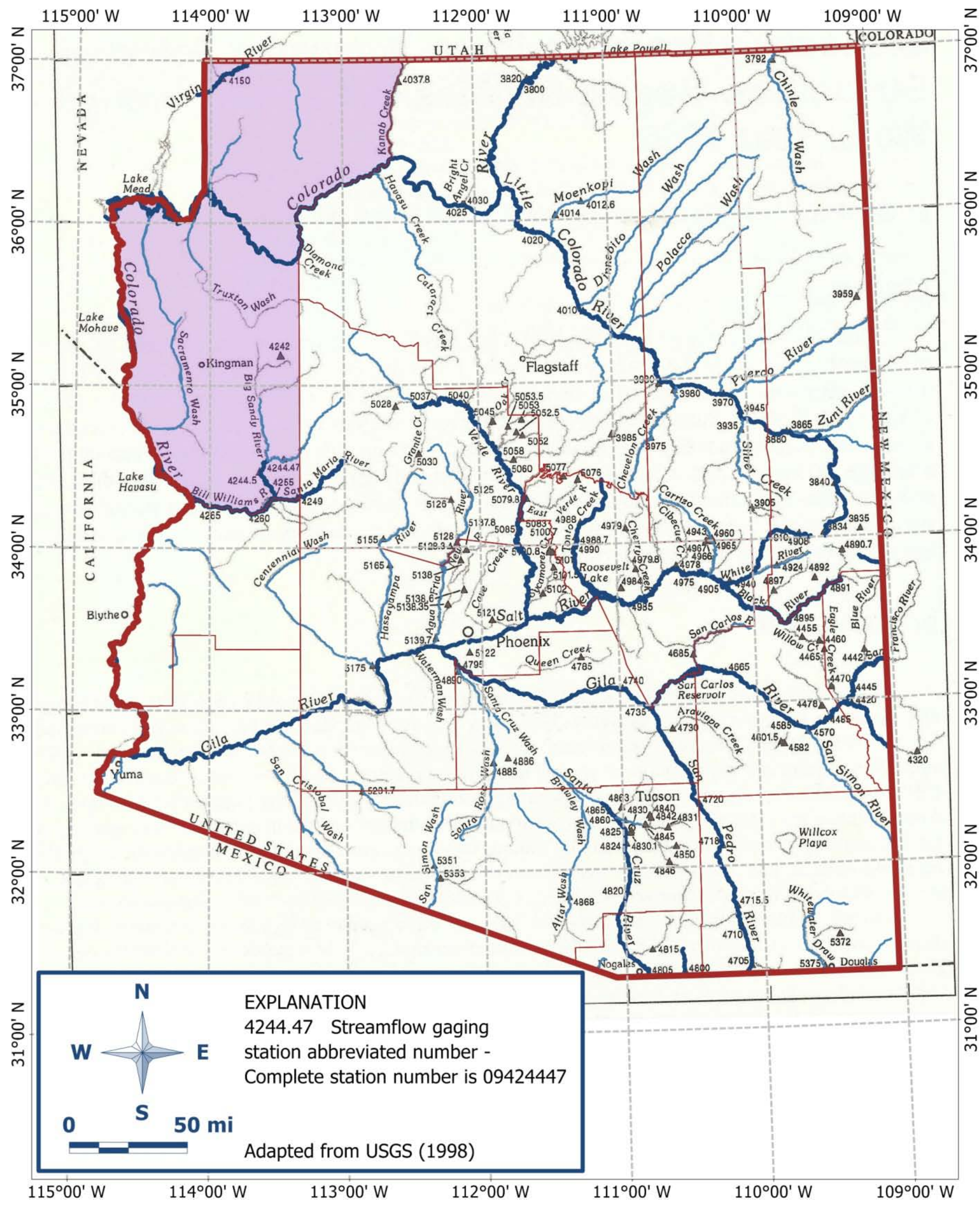
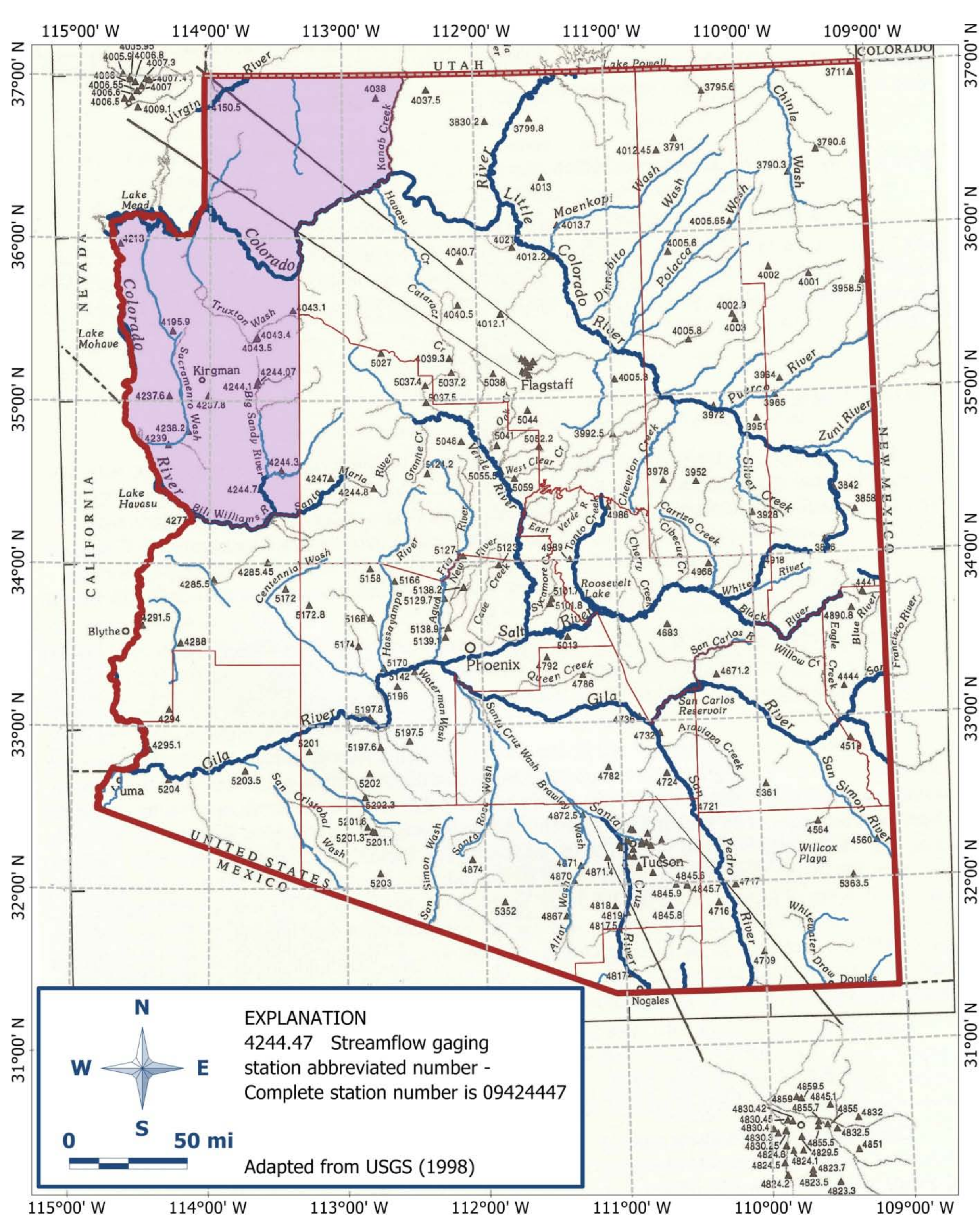


Figure 7.25 Location of USGS partial-record gaging stations in Arizona

Unregulated and partly regulated peak-flow partial record gaging stations through water year 1996



7.11.2.4 Indirect Method No.3 – Regional Regression Equations

An analysis was performed of stream flow data for a study area comprised of all of Arizona and Utah, and parts of California, Colorado, Idaho, Nevada, New Mexico, Oregon, Texas, and Wyoming (Thomas et al, 1997, USGS Water-Supply Paper 2433), as shown on Figure 7.26. The analysis resulted in sixteen sets of regional regression equations for the study area, as shown on Figure 7.27, that are applicable for watersheds less than 200 square miles in area. Five of those regions (6, 8, 10, 11 and 12) affect watersheds that are in, or adjacent to, Mohave County as shown on Figure 7.28. A map showing the location of USGS streamflow gages in the vicinity of Mohave County and used in the regression analyses is shown in Figure 7.29. These regional regression equations can be used to estimate flood magnitude-frequencies for watersheds affecting Mohave County. Regression equations are provided for all five regions to estimate flood peak discharges for frequencies of 2-, 5-, 10-, 25-, 50-, and 100-years. Use of the regression equations is recommended only if the values of the independent variables for the watershed of interest are within the range of the database used to derive the specific regression equation. The regression equations for all five regions are functions of drainage area. The regression equations for Flood Regions 6, 8 and 12 are also a function of mean basin elevation, and those for Flood Region 11 are also a function of mean basin evaporation.

The regression equations are provided for Flood Regions 6, 8, 10, 11, and 12 in Table 7.13, Table 7.14, Table 7.15, Table 7.16, Table 7.17, respectively. Figure 7.30, Figure 7.32, and Figure 7.37 are scatter diagrams of mean basin elevation versus drainage area for the database used to derive the regression equations for Flood Regions 6, 8, and 12. Figure 7.35 is a scatter diagram of mean basin evaporation versus drainage area for the database used to derive the regression equations for Flood Region 11. Also provided for each set of regression equations are graphs (Figure 7.31, Figure 7.33, Figure 7.34, Figure 7.36, and Figure 7.38) of the 100-year LP3 discharge estimates versus drainage area for Flood Regions 6, 8, 10, 11 and 12, respectively. A line depicting the relation between the 100-year peak discharge (computed from the regional regression equation) and drainage area is shown on each of those graphs. These graphs were taken from USGS Water-Supply Paper 2433 (Thomas, Hjalmarson and Waltemeyer, 1997).

Approximate mean annual evaporation in inches for Mohave County and vicinity is shown on Figure 7.39, for use with the Flood Region 11 regression equation. The evaporation contours were created using the mean annual evaporation for each gage used in the flood frequency analyses, as provided in Thomas, Hjalmarson and Waltemeyer (1997).

The scatter diagram and graph figures can be used as an additional check of the reasonableness of modeled peak discharges by plotting the model results for watersheds located within a specific flood region on the appropriate scatter diagram and graph. Reasonable model results should plot within the cloud of common values, below the envelope curve, and within the scatter of data points used for the regression equations.

Figure 7.26 Study area for USGS Water-Supply Paper 2433

(Source: Thomas et al, 1997)

Figure 7.27 Flood regions from USGS Water-Supply Paper 2433

(Source: Thomas et al, 1997)

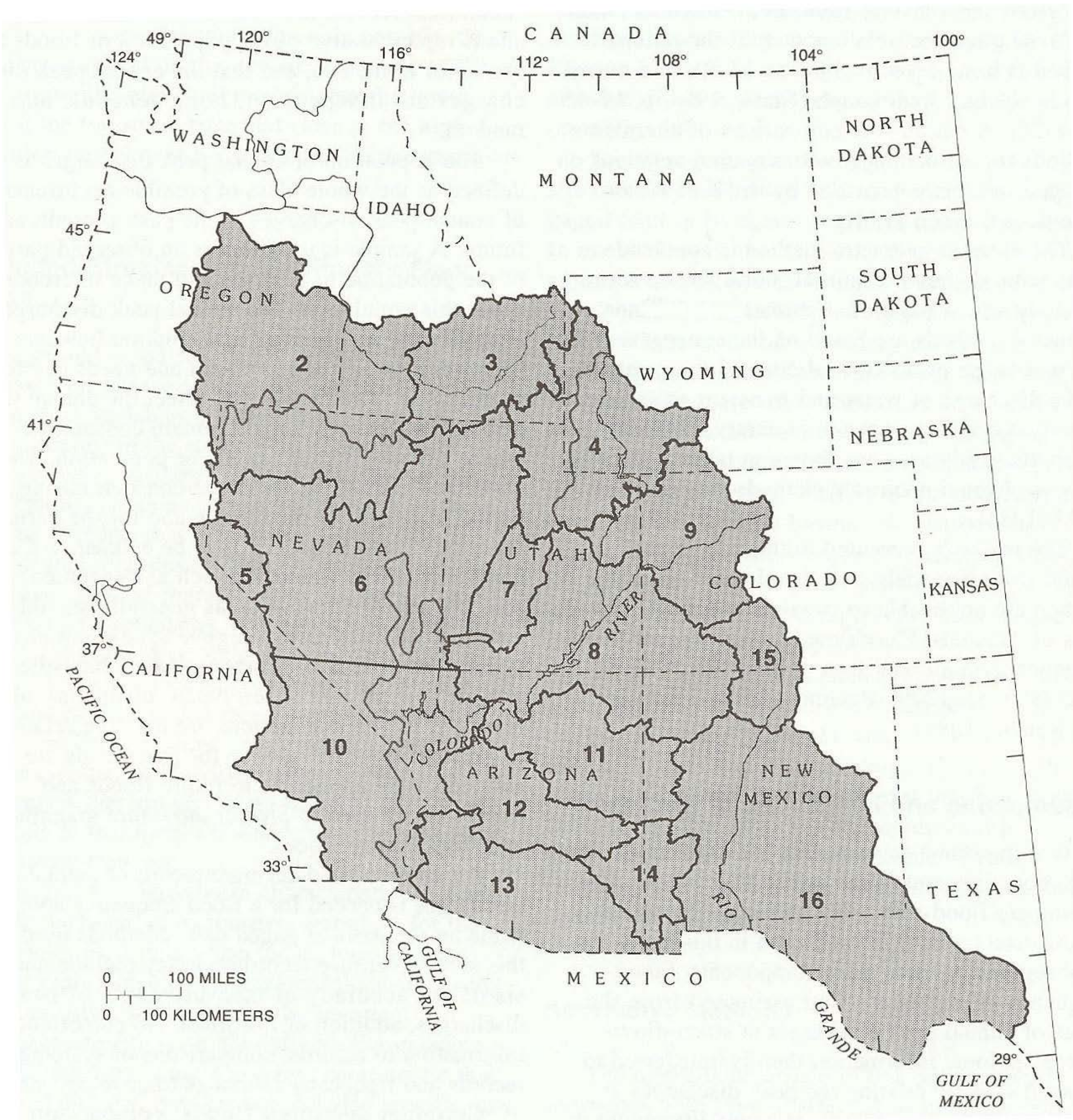


Figure 7.28 Flood regions affecting Mohave County

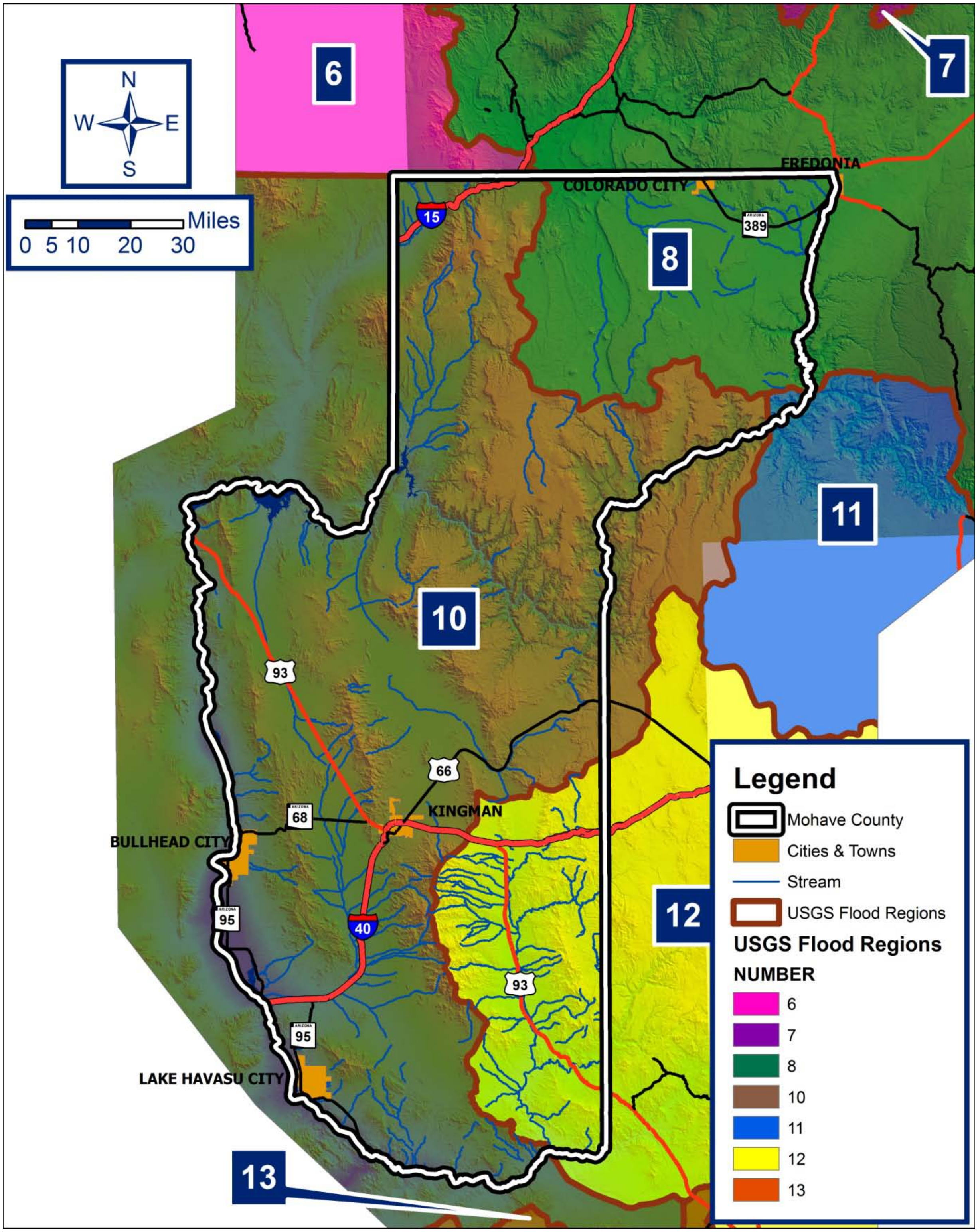


Figure 7.29 USGS stream gage locations for flood frequency equations

(Prepared using data contained in USGS(12998)

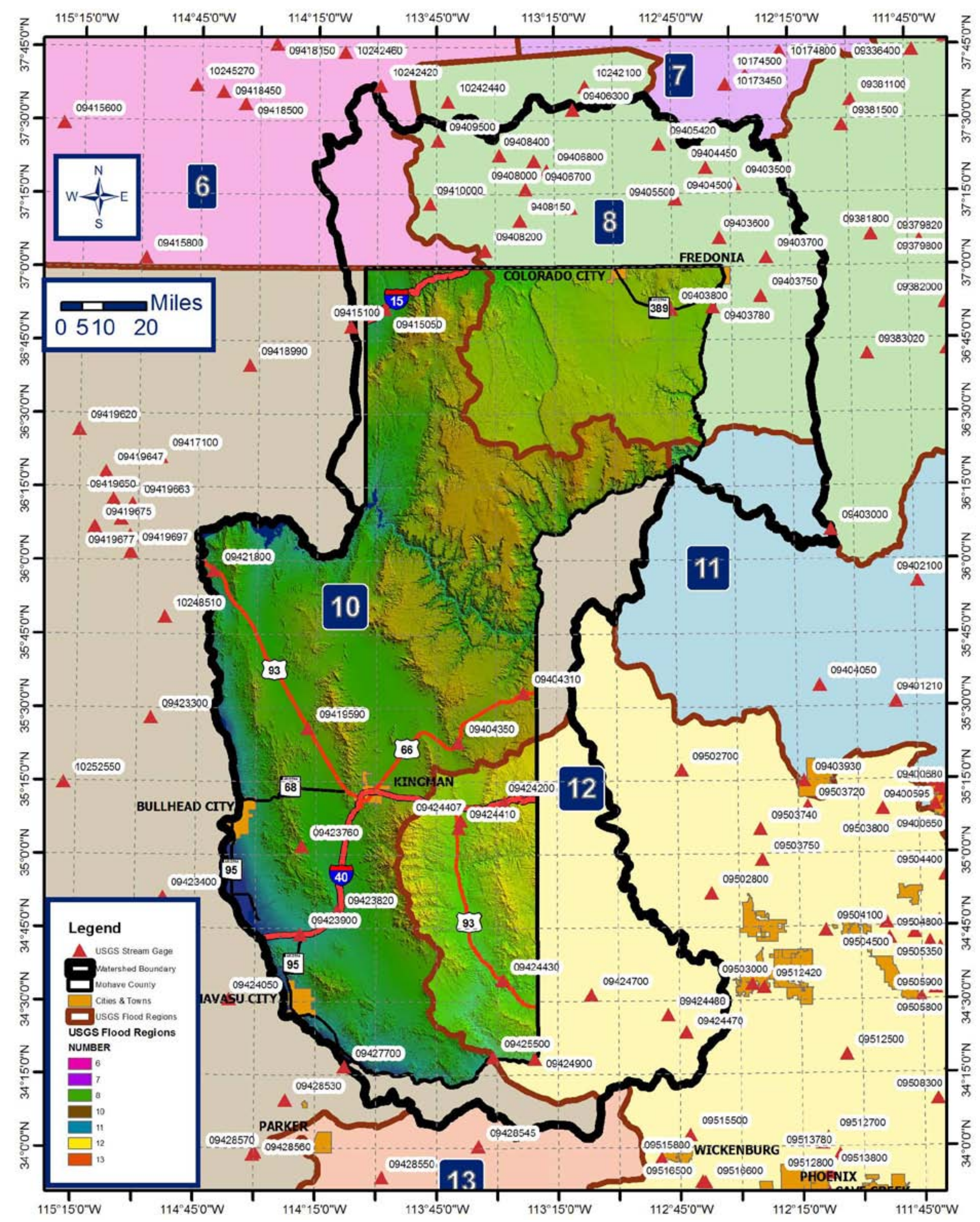


Table 7.13 Flood magnitude-frequency relations for the Northern Great Basin Region 6 (R6)

(Source: Thomas et al, 1997)

Recurrence Interval, in years	Equation	Estimated average standard error of regression, in log units	Equivalent years of record
(1)	(2)	(3)	(4)
2	$Q = 0$	---	---
5	$Q = 32AREA^{0.80}(\mathrm{ELEV}/1{,}000)^{-0.66}$	1.47	0.233
10	$Q = 590AREA^{0.62}(\mathrm{ELEV}/1{,}000)^{-1.6}$	1.12	0.748
25	$Q = 3{,}200AREA^{0.62}(\mathrm{ELEV}/1{,}000)^{-2.1}$	0.796	2.52
50	$Q = 5{,}300AREA^{0.64}(\mathrm{ELEV}/1{,}000)^{-2.1}$	1.10	1.75
100	$Q = 20{,}000AREA^{0.51}(\mathrm{ELEV}/1{,}000)^{-2.3}$	1.84	0.794

Figure 7.30 Scatter diagram of independent variables for Flood Region 6

(Source: Thomas et al, 1997)

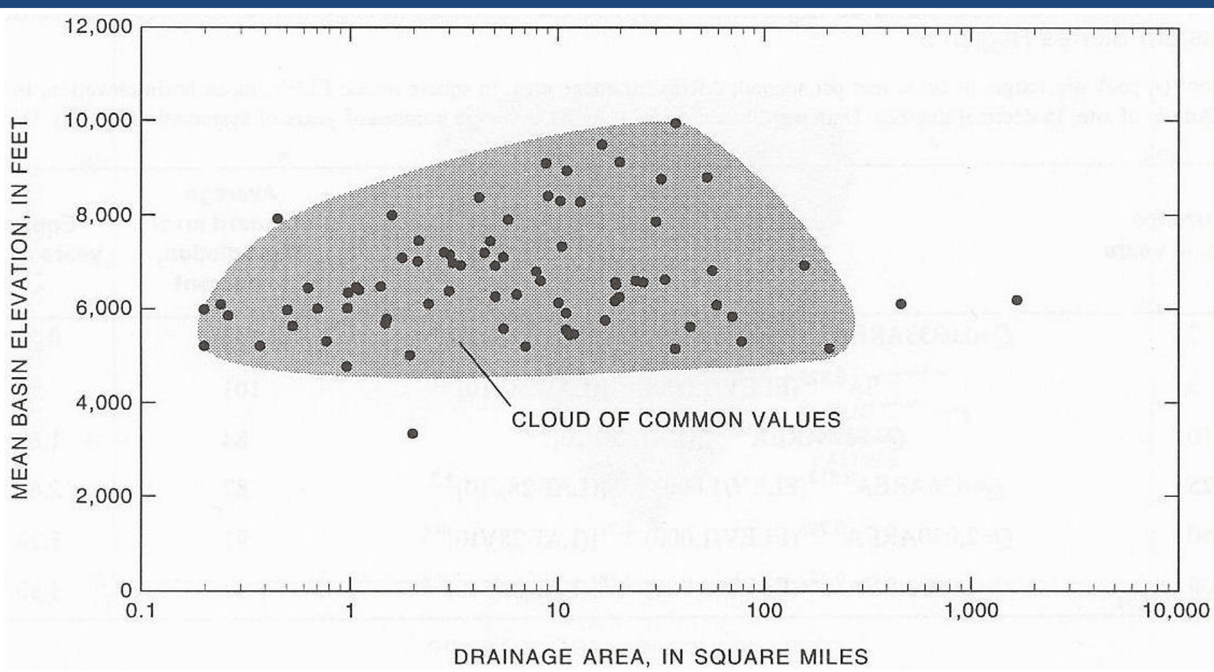


Figure 7.31 100-year peak discharge relation for Flood Region 6

(Source: Thomas et al, 1997)

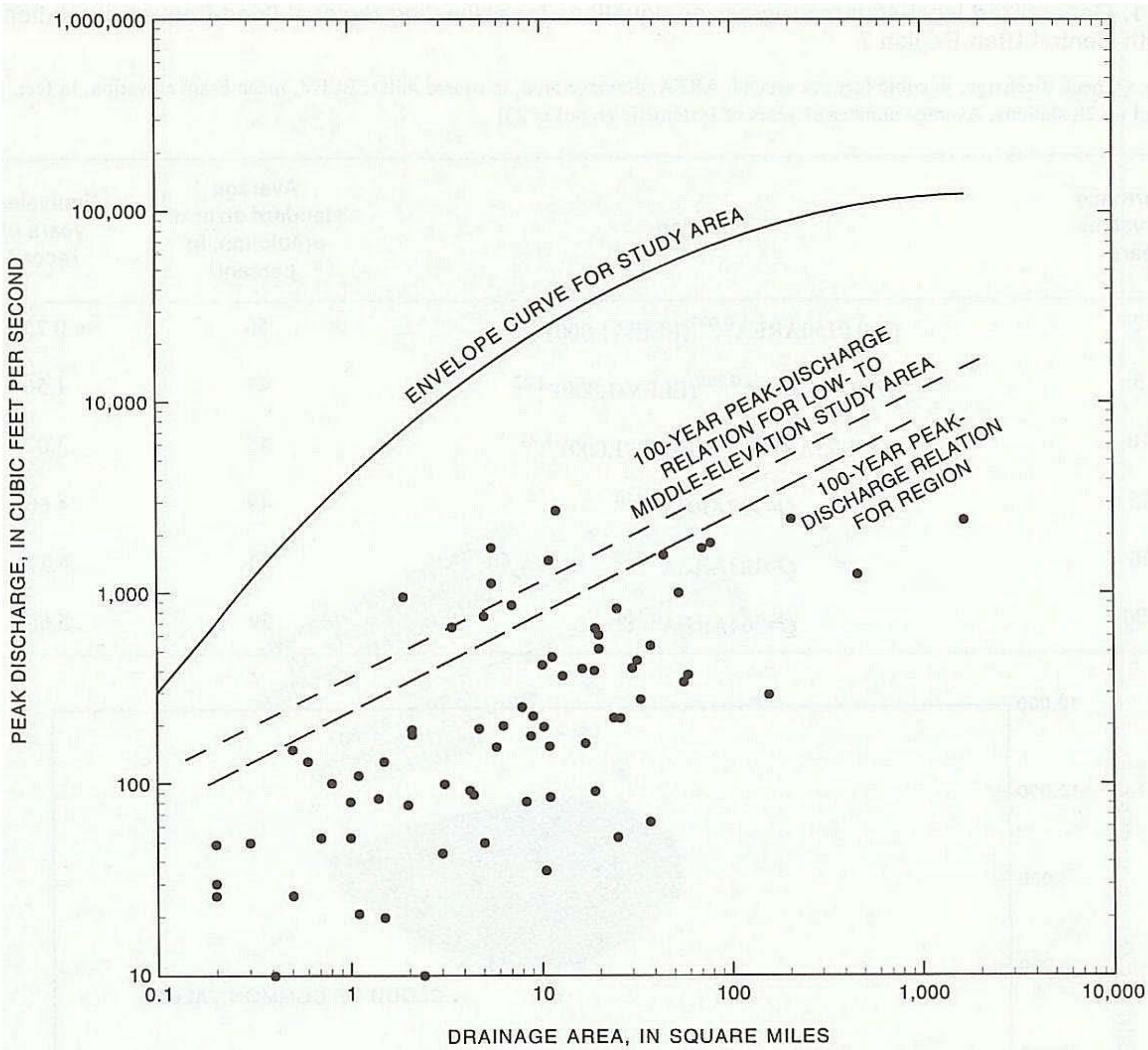


Table 7.14 Flood magnitude-frequency relations for the Southern Great Basin Region 8 (R8)

(Source: Thomas et al, 1997)

Recurrence Interval, in years	Equation	Estimated average standard error of regression, in log units	Equivalent years of record
(1)	(2)	(3)	(4)
2	$Q = 598AREA^{0.501}(\mathrm{ELEV}/1{,}000)^{-1.02}$	72	0.37
5	$Q = 2{,}620AREA^{0.449}(\mathrm{ELEV}/1{,}000)^{-1.28}$	62	1.35
10	$Q = 5{,}310AREA^{0.425}(\mathrm{ELEV}/1{,}000)^{-1.40}$	57	2.88
25	$Q = 10{,}500AREA^{0.403}(\mathrm{ELEV}/1{,}000)^{-1.49}$	54	5.45
50	$Q = 16{,}000AREA^{0.390}(\mathrm{ELEV}/1{,}000)^{-1.54}$	53	7.45
100	$Q = 23{,}300AREA^{0.377}(\mathrm{ELEV}/1{,}000)^{-1.59}$	53	9.28

Figure 7.32 Scatter diagram of independent variables for Flood Region 8

(Source: Thomas et al, 1997)

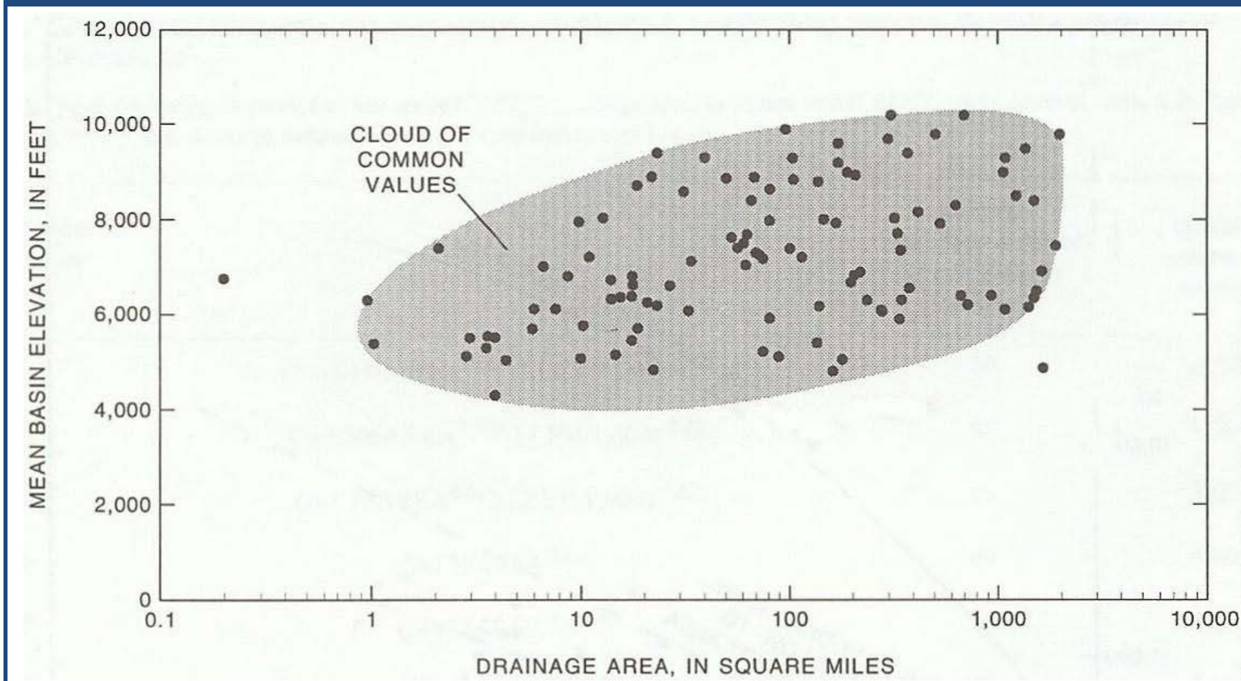


Figure 7.33 100-year peak discharge relation for Flood Region 8

(Source: Thomas et al, 1997)

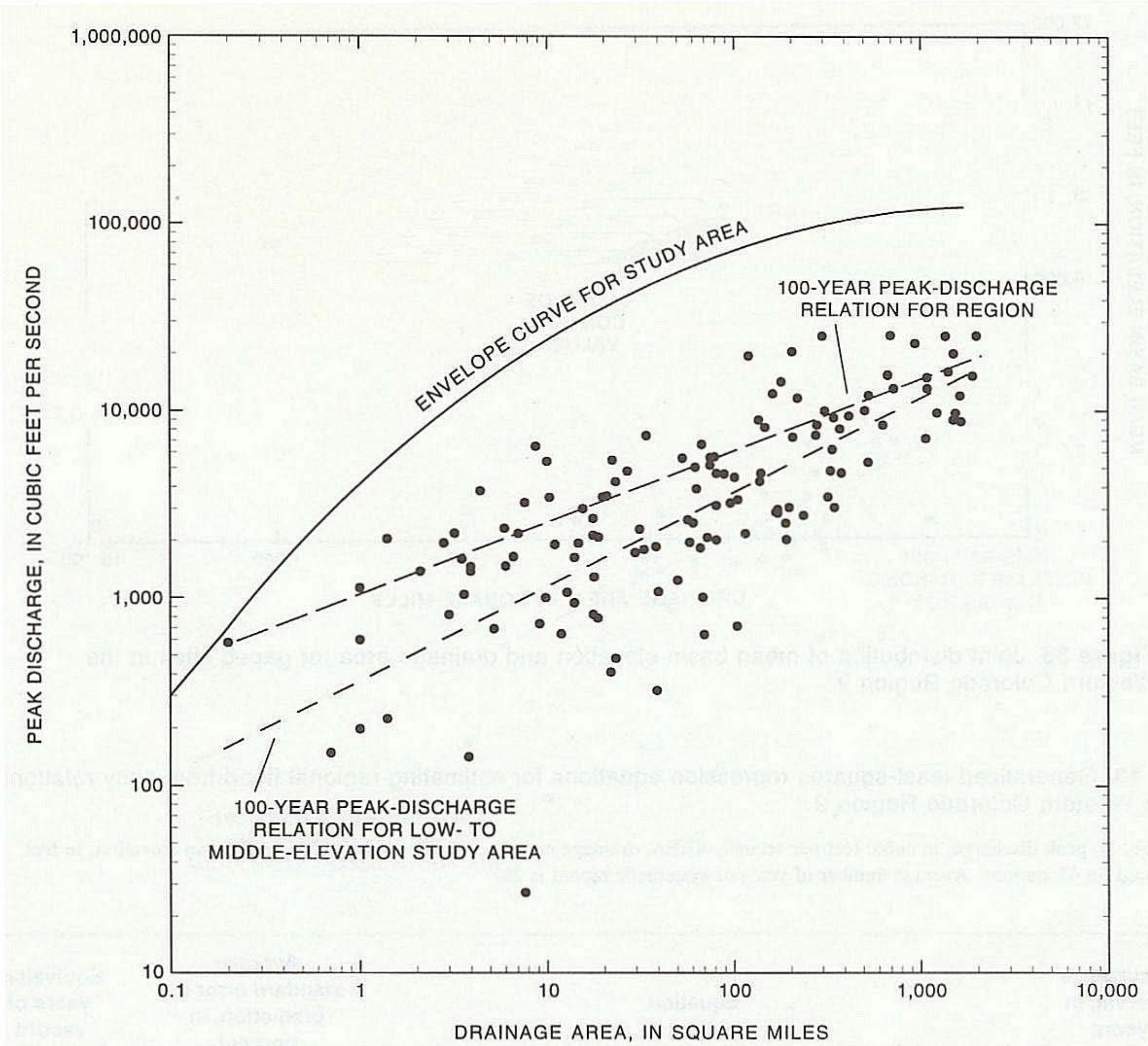


Table 7.15 Flood magnitude-frequency relations for the Four Corners Region 10 (R10)

(Source: Thomas et al, 1997)

Recurrence Interval, in years	Equation	Average standard error of prediction, in percent	Equivalent years of record
(1)	(2)	(3)	(4)
2	$Q = 12AREA^{0.58}$	1.14	0.618
5	$Q = 85AREA^{0.59}$	0.602	3.13
10	$Q = 200AREA^{0.62}$	0.675	3.45
25	$Q = 400AREA^{0.65}$	0.949	2.49
50	$Q = 590AREA^{0.65}$	0.928	3.22
100	$Q = 850AREA^{0.69}$	1.23	2.22

Figure 7.34 100-year peak discharge relation for Flood Region 10

(Source: Thomas et al, 1997)

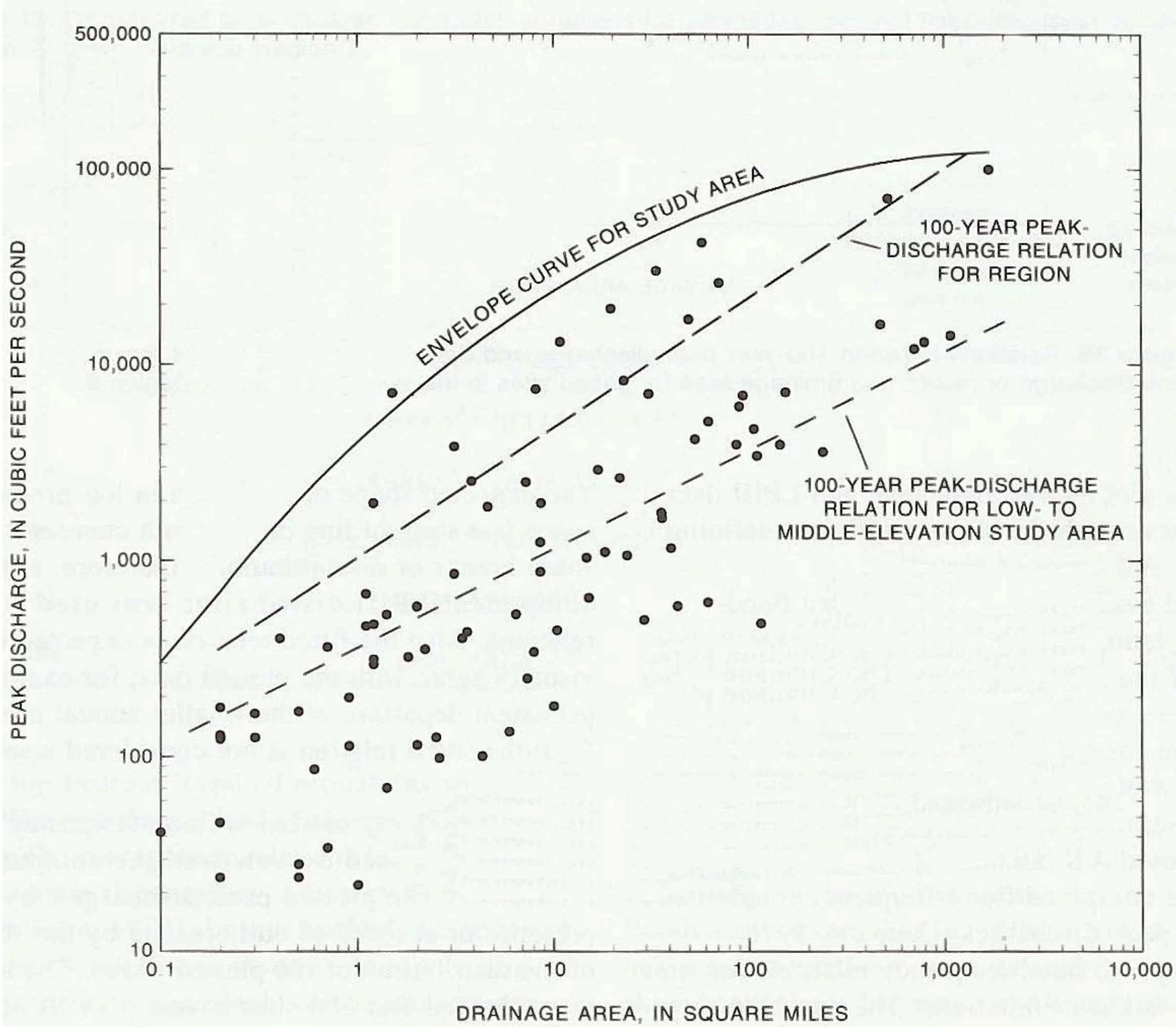


Table 7.16 Flood magnitude-frequency relations for Northeastern Arizona Region 11 (R11)

(Source: Thomas et al, 1997)

Recurrence Interval, in years	Equation	Average standard error of regression, in log units	Equivalent years of record
(1)	(2)	(3)	(4)
2	$Q = 26AREA^{0.62}$	0.609	0.428
5	$Q = 130AREA^{0.56}$	0.309	2.79
10	$Q = 0.10AREA^{0.52}\mathrm{EVAP}^{2.0}$	0.296	4.63
25	$Q = 0.17AREA^{0.52}\mathrm{EVAP}^{2.0}$	0.191	17.10
50	$Q = 0.24AREA^{0.54}\mathrm{EVAP}^{2.0}$	0.294	9.20
100	$Q = 0.27AREA^{0.58}\mathrm{EVAP}^{2.0}$	0.863	1.32

Figure 7.35 Scatter diagram of independent variables for Flood Region 11

(Source: Thomas et al, 1997)

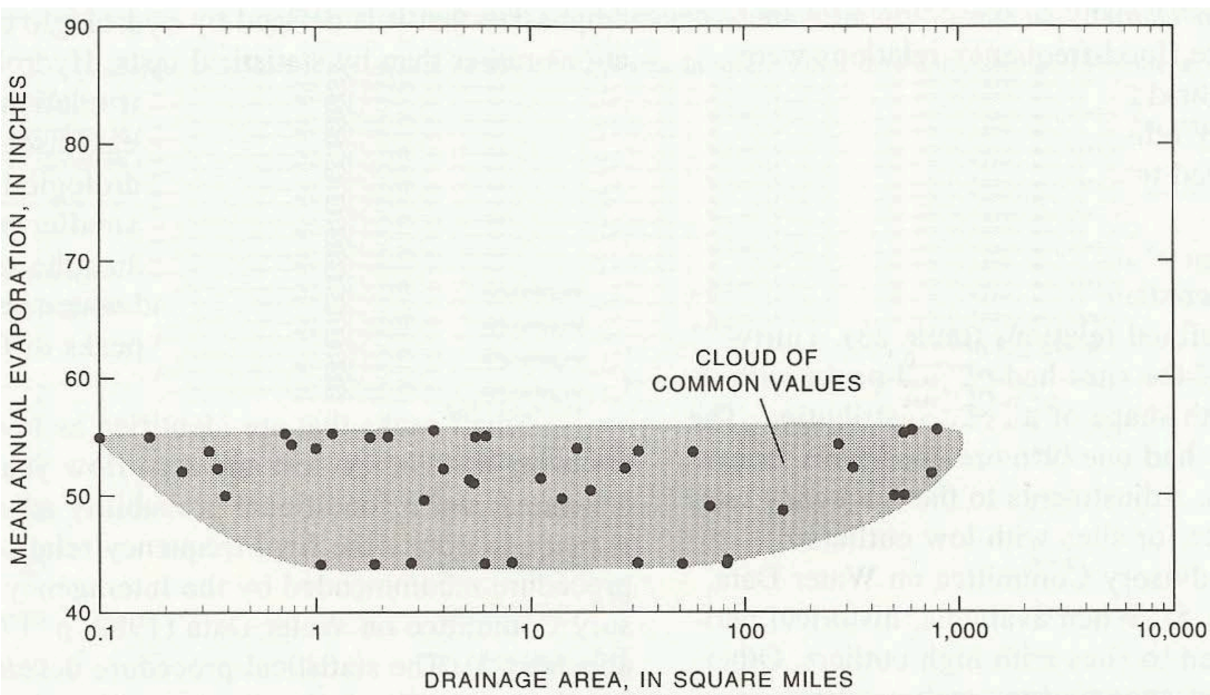


Figure 7.36 100-year peak discharge relation for Flood Region 11

(Source: Thomas et al, 1997)

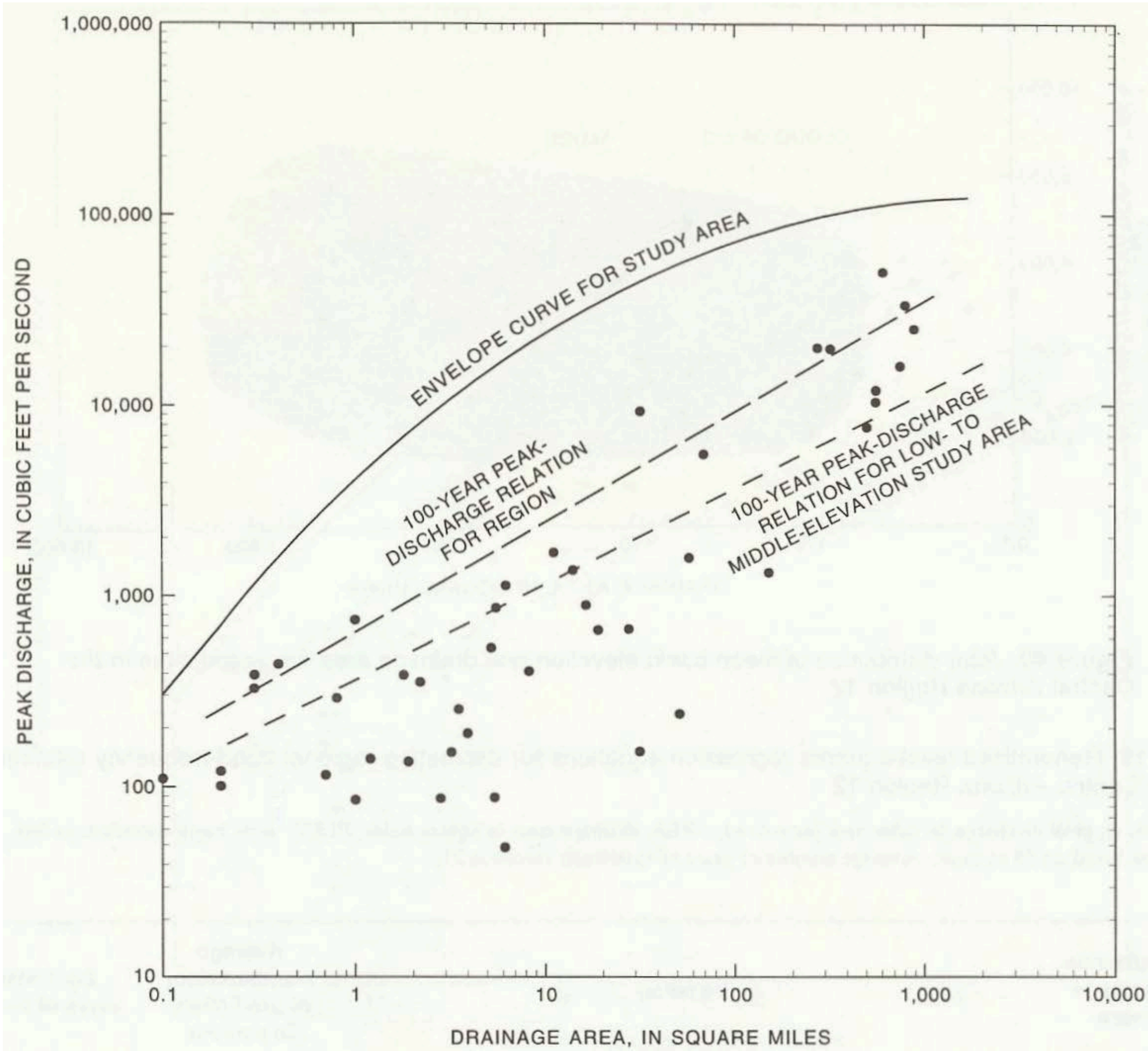


Table 7.17 Flood magnitude-frequency relations for Central Arizona Region 12 (R12)

(Source: Thomas et al, 1997)

Recurrence Interval, in years	Equation	Average standard error of prediction, in percent	Equivalent years of record
(1)	(2)	(3)	(4)
2	$Q = 41.1AREA^{0.629}$	105	0.23
5	$Q = 238AREA^{0.687}(\mathrm{ELEV}/1{,}000)^{-0.358}$	68	1.90
10	$Q = 479AREA^{0.661}(\mathrm{ELEV}/1{,}000)^{-0.398}$	52	6.24
25	$Q = 942AREA^{0.630}(\mathrm{ELEV}/1{,}000)^{-0.383}$	40	17.80
50	$Q = 10^{(7.36-4.17AREA^{-0.08})}(\mathrm{ELEV}/1{,}000)^{-0.440}$	37	27.50
100	$Q = 10^{(6.55-3.17AREA^{-0.11})}(\mathrm{ELEV}/1{,}000)^{-0.454}$	39	32.10

Figure 7.37 Scatter diagram of independent variables for Flood Region 12

(Source: Thomas et al, 1997)

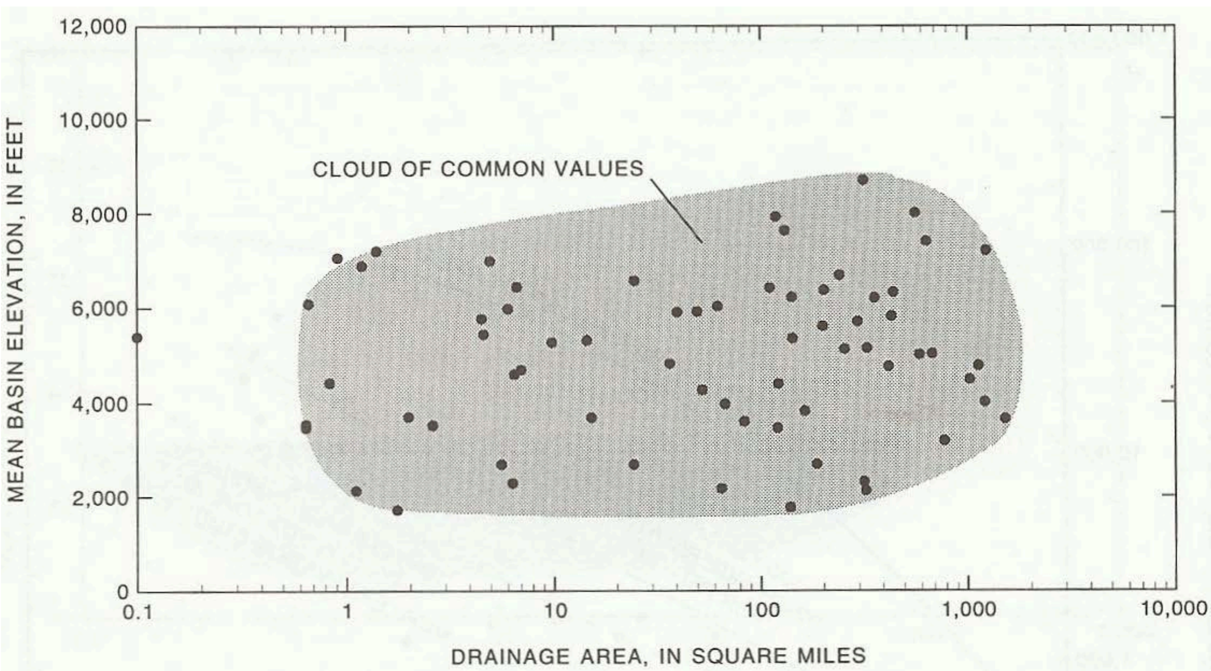


Figure 7.38 100-year peak discharge relation for Flood Region 12

(Source: Thomas et al, 1997)

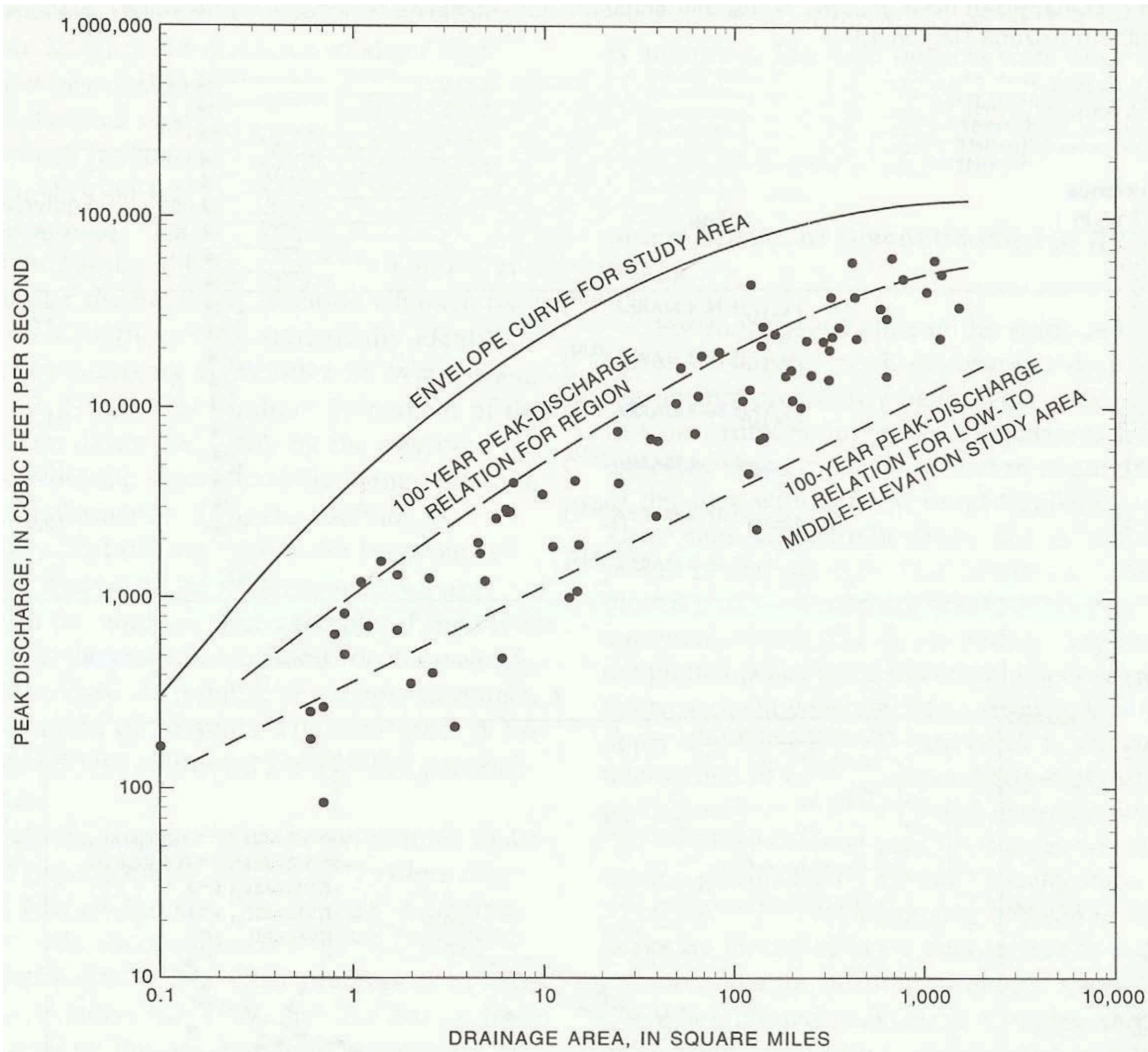
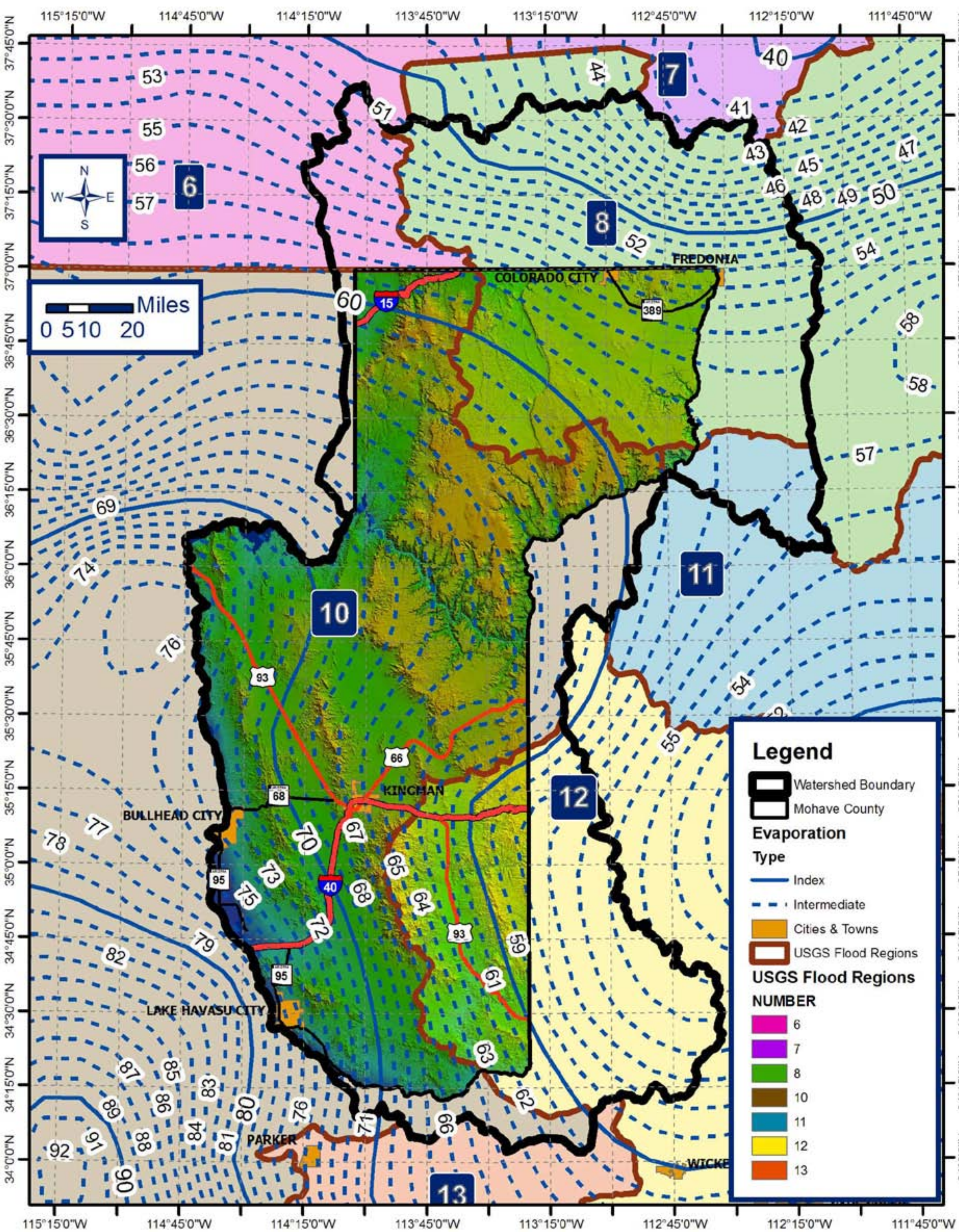


Figure 7.39 Mean annual evaporation for Mohave County



7.11.3 APPLICATIONS AND LIMITATIONS

The three indirect methods can be applied to any watershed in Mohave County gaged or ungaged. Limitations exist for the use of the Regional Regression Equations based on values of the watershed characteristics as compared to the values of watershed characteristics that were used to derive these regional regression equations. The interpretation and evaluation of the results of these methods must be conducted with awareness of several factors.

1. It must be noted that these are empirical methods and the results are only applicable to watersheds that are hydrologically similar to the data base used to derive the particular method.
2. The majority of the data in all three of these methods are for undeveloped, unregulated watersheds. Urbanized watersheds can have significantly higher discharges than the results that are predicted by any of these methods.
3. These methods (other than envelope curves) produce discharge values that are statistically based averages for watersheds in the database. Conditions can exist in any watershed that would produce flood discharges, either larger than or smaller than, those indicated by these methods. Watershed characteristics that should be considered when comparing the results of indirect methods to results by analytic methods and/or flood frequency analysis are:
   a. the occurrence and extent of rock outcrop in the watershed,
   b. watershed slopes that are either exceptionally flat or steep,
   c. soil and vegetation conditions that are conducive to low rainfall losses, such as clay soils, thin soil horizons underlain by rock or clay layers, denuded watersheds (forest and range fires), and disturbed land,
   d. soil and vegetation conditions that are conducive to high rainfall losses, such as sandy soil, tilled agricultural land, and irrigated turf,
   e. land-use, especially urbanization, but also mining, large scale construction activity, and over-grazing,
   f. transmission losses that may occur in the watercourses,
   g. the existence of distributary flow areas, and
   h. upstream water regulation or diversion.

7.11.4 INSTRUCTIONS FOR INDIRECT METHODS

The following instructions should be followed for verifying peak discharges that are derived by analytic methods, (Rational Method or rainfall-runoff modeling).

1. Verification with Unit Peak Discharge Curves:
   a. For a given watershed of drainage area (A), in square miles, divide the 100-year primary peak discharge estimate by (A).

b. Plot the unit peak discharge on a copy of Figure 7.20. Note the location of the plotted point in relation to the various curves in that figure.

2. Verification with USGS Data for Arizona:

   a. Calculate the 100-year peak discharge estimate by Equation 7.25.

   b. Select Figure 7.22 or Figure 7.23 according to watershed drainage area size, and plot the 100-year peak discharge estimate on a copy of that figure.

   c. Using watershed drainage area as a guide, identify gaged watersheds of the same approximate size from Appendix F. Tabulate the peak discharge statistics and watershed characteristics for those gaged watersheds by using the USGS report (Pope, Rigas and Smith, 1998). Compare these to the computed peak discharge estimates and watershed characteristics for the watershed of interest.

3. Verification with Regional Regression Equations:

   a. Calculate the mean basin elevation (*ELEV*) or mean basin evaporation (*EVAP*). *ELEV* can be estimated by placing a transparent grid over the largest scale topographic map available. The grid spacing should be selected such that at least 20 elevation points are sampled. The elevation at each grid point is determined and the elevations are then averaged. *EVAP* can be estimated using Figure 7.39.

   b. Determine the flood region (Figure 7.28).

   c. Check the drainage area using the appropriate scatter diagram to determine if the areas of the subject sub-basins are within the "cloud of common values." Proceed with the analysis regardless of the outcome, but clearly note if the variable values are not within the "cloud of common values."

   d. Calculate the peak discharge estimates using the applicable regression equations for the flood region within which the project site is located.

   e. Plot the 100-year peak discharge estimate on a copy of the appropriate Q_{100} data points and 100-year peak discharge relation graph.

1. For all three Indirect Methods:

   a. Quantitatively and qualitatively analyze the results of the three indirect method checks. Address watershed characteristics that may explain modeled peak discharge values that do not compare well with the indirect method checks.

   b. Prepare a summary of results by all methods and a qualitative evaluation of the results.

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7.12 HYDROLOGY POLICIES

7.12.1 SOURCE OF PEAK DISCHARGE AND RUNOFF VOLUME INFORMATION

7.12.1.1 Source of Peak Discharge and Runoff Volume Information

The following is the preferred order of hierarchy for obtaining peak discharges and runoff volumes for various floodplain and drainage design purposes:

1. The first choice is to obtain accepted peak discharges and runoff volumes of record from Mohave County drainage master plans or flood insurance studies. The results from these studies must be evaluated to determine if the assumptions made are still valid and appropriate for the intended purpose. Such studies may only provide information for the 100-year storm. Information for other storm frequencies may be obtained by appropriate revision of the existing computer models using the procedures defined in the Drainage Design Manual for Mohave County.
2. The second choice is the drainage plans and design reports from adjacent properties. This information may be used where available and if approved by the reviewing agency for use on the project.
3. If choices 1 and 2 above are not available options, or are deemed inappropriate, then peak discharges and runoff volumes should be estimated in accordance with the procedures in Section 7.

7.12.1.2 Order of Preference for Hydrologic Procedures

The following is the preferred hierarchy for applying the three hydrologic methods presented herein:

1. Watershed area less than or equal to 160 acres: The Rational Method should normally be used.
2. Regional Regression equations from Section 7.11.2.4 should be used, within the limitations of the data (watershed area and elevation) and assumptions used to create them, and if hydrographs are not needed. If hydrographs are needed or there are multiple concentration points requiring routing and combining of hydrographs, then the Unit Hydrograph Method should be used.
3. Watershed area greater than 160 acres and less than a total of 500 square miles: Unit Hydrograph Method.

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7.13 HYDROLOGY STANDARDS

The following sections contain specific Mohave County technical standards for application of the hydrologic methodology set forth herein.

7.13.1 DESIGN STORM DURATION CRITERIA

<table>
<tr><th colspan="2">Table 7.18 Design storm duration criteria for unincorporated Mohave County</th></tr>
<tr><th>Purpose / Method</th><th>Criteria</th></tr>
<tr><td>Retention Basins</td><td>100-year, 2-hour rainfall</td></tr>
<tr><td colspan="2">Analysis for undisturbed drainageways and design of engineered channels, bridges, and culverts:</td></tr>
<tr><td>Drainage Area: 0 to 160 acres
(Rational Method or Unit Hydrograph Method)</td><td>If only design peak discharges are needed, then the Rational Method is acceptable.</td></tr>
<tr><td>Drainage area: 160 acres to 20 square miles
(Unit Hydrograph Method)</td><td>6-hour local storm.
Engineering judgment may dictate use of a 24-hour storm depending on soil conditions, or other hydrologic parameters or criteria. Mohave County may require analysis of both the 6-hour and 24-hour storms, and require that the larger peak discharge be utilized.</td></tr>
<tr><td>Drainage area: 20 to 100 square miles
(Unit Hydrograph Method)</td><td>Either a critically centered 6-hour local storm or a 24-hour general storm.
Mohave County requires analysis of both the 6-hour local storm and the 24-hour general storm, and requires that the larger peak discharge and runoff volume be utilized.</td></tr>
<tr><td>Drainage area.100 to 500 square miles
(Unit Hydrograph Method)</td><td>24-hour general storm.</td></tr>
</table>

7.13.2 RATIONAL METHOD CRITERIA

Table 7.19 Rational Method parameters for unincorporated Mohave County

DDMSW ID	Mohave County Zoning Classification	K_b	Storm Frequency, in years					
			2	5	10	25	50	100
Agriculture								
390	Agriculture (AG)	0.08	0.12	0.15	0.18	0.20	0.22	0.25
Commercial								
120	A-D: Airport Development (1 acre minimum, C2)	0.04	0.75	0.80	0.85	0.88	0.92	0.95
220	C-1: Neighborhood Commercial (6,000 sf minimum, C1)	0.025	0.75	0.80	0.83	0.90	0.93	0.95
230	C-2: General Commercial (6,000 sf minimum, C2)	0.025	0.75	0.80	0.83	0.90	0.93	0.95
240	C-2H: General Commercial Highway Frontage (1-acre minimum, C2)	0.025	0.75	0.80	0.83	0.90	0.93	0.95
250	C-RE: Commercial Recreation (1 acre minimum, C1)	0.025	0.60	0.63	0.65	0.70	0.75	0.80
260	C-M: Commercial Manufacturing (1 acre minimum, C2)	0.025	0.75	0.80	0.83	0.90	0.93	0.95
270	C-MO: Commercial Manufacturing/ Open Lot Storage (1 acre minimum, C2)	0.025	0.75	0.80	0.83	0.90	0.93	0.95
Industrial								
280	M: General Manufacturing (1 acre minimum, I1)	0.025	0.70	0.70	0.70	0.75	0.80	0.85
290	M-X: Heavy Manufacturing (1 acre minimum, I2)	0.025	0.80	0.80	0.80	0.85	0.92	0.95
Natural								
500	Undeveloped Desert Rangeland. Little topographic relief, slopes <5%	0.08	0.33	0.35	0.38	0.40	0.42	0.45
510	Hillslopes, Sonoran Desert. Moderate topographic relief, slopes >5%	0.15	0.45	0.50	0.53	0.55	0.60	0.65
520	Mountain Terrain. High topographic relief, slopes >20%	0.20	0.55	0.60	0.65	0.70	0.75	0.80

Table 7.19 Rational Method parameters for unincorporated Mohave County

DDMSW ID	Mohave County Zoning Classification	K_b	Storm Frequency, in years					
			2	5	10	25	50	100
Open Space								
300	R-P: Regional Parks (LP3)	0.20	0.15	0.20	0.25	0.28	0.30	0.35
310	C-P: Community Parks (LP2)	0.20	0.25	0.30	0.35	0.40	0.45	0.50
320	N-P: Neighborhood Parks (LP1)	0.20	0.35	0.40	0.45	0.48	0.50	0.55
350	General Open Space (Open space where no detail available, NDR)	0.08	0.33	0.35	0.38	0.40	0.42	0.45
360	Golf Courses (LP3)	0.20	0.15	0.20	0.25	0.28	0.30	0.35
370	Passive Open Space (NDR)	0.08	0.33	0.35	0.38	0.40	0.42	0.45
400	Landscaping with impervious under treatment	0.08	0.60	0.70	0.80	0.85	0.90	0.92
410	Landscaping w/o impervious under treatment	0.08	0.33	0.35	0.38	0.40	0.42	0.45
Other								
380	Water	0.02	0.90	0.90	0.90	0.95	0.95	0.95
Residential								
110	A: General (> 20 acre minimum, VLDR)	0.05	0.35	0.37	0.39	0.44	0.46	0.50
112	A: General (5-20 acre minimum, VLDR)	0.05	0.36	0.38	0.40	0.46	0.50	0.55
114	A-R, R-E, R-1, R-MH, R-TT, R-M, R-O, R-O/A (2-5 acre minimum, VLDR)	0.04	0.37	0.40	0.42	0.48	0.55	0.60
130	A-R, R-E, R-1, R-MH, R-TT, R-M, R-O, R-O/A (1-2 acre minimum, VLDR)	0.04	0.40	0.42	0.45	0.50	0.60	0.65
140	R-E, R-1, R-MH, R-TT, R-M, R-O (20,000 sf – 1 acre minimum, LDR)	0.04	0.42	0.45	0.48	0.51	0.56	0.58
150	R-1, R-MH, R-TT, R-M, R-O (10,000-20,000 sf minimum, MDR)	0.04	0.50	0.55	0.58	0.55	0.60	0.65
160	R-1, R-MH, R-TT, R-M, R-O (7,000-10,000 sf minimum, MDR)	0.025	0.52	0.55	0.60	0.65	0.70	0.75
170	R-1, R-MH, R-TT, R-M, R-O (6,000-7,000 sf minimum, MDR)	0.025	0.55	0.60	0.63	0.70	0.75	0.78

Table 7.19 Rational Method parameters for unincorporated Mohave County								
DDMSW ID	Mohave County Zoning Classification	K_b	Storm Frequency, in years					
			2	5	10	25	50	100
180	R-1, R-MH, R-TT, R-M, R-O with lot coverage increase waiver (6,000 sf minimum, MFR)	0.025	0.70	0.73	0.75	0.80	0.87	0.92
210	S-D: Special Development (MFR)	0.025	0.70	0.73	0.75	0.80	0.87	0.92
Transportation								
330	General Transportation (Transportation where no detail available; P, GR)	0.02	0.70	0.75	0.80	0.85	0.90	0.92
340	Transportation (Includes railroads, rail-yards, transit centers, freeways; P, GR)	0.02	0.65	0.70	0.75	0.80	0.85	0.90
420	Pavement and Rooftops (P)	0.02	0.85	0.85	0.85	0.90	0.93	0.95
430	Gravel Roadways & Shoulders (GR)	0.03	0.70	0.70	0.70	0.75	0.80	0.85

7.13.3 UNIT HYDROGRAPH METHOD CRITERIA

Table 7.20 Unit hydrograph parameters for unincorporated Mohave County						
DDMSW ID	Mohave County Zoning Classification	Vegetation Cover	*RTIMP*	IA	*DTHETA* Cond.	*XKSAT*[3]
Agriculture						
390	Agriculture (AG)	85	0	0.50	NORMAL	N
Commercial						
120	A-D: Airport Development (1 acre minimum, C2)	20	90	0.25	NORMAL	D
220	C-1: Neighborhood Commercial (6,000 sf minimum, C1)	65	80	0.10	NORMAL	D
230	C-2: General Commercial (6,000 sf minimum, C2)	75	80	0.10	NORMAL	D
240	C-2H: General Commercial Highway Frontage (1-acre minimum, C2)	75	80	0.10	NORMAL	D

[3] N: Use $XKSAT_N$; D: Use $XKSAT_D$

Table 7.20 Unit hydrograph parameters for unincorporated Mohave County

DDMSW ID	Mohave County Zoning Classification	Vegetation Cover	*RTIMP*	IA	*DTHETA* Cond.	*XKSAT*[3]
250	C-RE: Commercial Recreation (1 acre minimum, C1)	60	80	0.10	NORMAL	D
260	C-M: Commercial Manufacturing (1 acre minimum, C2)	60	80	0.10	NORMAL	D
270	C-MO: Commercial Manufacturing/ Open Lot Storage (1 acre minimum, C2)	60	80	0.10	NORMAL	D
	Industrial					
280	M: General Manufacturing (1 acre minimum, I1)	60	55	0.15	NORMAL	D
290	M-X: Heavy Manufacturing (1 acre minimum, I2)	60	60	0.15	NORMAL	D
	Natural					
500	Undeveloped Desert Rangeland. Little topographic relief, slopes <5%	varies	varies	0.35	DRY	N
510	Hillslopes, Sonoran Desert. Moderate topographic relief, slopes >5%	varies	varies	0.15	DRY	N
520	Mountain Terrain. High topographic relief, slopes >20%	varies	varies	0.25	DRY	N
	Open Space					
300	R-P: Regional Parks (LP3)	60	10	0.20	NORMAL	varies
310	C-P: Community Parks (LP2)	80	15	0.20	NORMAL	D
320	N-P: Neighborhood Parks (LP1)	80	15	0.20	NORMAL	D
350	General Open Space (Open space where no detail available, NDR)	10	10	0.10	DRY	N
360	Golf Courses (LP3)	90	5	0.20	NORMAL	D
370	Passive Open Space (NDR)	20	10	0.10	DRY	
400	Landscaping with impervious under treatment	30	95	0.10	NORMAL	D
410	Landscaping w/o impervious under treatment	30	0	0.20	NORMAL	D
	Other					

Table 7.20 Unit hydrograph parameters for unincorporated Mohave County						
DDMSW ID	**Mohave County Zoning Classification**	**Vegetation Cover**	***RTIMP***	**IA**	***DTHETA* Cond.**	***XKSAT*[3]**
380	Water	0	100	0.00	NORMAL	n/a
Residential						
110	A: General (> 20 acre minimum, VLDR)	30	5	0.30	NORMAL	N
112	A: General (5-20 acre minimum, VLDR)	30	10	0.30	NORMAL	N
114	A-R, R-E, R-1, R-MH, R-TT, R-M, R-O, R-O/A (2-5 acre minimum, VLDR)	50	15	0.30	NORMAL	N
130	A-R, R-E, R-1, R-MH, R-TT, R-M, R-O, R-O/A (1-2 acre minimum, VLDR)	50	20	0.25	NORMAL	D
140	R-E, R-1, R-MH, R-TT, R-M, R-O (20,000 sf – 1 acre minimum, LDR)	50	25	0.25	NORMAL	D
150	R-1, R-MH, R-TT, R-M, R-O (10,000-20,000 sf minimum, MDR)	50	35	0.25	NORMAL	D
160	R-1, R-MH, R-TT, R-M, R-O (7,000-10,000 sf minimum, MDR)	50	50	0.25	NORMAL	D
170	R-1, R-MH, R-TT, R-M, R-O (6,000-7,000 sf minimum, MDR)	50	60	0.25	NORMAL	D
180	R-1, R-MH, R-TT, R-M, R-O with lot coverage increase waiver (6,000 sf minimum, MFR)	50	70	0.25	NORMAL	D
210	S-D: Special Development (MFR)	60	75	0.25	NORMAL	D
Transportation						
330	General Transportation (Transportation where no detail available; P, GR)	0	95	0.05	NORMAL	D
340	Transportation (Includes railroads, rail-yards, transit centers, freeways; P, GR)	0	95	0.05	NORMAL	D
420	Pavement and Rooftops (P)	0	95	0.05	NORMAL	D

Table 7.20 Unit hydrograph parameters for unincorporated Mohave County

DDMSW ID	Mohave County Zoning Classification	Vegetation Cover	*RTIMP*	IA	*DTHETA* Cond.	*XKSAT*[3]
430	Gravel Roadways & Shoulders (GR)	0	5	0.05	NORMAL	D

7.13.4 SOILS XKSAT VALUES FOR UNIT HYDROGRAPH METHOD

The standard procedure is to use the Green and Ampt and Initial and Uniform Loss method parameters supplied with the DDMSW computer program and the standard soils GIS shape files proved on the Mohave County Flood Control District web site. However, the engineer/hydrologist is strongly encouraged to field verify the soils physical characteristics present in the watershed being modeled. If the engineer/hydrologist determines that the soil physical properties differ significantly from those provided by the NRCS, a site-specific geotechnical investigation is encouraged. Such an investigation should include determination of the sand, clay, gravel and clay content of the soils in the top 6-inch soil horizon. Sufficient tests should be taken to provide a representative set of average values for a specific spatial extent. Aerial photographs, field reconnaissance, geomorphologic techniques, and engineering judgment should be used to determine the spatial extent of individual soil types. The Saxton and Rawls (2005) *Soil Water Characteristics* computer program may then be used to compute an estimate of K_s for each soil type identified. K_s should then be corrected using Equation 7.8 to obtain XKSAT. This same process should be followed if there are Miscellaneous Component soils present in the watershed. Refer to Section 7.4.2.3.

8 FLOODPLAIN MANAGEMENT

8.1 INTRODUCTION

Mohave County participates in the National Flood Insurance Program (NFIP) which provides flood insurance to its citizens, flood mitigation assistance and emergency assistance to flood victims. FEMA oversees the NFIP. FEMA has regulations pertaining to floodplain management that must be followed in order for Mohave County to continue as a member of the NFIP.

Mohave County has local policies to manage floodplains in a uniform and consistent manner. These policies are categorized as being FEMA related and non-FEMA related in nature. The policies strictly adhere to State of Arizona and Federal regulations governing floodplains and drainage design.

8.2 FLOODPLAIN MANAGEMENT POLICIES

8.2.1 FEMA

FEMA is an independent agency of the federal government, reporting to the President. Since its founding in 1979, FEMA's mission has been clear: To reduce loss of life and property and protect our nation's critical infrastructure from all types of hazards through a comprehensive, risk-based, emergency management program of mitigation, preparedness, response and recovery.

The Mohave County policies pertaining to FEMA regulatory floodways and floodplains are as follows:

8.2.1.1 Floodways

No development shall be allowed in a FEMA Regulatory Floodway that results in any increase in flood levels during the occurrence of the base flood discharge. A "Regulatory Floodway" means the channel of a river or other watercourse and the adjacent land areas that must be reserved in order to discharge the base flood without cumulatively increasing the water surface elevation more than a designated height. In Arizona, that height is one (1) foot.

8.2.1.2 Basements

Basements shall not be allowed in Special Flood Hazard Areas for residential structures. Non-residential structures are allowed basements below the base flood elevation (BFE) as long as a registered professional engineer certifies all required "floodproofing" for the structure.

8.2.1.3 Levees, berms, or floodwalls

Levees, berms, or floodwalls while discouraged, must comply with FEMA standards and be reviewed and approved by Mohave County prior to construction. Levees or berms should not obstruct side or interior drainage to a channel.

8.2.1.4 Variances

Variances may be granted in conformance with Section 6.0 of the Mohave County Flood Control Ordinance - 2000.

8.2.2 NON-FEMA

8.2.2.1 Lowest Floor Elevation

The lowest floor elevation for dwelling units not within 100-year floodplain areas shall be controlled by the provisions of the 2003 International Building Code, as adopted by Mohave County.

8.2.2.2 Base Flood Elevation Establishment

In locations where a FEMA regulatory BFE does not exist and the 100-year discharge exceeds 500 cfs, a BFE shall be established using the standards and procedures herein and shall require DISTRICT approval.

8.2.2.3 Erosion Setback

In locations where the 100-year discharge in a wash exceeds 500 cfs and is contained within the existing channel banks, erosion setbacks consistent with ADWR State Standard 5-96 (ADWR, 1996b) shall be required for all properties developed where watercourses are to be left in an undisturbed state.

Any variances to the Mohave County floodplain policies shall require approval by the MOHAVE COUNTY ENGINEER.

8.3 FLOODPLAIN MANAGEMENT STANDARDS

8.3.1 FEMA

It is the intent of the DISTRICT that floodplains and floodways be delineated for areas meeting the criteria in the following sections, and that those delineations be submitted to FEMA for approval. The COUNTY/DISTRICT will require a developer to delineate floodplains and floodways for areas that meet these criteria. Delineations may also be done by the DISTRICT as funding permits. The Floodplain Administrator may elect to temporarily not submit a delineation to FEMA due to extenuating circumstances.

The DISTRICT will only regulate floodplains based on FEMA FIRM maps or best available information developed based on these standards. The Mohave County standards pertaining to FEMA regulatory floodways and floodplains are as follows:

8.3.1.1 Floodplain Delineation Requirement

At a minimum, delineation of the 100-year floodplain and submittal to FEMA for approval shall be required for new developments where:

1. Q_{100} >= 500 cfs, and
2. Watershed areas >= 1 sq. mi., or
3. Developments meeting criteria 1 or 2 that are 5 acres in area or greater or will have 50 or more lots.

Floodplains shall be delineated in accordance with the most current version of ADWR State Standard 9-02 (ADWR, 2002).

8.3.1.2 Floodway Delineation Requirement

Delineation of the 100-year floodway and submittal to FEMA for approval may be required when a floodplain delineation is required under Section 8.3.1.1. Floodways shall be delineated in conformance with the most current version of ADWR State Standard 9-02 (ADWR, 2002). If the DISTRICT elects to not require delineation of a floodway, the provisions of Section 8.3.1.4 shall apply. The provisions of ADWR State Standard 3-94 (ADWR, 1994) shall apply when defining floodways for supercritical or near supercritical flow conditions.

8.3.1.3 Technical Documentation Requirement for Submittal to FEMA

A Technical Data Notebook shall be prepared in conformance with ADWR State Standard 1-97 (ADWR, 1997) requirements if the DISTRICT requires that the 100-year floodplain delineation be submitted to FEMA for approval.

8.3.1.4 Existing FEMA Floodplains without a Designated Floodway

An Administrative Floodway analysis is required in conformance with ADWR State Standard 2-96 (ADWR, 1996a) where a FEMA floodplain exists without a designated floodway. At minimum, an assessment of flood depth and velocity should be performed and structures should not be placed within the area where the following criteria are exceeded unless mitigation measures are constructed to protect the dwelling from flooding and erosion hazards, and flow conveyance is maintained as approved by COUNTY/DISTRICT:

1. Houses built on foundations: Depth x Velocity > 10 and Depth > 2.5 ft.
2. Mobile homes: Depth x Velocity > 6 and Depth > 1.5 ft.

8.3.1.5 Lowest Floor Elevation Requirement

The lowest floor elevation of a dwelling shall be a minimum of 1-foot above the highest adjacent Base Flood Elevation. Note that to file a CLOMR-F with FEMA and remove the dwelling from the floodplain for flood insurance purposes, the grade adjacent to the dwelling must be at or above the Base Flood Elevation prior to construction of the structure.

8.3.2 NON-FEMA

The COUNTY shall require, as part of a subdivision and/or drainage review, that all runoff impacting a development be identified and that all structures within the proposed development be free from flooding during a 100-year storm event. The provisions of ADWR State Standard 2-96 (ADWR, 1996a) shall apply for these areas.

8.3.2.1 Floodplain Delineation Requirement

In all cases, the 100-year floodplains shall be delineated, and 100-year water surface elevations calculated, for developments meeting the following criteria:

1. Contributing watershed of 0.25 sm or greater, or
2. 100-year peak discharge of 500 cfs or greater, whichever is the most restrictive.

The procedures for Level 1, 2, or 3 from ADWR State Standard 2-96 (ADWR,1996a) shall be used, as approved by the COUNTY/DISTRICT.

8.3.2.2 Floodway Delineation Requirement

An administrative floodway is required using the procedures specified in Section 8.3.1.4.

8.3.2.3 Lowest Floor Elevation Requirement

The lowest floor elevation of a dwelling located within a floodplain delineated under Section 8.3.2.1 shall be elevated a minimum of 12-inches above the highest adjacent 100-year water surface elevation (base flood elevation).

8.3.3 GRADING AND DRAINAGE PLAN AND FINAL PLAT REQUIREMENTS

The 100-year floodplain, water surface elevations, and floodways delineated as required under both the FEMA and NON-FEMA requirements listed above shall be shown on the Grading and Drainage Plans and the Final Plat.

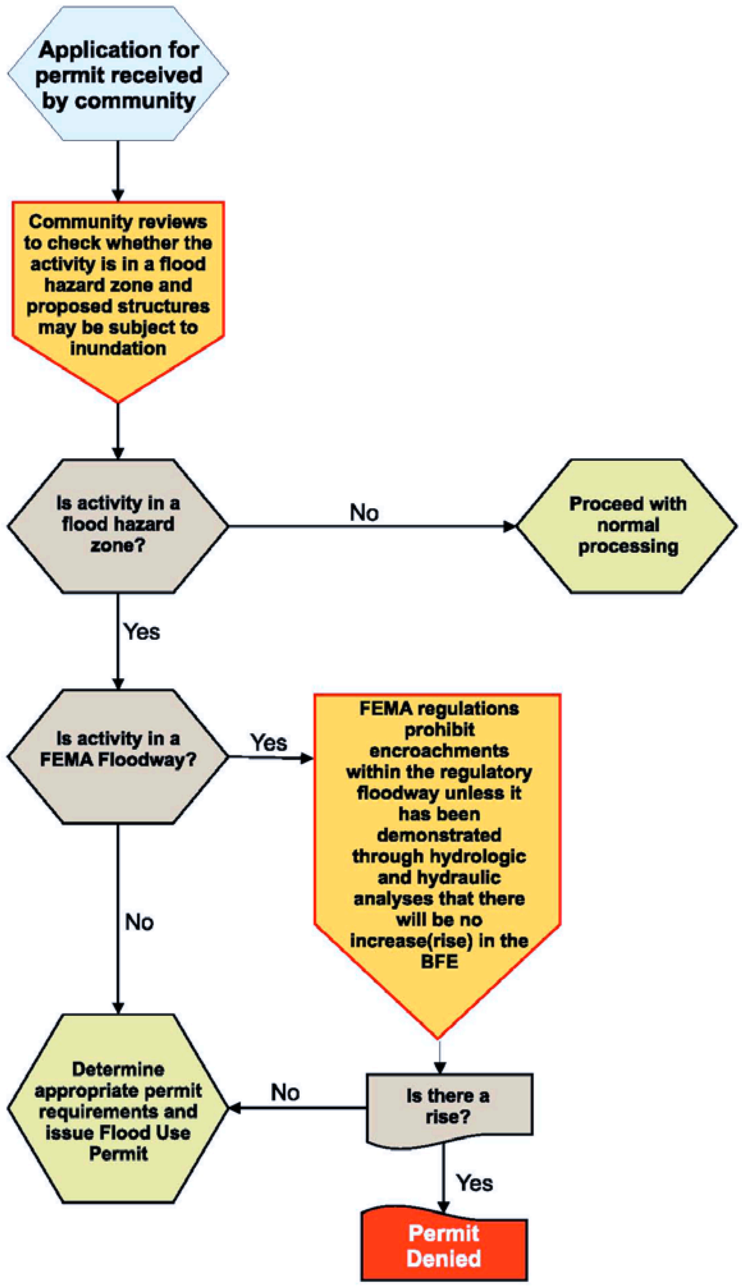
8.3.4 LOWEST FLOOR REQUIREMENTS WITHIN NON-FLOODPLAIN AREAS

The lowest floor elevation for dwelling units not within 100-year floodplain areas defined above shall be controlled by the provisions of the 2003 International Building Code, as adopted by Mohave County.

8.3.5 PERMITTING PROCESS

Figure 8.1 contains a flow chart of the floodplain use permit review process employed by the DISTRICT.

Figure 8.1 Floodplain use permit process



9 ROADWAY DRAINAGE

9.1 INTRODUCTION

The primary purpose of roadways is to serve transportation needs. Accommodation of roadway drainage is provided so that motorists and emergency vehicles have a reasonable level of access and safety during storm events. Storm water flowing within or across a roadway shall be managed in accordance with the Drainage Design Manual for Mohave County.

9.2 HYDRAULIC TECHNICAL DESIGN CRITERIA

Technical guidance for design of roadway and street drainage shall be in accordance with the street drainage chapter of the Hydraulics Manual. Specific design policies and standards for Mohave County needed for application of the technical information contained in the Hydraulics Manual are defined herein.

9.3 ROADWAY DRAINAGE POLICIES

1. Roadways shall be designed to convey storm water runoff so as to provide motorists and emergency vehicles access and safety during a storm event.
2. Roadways will be designed to convey storm water in conformance with Section 9.4.
3. Roadway flow velocities in excess of those established in Section 9.4 shall require prior approval by the County.
4. Inverted crown roadways shall not be permitted for County Highway or arterial roadways. Inverted crowns on collector/distributor and local roadways shall require prior approval.
5. Historic drainage divides should be retained wherever possible.
6. All runoff from the 100-year frequency storm should be conveyed within the roadway right-of-way.

9.4 ROADWAY DRAINAGE STANDARDS

9.4.1 GENERAL

The conveyance of storm water in a roadway is influenced by the typical roadway cross-section, cross-slope, longitudinal slope and roadway material. The following are standards to be used in the evaluation of roadway drainage conveyance:

9.4.1.1 Lowest Floor Elevations

Lowest floor elevations of buildings adjacent to the roadway right-of-way shall be elevated to provide a minimum of 12-inches of freeboard above the 100-year water surface elevation.

9.4.1.2 Hydrologic Method for Catch Basin Sizing

Runoff calculations for the sizing of storm drain catch basin inlets and connector pipes shall be based on the Rational Method.

9.4.1.3 Street Drainage n-value

A Manning's "n" value of 0.015 shall be used for roadway flow on paved roadways unless special conditions exist.

9.4.1.4 Valley Gutters

Valley gutters are only allowed on local and collector roadways. For valley gutters crossing collector roadways, the valley gutters shall provide mild slope transitions (maximum 5% total algebraic break over) to provide smooth vehicular ride across them, and shall be at least seven-feet wide.

9.4.1.5 Curb Returns

Curb returns should have a minimum slope of 0.01 foot of fall for every one foot of curb radius. For example, a 25 foot radius curb return should have at least 0.25 foot of fall from one end to the other.

9.4.1.6 Catch Basins on Continuous Grade

Catch basins on continuous grade are not required to intercept 100% of the design flow rate. 100% interception of the design flow rate may be required at roadway intersections.

9.4.1.7 Catch Basin Design

The curb opening for a catch basin shall not be greater than six (6) inches in height. Permissible catch basins are contained in the MAG Standard Details. The reduction factors, as identified in Table 9.1, shall be applied to the theoretical catch basin capacity to obtain the interception capacity used for design. The use of grated catch basins is discouraged within roadway sections. If a grated catch basin is used within a roadway section, only those grate types with bars transverse to traffic, or reticuline types, are acceptable. The reduction factors shall be applied to the design peak discharge to obtain the required interception capacity used for design.

9.4.2 DESIGN FREQUENCY, DEPTH AND VELOCITY REQUIREMENTS

The following are the drainage design standards for each roadway functional classification as defined in the Standard Details. The term "All-weather Access" refers to roadways designated by the COUNTY as providing primary access to a specific area for emergency ingress and egress purposes during extreme weather conditions. The roadway classifications "Cul-de-Sacs" and "Hillside/Frontage" defined in the Standard Details are included under the Local Roadway classification for the purposes of these standards. The frequency, such as 10-year or 100-year, refers to the conveyance of the storm water runoff peak discharge corresponding to the listed storm frequency (i.e. a 100-year storm has a one percent probability of occurring or being exceeded in any given year). Peak discharges for roadway drainage design are normally estimated using the Rational Method (Section 7.3.2). Peak discharges estimated using the Unit Hydrograph Method may also be used where appropriate, particularly where runoff from large offsite watersheds is conveyed in a channel within the roadway right-of-way. Maximum flow depth and velocity criteria are for any location within the roadway travel lanes, as defined on the Standard Details. The maximum velocity is for all design storms up to and including the 100-year storm frequency.

The 100 year flow depth may not exceed the road right-of-way limits. If flows exceed the hydraulic carrying capacity of the street within the right-of-way limits, then additional drainage conveyance will be required. Such additional drainage conveyance may include but is not limited to: 1) a permanent drainage channel within an easement or dedicated drainage parcel adjacent to the street right-of-way; 2) storm sewer under or adjacent to the street; 3) other appropriate good engineering design practice. Additional depth-velocity criteria are specified in the following sections.

When analyzing flow entering streets or within the street section, superelevation effects on the water surface due to bends must be taken into account and calculated to ensure adequate freeboard for all flows turning into cross streets at an intersection, or street curves.

9.4.2.1 Roadways with Curb and Gutter:

Channel and/or storm drain systems are to be installed as needed to meet the following roadway drainage criteria for depth and velocity.

1. County Highway/Arterial/All-weather Access:
    a. Inverted crowns are not allowed.
    b. 10-year: One 12-foot dry lane each direction, and flow depth shall not exceed curb height.
    c. 100-year: Maximum flow depth between curbs <= 8-inches
    d. Maximum velocity = 8 fps.
2. Collector/Distributor/Local Roadways:
    a. Inverted crowns are allowed.
    b. 10-year: Flow depth shall not exceed curb height.
    c. 100-year, normal crown: Maximum flow depth between curbs <= 8-inches
    d. 100-year, inverted crown: Maximum flow depth between curbs <= 12-inches
    e. Maximum velocity = 8 fps.

9.4.2.2 Roadways without Curb and Gutter:

1. All Roadway Classifications: 100-year peak discharge shall be contained in a channel with the maximum design storm flow depth not to exceed the outside shoulder hinge point shown on the Standard Details.

9.4.3 CATCH BASINS

Capacity reduction factors for catch basins are listed in Table 9.1. The reduction factors shall be applied to the theoretical catch basin capacity to obtain the interception capacity used for design.

Table 9.1 Inlet Capacity Reduction Factors

Condition	Inlet Type	Reduction Factor
Sump	Curb Opening	0.80
Sump	Grated	0.50
Sump	Combination	0.65
Continuous Grade	Curb Opening	0.80
Continuous Grade	Longitudinal Bar Grate Longitudinal Bar Grate with recessed transverse bars Longitudinal Bar Grate with transverse bars Reticuline Grate	0.75 0.60 0.40 0.35
Continuous Grade	Combination	Apply factors separately to grate and curb opening
Shallow Sheet Flow (1)	Slotted Drains	0.80

(1) Slotted drains are most effective for shallow sheet flow conditions. With greater depths and flows, a different type of inlet should be used.

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10 STORM DRAINS

10.1 INTRODUCTION

This chapter contains technical standards that should be used for the hydraulic design of a storm drain system. In this manual, a storm drain system refers to a coordinated group of inlets, underground conduits, manholes, and various other appurtenances which are designed to collect stormwater runoff from the design storm and convey it to a point of discharge into a major or regional drainage outfall. Storm drains should generally only be considered for minor watercourses in urban areas. The use of storm drains is generally discouraged for unincorporated Mohave County primarily because of maintenance issues associated with sedimentation. Storm drains typically are not economical for the flows conveyed within larger watercourses. Therefore, the storm drain system will collect runoff to a point where storm drains become too large to be economical and will then discharge into a major or regional watercourse outfall consisting of a man-made channel, or natural watercourse.

10.2 HYDRAULIC TECHNICAL DESIGN CRITERIA

Technical guidance for design of storm drains shall be in accordance with the storm drains chapter of the Hydraulics Manual. Specific design standards for Mohave County needed for application of the technical information contained in the Hydraulics Manual are defined herein.

10.3 STORM DRAIN STANDARDS

The following minimum standards including the requirements in Table 10.1 are to be met for the design of storm drains that will be placed into the Mohave County Public Works or Mohave County Flood Control District maintenance systems.

10.3.1 HYDRAULIC DESIGN REQUIREMENTS

10.3.1.1 Design Discharge

The design discharge (Q_{design}) shall be the peak discharge that must be conveyed in the storm drain in order to meet the roadway drainage design criteria for depth and velocity specified in Section 9.4.2.1. Storm drains with slopes less than 0.5%, and flow velocities less than five fps for Q_{design} or in excess of 15 fps shall require special approval by the COUNTY/DISTRICT.

Table 10.1 Storm drain hydraulic design standards	
Design Variable	**Design Standard**
Minimum Velocity.	5 fps for Q_{design} The lesser of 3 fps for 0.5 x Q_{design} or 3 fps at flow depth = 1'
Maximum Velocity.	15 fps
Minimum Slope	0.005 ft/ft
Minimum Pipe Size. Main Line (straight horizontal and vertical alignment) Main Line (curved horizontal and/or vertical alignment) Lateral and Connectors	18-inches 24-inches 15-inches
Pipe Diameter Changes.	The elevation of pipe crowns, not inverts, are to be matched at manholes and structures.
Pipe Joint Deflections: Main Line Lateral and Connectors	Maximum of 0.5 times the manufacturer's recommended tolerance. Not allowed
Maximum Distance to First Catch Basin.	The first catch basin should be placed where the roadway drainage hydraulic criteria from Section 9.4.2.1 is first exceeded.
Manhole Spacing (SD = Storm Drain Diameter).	≤30 inches SD (straight) = 330 feet max 33-45 inches SD = 440 feet max 48-84 inches SD = 660 feet max >84 inches SD = 1,320 feet max
Maximum Hydraulic Grade Line Elevation, Q_{design}.	Shall not be higher than 12 inches below inlet gutter flowline elevation
Maximum Energy Grade Line Elevation, Q_{design}.	Shall not exceed gutter flowline elevation

10.3.1.2 Hydraulic and Energy Grade Lines

Storm drain systems shall be designed for Q_{design} so that the hydraulic grade line is at least 12 inches below the inlet gutter flowline elevation, and the energy grade line shall not exceed the elevation of the gutter flowline. Hydraulic and energy grade line information for all main line and connector storm drain pipes shall be prepared by the design engineer and submitted to the COUNTY/DISTRICT for approval. The information shall be provided in tabular and profile format and shall include: pipe stationing, pipe size, pipe discharge (Q), pipe velocity, pipe material, hydraulic grade line, energy grade line, and finish grade over pipe.

10.3.1.3 Storm Drain Pipe Materials

Pipe culverts are to be corrugated steel pipe (CSP) or rubber gasket reinforced concrete pipe (RGRCP). Pipe wall thickness, corrugations, and linings are to be determined based on engineering analysis using the design guidelines provided by the Corrugated Steel Pipe Institute (CSPI) and the American Iron and Steel Institute (AISI), *Handbook of Steel Highway and Drainage Construction Products* (CSPI, 2002), and the American Concrete Pipe Association (ACPA) *Concrete Pipe Design Manu*al (ACPA, 2007).

If soil resistivity readings are below 1500 ohms per cubic centimeter, CSP will not be allowed.

The minimum gauge for CSP storm drain pipe shall be 14 gage. The specific gage specified, in addition to meeting loading requirements, shall provide a design life of at least 75 years to first perforation based on soil conditions (see Figure 10.1).

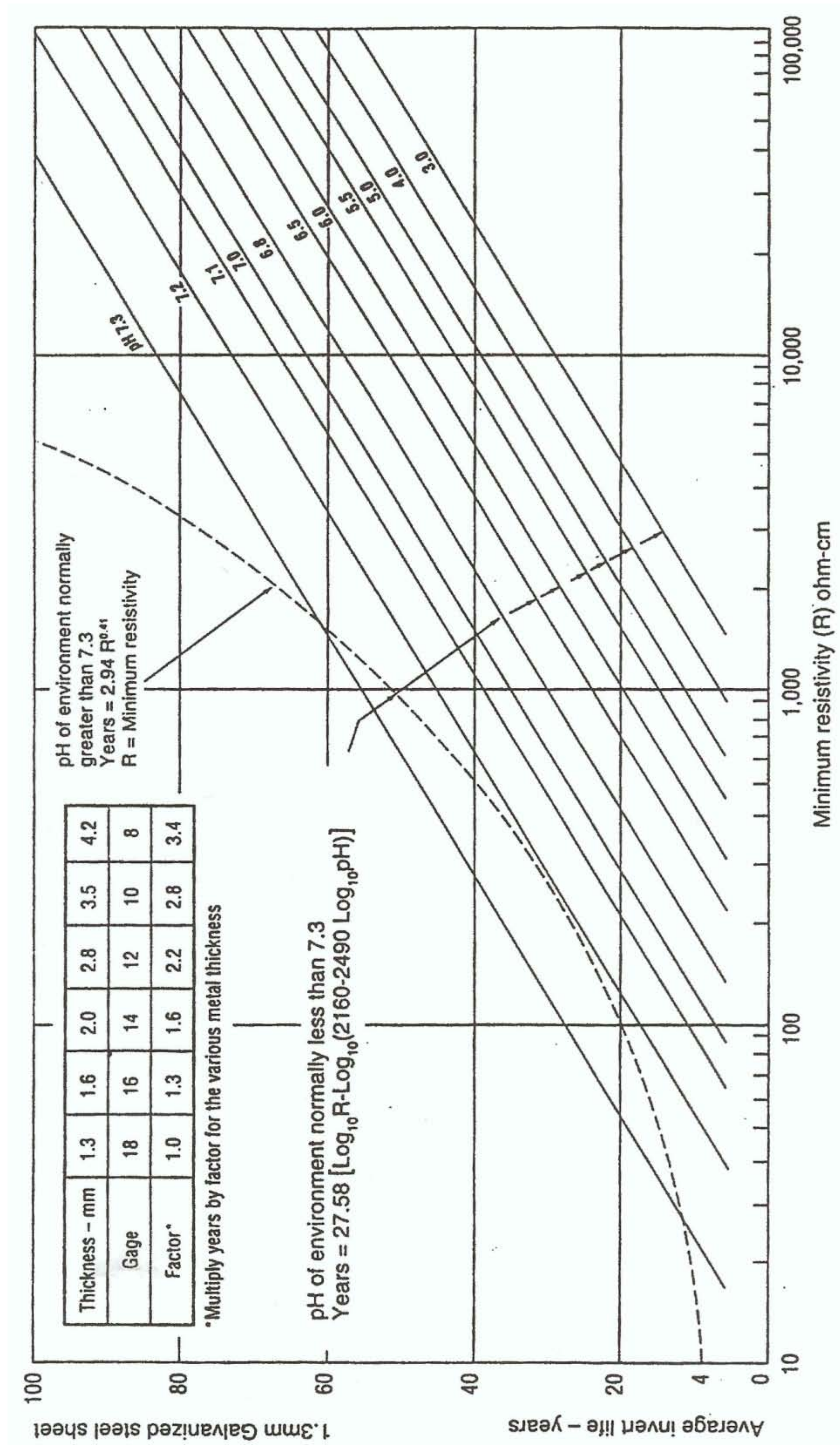
RGRCP should be designed to fall within the allowable ranges identified in the "D" load table (see Figure 10.2). The use of Figure 10.2 will provide a conservative design. If a more economical design is deemed necessary, the Direct Design Method set forth in ACPA (2007) may be used. D-Load requirements shall be determined using a 140 pcf earth load. In ordinary soil conditions, the "positive projected condition" shall be used for up to ten feet of cover. "Trench condition" shall be used for deeper trenches in ordinary soil conditions. If the soil information indicates unstable soil, the positive projected condition shall be used exclusively in determining D-Load requirements.

10.3.1.4 Minimum Cover Requirements

Minimum cover of fill over storm drains should be at least four (4) feet in order to minimize conflicts with other utilities. A lesser minimum cover may be may allowed if engineering justification is submitted, and approved by COUNTY/DISTRICT.

Figure 10.1 Minimum gage thickness for CSP

(from CSPI, 2002, pg 350)

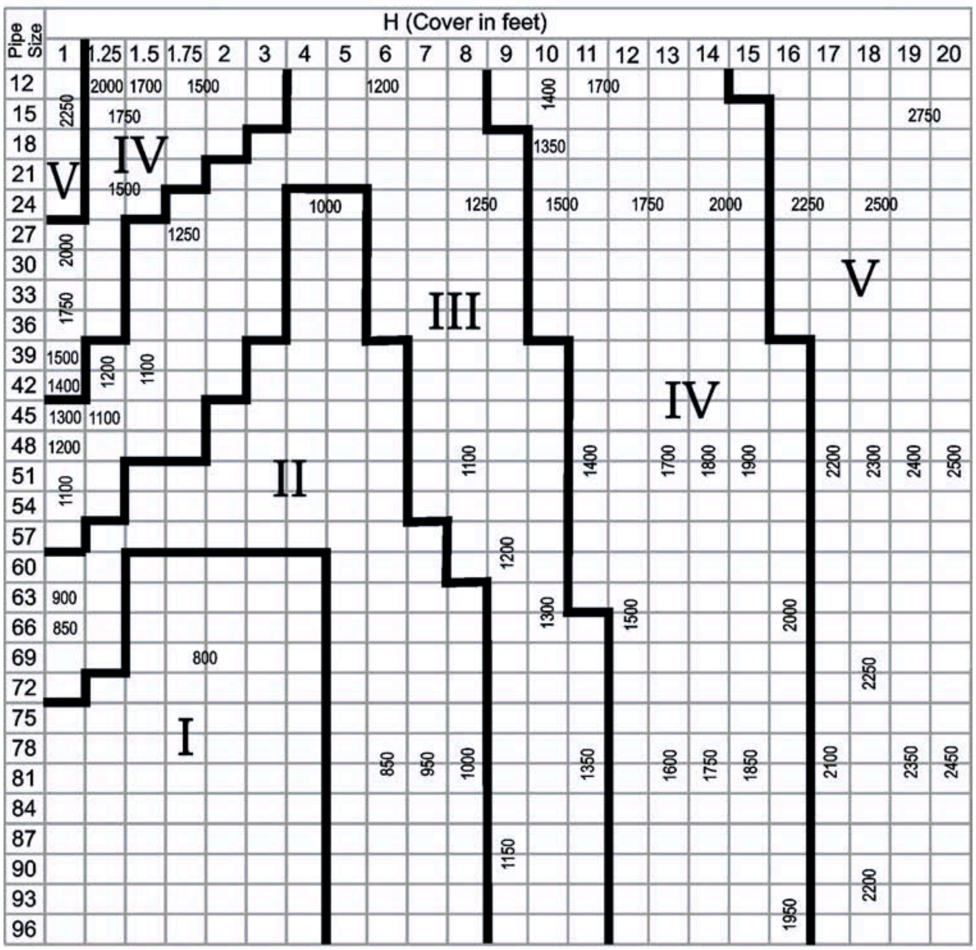


Thickness – mm	1.3	1.6	2.0	2.8	3.5	4.2
Gage	18	16	14	12	10	8
Factor*	1.0	1.3	1.6	2.2	2.8	3.4

Figure 10.2 Required "D" Load for reinforced concrete pipe

(from City of Phoenix, 2004)

Required “D” Load For Reinforced Concrete Pipe
Positive Projection Condition
(140 P.C.F. Earth Load)



Structural Notes:

1. Ku = 0.1924
2. For H< 10, P = 1.0, He/Bc = 1.7
3. For H> 10, P = 0.5, He/Bc = 1.4
4. Dead Load Factor = 1.9
5. Safety Factor = 1.0
6. Earth Loading = 140 pcf (Marston’s Formula)
7. Live Loading: H-20, S-16 Truck, AASHO Impact

10.3.1.5 Manhole Requirements

Manholes are required for all mainline storm drain pipe size changes, vertical grade breaks, horizontal angle deflections great than five (5) degrees, mainline pipe intersections, and periodic locations for access and maintenance. Maximum manhole spacing shall conform to the requirements in Table 10.1. Typically, when mainline pipe size changes, the inside top of pipe elevations (pipe crowns) shall be matched. In the event where the downstream pipe is smaller than the upstream (such as when a required 6-inch oversized alternate pipe is attached to RGRCP), then the pipe invert elevations shall be matched.

10.3.1.6 Connector Pipe Requirements

Catch basin connector pipes shall be connected to new mainline storm drain pipes with prefabricated tees. One catch basin connector pipe may be joined to the mainline at a manhole if the standard required mainline manhole spacing provides a convenient location for it. If there are two directly opposing catch basins to connect to the mainline at a manhole, only one connector pipe may connect to the manhole and the second connector pipe shall connect to the mainline by way of a prefabricated tee at least 5-feet downstream of the manhole. Where connector pipes are to be joined to existing mainline storm drains, they shall be connected by manhole. Connection of new mainline pipe to new mainline pipe or existing mainline pipe shall be by manhole or special junction structure depending on size and feasibility of manhole or special junction structure installation.

To minimize headloss and maintain structural integrity of the mainline pipe, the following shall apply:

1. Opposing catch basin connector pipes connecting to a mainline storm drain pipe shall be offset a minimum of five feet horizontally as measured from the centerline of each connector pipe.
2. Opposing storm drain laterals greater than 24 inches in diameter shall be joined by a special junction structure designed to minimum HS-20 loading by a registered professional engineer. The junction structure shall be designed to be hydraulically efficient.

10.3.1.7 Pipe Deflections and Bends

The minimum diameter for pipes designed on a horizontal or vertical curve shall be 24-inch. Joint deflections used to form horizontal and vertical curves in mainline storm drain pipe shall not exceed 0.5 times the manufacturer's recommended maximum tolerance. Connector pipes for laterals to catch basins shall not be curved. Specifications for horizontal deflection using this method shall be noted on the construction plans, citing manufacturer's requirements. An angular bend in catch basin connector pipe (horizontal or vertical), up to, and including 22½ degrees may be accomplished by using a standard MAG Detail 505 pipe collar. Prefabricated pipe bends shall be required for deflections in catch basin connector pipes greater than 22½ degrees. In addition, the maximum angle for a catch basin connector pipe to exit any wall of a catch basin shall be 22½ degrees from perpendicular.

10.3.1.8 Outlet Protection Requirements

Storm drain outlet requirements shall conform to the requirements set forth in Section 11.4.12.

10.3.1.9 Construction Plan Requirements

Storm drain pipes and manholes shall be shown in plan and profile along with existing and proposed grades of the pipeline and both the existing and proposed ground surface above the pipeline. Catch basin and connector pipe profiles shall also be provided in the design drawings. The pipe size and slope to four significant figures shall be shown. All existing utilities, including water and sanitary sewer, crossing the proposed storm drain shall be shown in plan, and in profile at their proper elevation. Proposed new utilities shall be labeled and shown at anticipated locations and standard depths when exact information is not yet known. Clearance with water and sewer facilities require a minimum of six feet horizontally and one foot vertically. Clearance with other utilities shall be a minimum of one foot both horizontally and vertically.

Refer to Section 18.3 for additional construction drawing requirements.

10.3.1.10 Soil Boring Requirement

Soil boring information is required for all pipe materials. Soil boring logs shall be provided with the design documentation for all storm drains within public right-of-way. Storm drains in excess of 660 feet in length shall have multiple borings at intervals not to exceed 660 feet. Boring depths shall be at least two feet below the proposed pipe invert. If cemented or rock material is encountered during drilling which results in refusal, then the geotechnical engineer shall determine the specific limits of rock and identify the type and extent of refusal to at least two feet below the proposed pipe invert.

Borings shall be located in plan and profile view and tied to the same vertical datum as the proposed project. Resistivity and pH testing of the soils shall be required to support pipe design in terms of alternate pipe material selection. If resistivity readings fall below 1500 ohms per cubic centimeter, additional readings shall be made at intervals of not less than 25 feet or more than 100 feet until the limits of the area of low resistance soil are fully defined.

Boring log data shall include the following information:

1. The name of the company that produced the soil report.
2. The name of the project that the borings apply to.
3. The date the test boring was made.
4. The type of equipment used to drill the hole and take the samples.
5. The size of the auger used.
6. The vertical datum used.
7. A description of caving that occurred during the excavation, if any.
8. Horizons of each type of soil encountered.
9. Description of the soil.
10. Classifications by the Unified Soil Classification System.
11. Plasticity index.
12. Percent passing No. 200 sieve.
13. Water encountered.
14. Pavement structure (AC thickness, sub-base thickness, if applicable).
15. Relative moisture content (specify depth taken).
16. Representative unit weight of native material (specify depth taken)
17. Laboratory calculated optimum moisture content.
18. Resistivity and pH readings.

10.3.1.11 Parcel or Easement Requirement

A County owned property, dedicated right-of-way, privately owned drainage parcel, or drainage easement shall be a minimum of 16 feet wide for underground storm drains if not under a designated road right-of-way. Additional width may be required for large storm drains and/or trench depths.

11 CULVERTS AND BRIDGES

11.1 INTRODUCTION

Bridges are defined as structures designed to span a watercourse, including bridges for vehicular roadways and pedestrian-only uses. Culverts are buried pipe or box hydraulic conveyance structures designed to convey stormwater from one side of a roadway, embankment, or service area to the other side. The following are minimum policies and standards to be employed in the design of culverts and bridges that will be placed into the Mohave County maintenance system. These policies and standards may also apply to private streets and roadways.

11.2 HYDRAULIC TECHNICAL DESIGN CRITERIA

Technical guidance for design of roadway culverts and bridges shall be in accordance with the culverts and bridges chapter of the Hydraulics Manual. Additional guidance may be obtained from FHWA (2005c). Specific design policies and standards for Mohave County needed for application of the technical information contained in the Hydraulics Manual are defined herein. General recommended practices for culvert design and construction are:

- A single more hydraulically efficient culvert should be used instead of multiple smaller culverts when possible. This is desirable to minimize cost and to facilitate maintenance.
- Multiple culverts should be used when mimicking natural hydraulic conditions and large width to depth ratios need to be maintained. Multiple culverts may also be necessary when there is a need to minimize backwater effects.
- Pipe inlet and outlets should extend past the toe of the backfill or necessary slope protection unless headwalls or end sections are provided and properly situated.
- The combination of culvert vertical alignment and hydraulic characteristics should be carefully designed to facilitate sediment conveyance and to minimize scour and head cutting.
- New roadway crossings requiring culverts should be designed perpendicular to the natural drainage whenever possible.

- Culverts shall be checked for both inlet and outlet control conditions and the design based on the condition with the highest headwater depth at the inlet.
- Culverts shall be properly bedded and backfilled in conformance with MAG Standards and Specifications.

11.3 CULVERT AND BRIDGE POLICIES

The following are Mohave County policies related to culverts and bridges.

11.3.1 CULVERT/BRIDGE BACKWATER PONDING

A county-owned property, right-of-way, or privately-owned drainage tract or easement shall be provided for the area inundated by backwater from the culverts for the peak 100-year event. The 100-year floodplain limits are to be delineated and shown on the subdivision Final Plat or Map of Dedication. If the culvert or bridge lies within a FEMA-delineated 100-year floodplain, the encroachment shall not result in a rise in 100-year water surface elevation above the Regulatory Flood Elevation or a change in the Regulatory Floodway without processing of a CLOMR and a LOMR.

11.4 CULVERT AND BRIDGE STANDARDS

11.4.1 REQUIREMENT TO PROVIDE CULVERTS OR BRIDGES

Except where low water crossings are allowed as specified herein, watercourses found to meet the following conditions are to be culverted or bridged,:

1. A watercourse with a 100-year peak discharge of 25 cfs or greater,
2. A watercourse that is a regulatory area designated as “Waters of the United States” under Section 404 of the Clean Water Act (refer to Section 2.3.2), or
3. As necessary in order to preserve natural flow patterns and prevent adverse impacts on adjacent, upstream and downstream properties.

11.4.2 CULVERT MATERIAL REQUIREMENTS

Pipe culverts are to be corrugated steel pipe (CSP), rubber gasket reinforced concrete pipe (RGRCP), and reinforced concrete box culverts (RCBC). CSP and RGRCP may be either circular or arch pipes. Steel structural plate pipes or arches may also be used but require prior approval by COUNTY/DISTRICT. Concrete pipes and box culverts may be either precast or cast-in-place.

Pipe wall thickness, corrugations, and linings are to be determined based on engineering analysis using the design guidelines provided by the Corrugated Steel Pipe Institute (CSPI) and the American Iron and Steel Institute (AISI), *Handbook of Steel Highway and Drainage Construction Products* (CSPI, 2002). Refer to Section 10.3.1.3 for resistivity and minimum pipe thickness or design class requirements for storm drains, which also apply to CSP and RGRCP used for culverts.

11.4.3 DESIGN STORMS

Culverts are to be designed to convey, as a minimum, the design storm frequency peak discharge listed below by roadway classification with the design storm maximum water surface not to exceed the shoulder hinge point as defined for each roadway classification in the Standard Details.

1. County Highway/Arterial/All-weather Access:
   4-lanes 50-year peak discharge
   2-lanes 25-year peak discharge
2. Collector/Distributor and Local: 10-year peak discharge.
3. All roadway classifications:
   a. Roadway Overtopping. The maximum depth in the roadway travel lanes in dip sections for the 100-year peak discharge shall be 12-inches. If culvert overtopping flow does not cross the road, but turns and drains down the roadway, the maximum depth of flow within the roadway travel lanes shall be 8-inches.
   b. Bridges. Runoff to be conveyed under the road with 2-feet of freeboard below the bridge low chord, including the effects of pier clogging.
   c. Driveway Culverts within Mohave County Right-of-Way: Sized so the design storm criteria for the roadway classification are met.

11.4.4 MINIMUM PIPE DIAMETER

1. All roadway classifications: 24-inch minimum
2. Driveway culverts in Mohave County Right-of-Way: 18-inch minimum.

11.4.5 MINIMUM COVER REQUIREMENTS

Minimum cover of fill over culverts should be at least eighteen (18) inches. A lesser minimum cover may be may allowed if engineering justification is submitted, and approved by COUNTY.

11.4.6 CLOGGING FACTORS

1. **Culverts**. In high debris/sediment areas, COUNTY may elect to require the following clogging factors be applied to the design cross section area of the culvert opening.

Culvert Size	Clogging Factor
Equivalent diameter <= 48 inches	Reduce available opening area by 50%
Equivalent diameter > 48 inches	Reduce available opening area by 20%

2. **Bridges**.

   Floating Debris Allowance for Bridge Modeling and Design:

   Hydraulic Analysis of Bridges: Bridge pier sizes shall be modeled as twice their structural width or 1 foot on each side, whichever is greater. When warranted by the potential for debris from the watershed, larger debris width increases may be required.

   Hydraulic Analysis of Box Culverts: When warranted by the potential for debris from the watershed, an allowance of 1 foot of debris on each side of box culvert inlets at interior walls shall be considered when calculating the hydraulic capacity of box culverts.

   Bridge Pier Modeling For Local Scour Calculations: The following minimum modifications to the pier shape shall be applied in hydraulic models for structural design purposes to calculate pier local scour depth and water pressure on piers. These minimum modifications are intended to supplement AASHTO LRFD Bridge Design Specifications (AASHTO, 2004), unless otherwise directed by COUNTY/DISTRICT.

   a. To account for drift/debris build-up, increase pier column width/diameter, within the top 12 feet of water depth (per ADOT Bridge Design Guidelines), to twice the design value, but no less than two feet on each side.

   b. Larger pier width increases up to half span length on each side may be considered when warranted by the potential for debris from the watershed,.

   c. For deep drilled shaft foundations, in the area below the bottom of casing, increase the shaft design diameter by one foot on each side.

3. **Trash and Debris Racks**. Where trash and/or debris racks are required under Chapter 4, culvert design capacity shall be adjusted as defined in the Hydraulics Manual.

11.4.7 LOW WATER CROSSINGS

1. Low water crossings are allowable for long areas of shallow or distributary flow where the MOHAVE COUNTY ENGINEER determines that construction of culverts is impractical, would significantly change historical flow patterns, or would result in significant adverse impacts to properties.

2. Low water crossings shall have erosion protection sufficient to withstand the impacts of a 100-year flood.

3. Low water crossings shall be signed in accordance with Mohave County Public Works requirements.

4. The maximum flow depth in the travel lanes for the 100-year peak discharge is 12-inches for all roadway classifications.

11.4.8 PONDING OUTSIDE OF RIGHT-OF-WAY

Backwater ponding limits that extend outside of the roadway right-of-way shall be delineated and a drainage easement or right-of-way obtained from the property owner. Drainage easements shall be recorded and attached to the deed for the property.

11.4.9 HEADWALL REQUIREMENTS

Headwalls are required at the inlet and outlet of all culvert installations unless otherwise approved by COUNTY. Pipe sizes of 48-inch or greater shall have concrete headwalls. Pipe sizes less than 48-inch shall have concrete headwalls if trash racks are required to comply with safety requirements specified in Section 4. Otherwise, pipe sizes less than 48-inch shall have flared end sections or concrete/masonry headwalls.

11.4.10 MAINTENANCE ACCESS

Ramped, vehicular access for maintenance is required at the upstream and downstream ends of all culverts that are not accessible from the roadway. The maintenance access route shall be within public right-of-way or a COUNTY approved easement and shall have a minimum drivable width of twelve (12) feet.

11.4.11 VELOCITY REQUIREMENTS

1. Design velocity and slope requirements shall conform to those specified in Table 11.1.
2. Culverts are to be designed with consideration to the guidelines presented in the Culverts and Bridges, and Sedimentation chapters in the Hydraulics Manual.
3. The culvert shall be designed so minimum velocities facilitate sediment transport to keep the culvert clean.
4. The maximum velocity in the culvert should be consistent with channel stability requirements at the culvert outlet. Aggradation or degradation at culvert crossings shall be examined in the design of culverts.

Table 11.1 Culvert hydraulic design standards

Design Variable	Design Standard
Minimum Velocity.	5 fps for Q_{design} The lesser of 3 fps for 0.5 x Q_{design} or 3 fps at flow depth = 1'
Maximum Velocity.	15 fps

Minimum Slope	0.005 ft/ft
Pipe Curvature	Not allowed.

11.4.12 OUTLET PROTECTION REQUIREMENTS

11.4.12.1 General

Culvert outlet protection requirements shall conform to the design criteria set forth in Table 11.2. Design guidelines for the types of outlet protection listed in Table 11.2 can be found as follows:

Cut-off Walls: See Section 11.4.12.2.

Riprap Aprons: See Section 11.4.12.3.

Riprap Basins: See Chapter 8, Section 8.4.3 of the Hydraulics Manual

Concrete Outlet Structures: See Section 8.4.4 of the Hydraulics Manual.

<table>
<tr><th colspan="4">Table 11.2 Design criteria for culvert outlets</th></tr>
<tr><th rowspan="2">Outlet Protection Type</th><th rowspan="2">Froude Number Criteria</th><th colspan="2">Velocity Criteria $(V_{max} = 15\ fps)$</th></tr>
<tr><th>Natural Channel</th><th>Artificial Channel</th></tr>
<tr><td>None</td><td colspan="3">$F_r \leq 1.0$ and:
Scour analysis shows outlet protection is not necessary</td></tr>
<tr><td>Cut-off wall</td><td>$F_r \leq 1.0$</td><td>Up to 1.3 times existing channel velocity</td><td>Up to maximum allowable velocity for channel lining</td></tr>
<tr><td>Simplified Riprap Apron Method

(single culvert installations only)</td><td>$F_r \leq 2.5$
Circular culverts 60-inches and smaller where:
$\frac{Q}{D_c^{2.5}} \leq 6$</td><td rowspan="3">1.3 to 2.5 times existing channel velocity.
A limiting velocity at the downstream end of the apron of 5 fps.</td><td rowspan="3">1.0 to 2.5 times allowable channel lining velocity</td></tr>
<tr><td>Detailed Riprap Apron Method

(single and multiple culvert installations)</td><td>$F_r \leq 2.5$
Circular culverts where:
$\frac{Q}{D_c^{2.5}} \leq 6$
Box culverts where:
$\frac{Q}{WH^{1.5}} \leq 8$</td></tr>
<tr><td>Riprap Basin</td><td>$F_r \leq 2.5$</td></tr>
<tr><td>Concrete Outlet Structure</td><td>n/a</td><td>Velocities greater than 2.5 times existing channel velocity</td><td>Velocities greater than 2.5 times allowable channel lining velocity</td></tr>
<tr><td colspan="4">where:
F_r = Froude number in culvert barrel at outlet
D_c = culvert diameter, in feet
W = box culvert opening width, in feet
H = box culvert opening height, in feet</td></tr>
</table>

11.4.12.2 Cut-off Walls

Culverts with headwalls shall have cut-off walls where dictated by scour depth. If cut-off walls are determined to be necessary, then minimum cut-off wall depths shall be as indicated in Table 11.3. For pipes larger than 24 inches, cut-off wall depth shall be dictated by the greater of the depth shown in the table or that depth calculated using the depth of scour equation identified in Chapter 5 of the Hydraulics Manual. Cut-off walls will normally be concrete in conformance with the MAG Standards. Riprap may be used as an alternative. When riprap is used, multiply the depth specified in Table 11.3 by 1.25. Extend the riprap horizontally a minimum of one pipe diameter upstream from the culvert outlet at that depth.

Table 11.3 Design criteria for culvert cut-off walls

MAG Standard Pipe Diameter	Minimum Inlet & Outlet Cutoff Wall Depth (feet)
24” to 48”	2.0
48” to 84”	4.0

11.4.12.3 Riprap Aprons

Most culvert outlet protection designs will utilize a riprap apron approach. For riprap aprons, the riprap rock size and apron length may be designed for single circular culverts less than or equal to 60-inches in diameter using the Simplified Riprap Apron Method. This method will provide a conservative design for most applications. If a less conservative design is desired, the design requires the use of a box culvert or multiple culverts, or culvert sizes greater than 60-inch are required, the Detailed Riprap Apron Method should be applied, or a riprap basin approach used.

Detailed Riprap Apron Method

The most commonly used device for conduit/culvert outlet protection is a riprap apron. The procedures presented in this section are derived from *Urban Drainage and Flood Control District Drainage Criteria Manual* (UDFCD, 2001). Scour resulting from highly turbulent, rapidly decelerating flow is a common problem at conduit outlets. The riprap apron protection design protocol is suggested for conduit and culvert outlet Froude numbers up to 2.5 where the channel and conduit slopes are parallel with the channel gradient and the conduit outlet invert is flush with the rip-rap channel protection. Here, Q is the discharge in cfs, D_c is the diameter of a circular conduit in feet and *W* and *H* are the width and height, respectively, of a rectangular conduit in feet.

The procedure in this section was evaluated by the FHWA in HEC-14, Appendix D (FHWA, 2006). It was found to be an acceptable procedure, but less conservative than the procedure recommended in Chapter 10 of HEC-14. The procedure in this section allows for design of culvert outlet protection for both circular and box culverts, while the FHWA HEC-14 Chapter 10 procedure only addresses circular pipe culverts. The selection was made based on the need for an acceptable procedure applicable to both circular and box culverts, the need for a more definitive method of defining the extent of protection required than the FHWA HEC-14 Chapter 10 provides, and the need for a procedure that is safe but not overly conservative. An example schematic of an apron from UDFCD (2001) is shown in Figure 11.1. Figure 11.1 illustrates typical riprap protection of culverts and major drainageway conduit outlets. The additional thickness of the riprap just downstream from the outlet is to assure protection from turbulent flow conditions that might precipitate rock movement in this region. The procedures in this section should only be applied for the outlets of circular pipes 72-inch in diameter or smaller, box culverts with an opening height of 72-inches and smaller, and for a riprap d_{50} requirement of 24-inches or less, within the limitations shown on Figure 11.2 and Figure 11.3. The procedure may be applicable to other situations, but the designer should receive prior approval from COUNTY/DISTRICT before applying it. For conduits/culverts that do not meet the requirements of this procedure, a riprap basin or concrete outlet structure should be used (refer to Hydraulics Manual Section 8.4.3 or Section 8.4.4).

Required Rock Size. The required rock size may be selected from Figure 11.2 for circular conduits and from Figure 11.3 for rectangular conduits. Figure 11.2 is valid for $Q/D_c^{2.5}$ of 6 or less and Figure 11.3 is valid for $Q/WH^{1.5}$ of 8.0 or less. The parameters in these two figures are:

1. $Q/D_c^{1.5}$ or $Q/WH^{0.5}$ in which Q is the design discharge in cfs, D_c is the diameter of a circular conduit in feet, and W and H are the width and height of a rectangular conduit in feet.
2. Y_t/D_c or Y_t/H in which Y_t is the tailwater depth in feet, D_c is the diameter of a circular conduit in feet, and H is the height of a rectangular conduit in feet. In cases where Y_t is unknown or a hydraulic jump is suspected downstream of the outlet, use $Y_t/D_c = Y_t/H = 0.40$ when using Figure 11.2 and Figure 11.3.
3. The riprap size requirements in Figure 11.2 and Figure 11.3 are based on the non-dimensional parametric Equation 11.1 and Equation 11.3 (Steven, Simons, and Watts

1971, and Smith 1975). Both equations are solved for d_{50} to produce Equation 11.2 and Equation 11.4.

Circular culvert:

$$\frac{\left(\frac{d_{50}}{D_c}\right)\left(\frac{Y_t}{D_c}\right)^{1.2}}{\frac{Q}{D_c^{2.5}}} = 0.023 \qquad 11.1$$

Solving for d_{50}:

$$d_{50} = \frac{0.023 Q D_c^{-1.5}}{\left(\frac{Y_t}{D_c}\right)^{1.2}} \qquad 11.2$$

where:

d_{50} = median rock size, in feet
D_c = culvert diameter, in feet
Y_t = tailwater depth, in feet
Q = design peak discharge, in cfs

Rectangular culvert:

$$\frac{\left(\frac{d_{50}}{H}\right)\left(\frac{Y_t}{H}\right)}{\frac{Q}{WH^{1.5}}} = 0.014 \qquad 11.3$$

Solving for d_{50}:

$$d_{50} = \frac{0.014 Q H^{0.5}}{Y_t} \qquad 11.4$$

where:

W = width of rectangular culvert, in feet
H = height of rectangular culvert, in feet

The rock size requirements were determined assuming that the flow in the culvert barrel is sub-critical. It is possible to use Equation 11.1, Equation 11.2, Equation 11.3 and Equation 11.4 when the flow in the culvert is supercritical and less than full if the value of D_c or H is modified for use in Figure 11.2 and Figure 11.3. Whenever the flow is supercritical in the culvert, substitute D_a for D_c and H_a for H, in which D_a is defined as:

$$D_a = \frac{(D_c + Y_n)}{2} \qquad 11.5$$

in which the maximum value of D_a shall not exceed D_c, and

$$H_a = \frac{(H + Y_n)}{2} \qquad 11.6$$

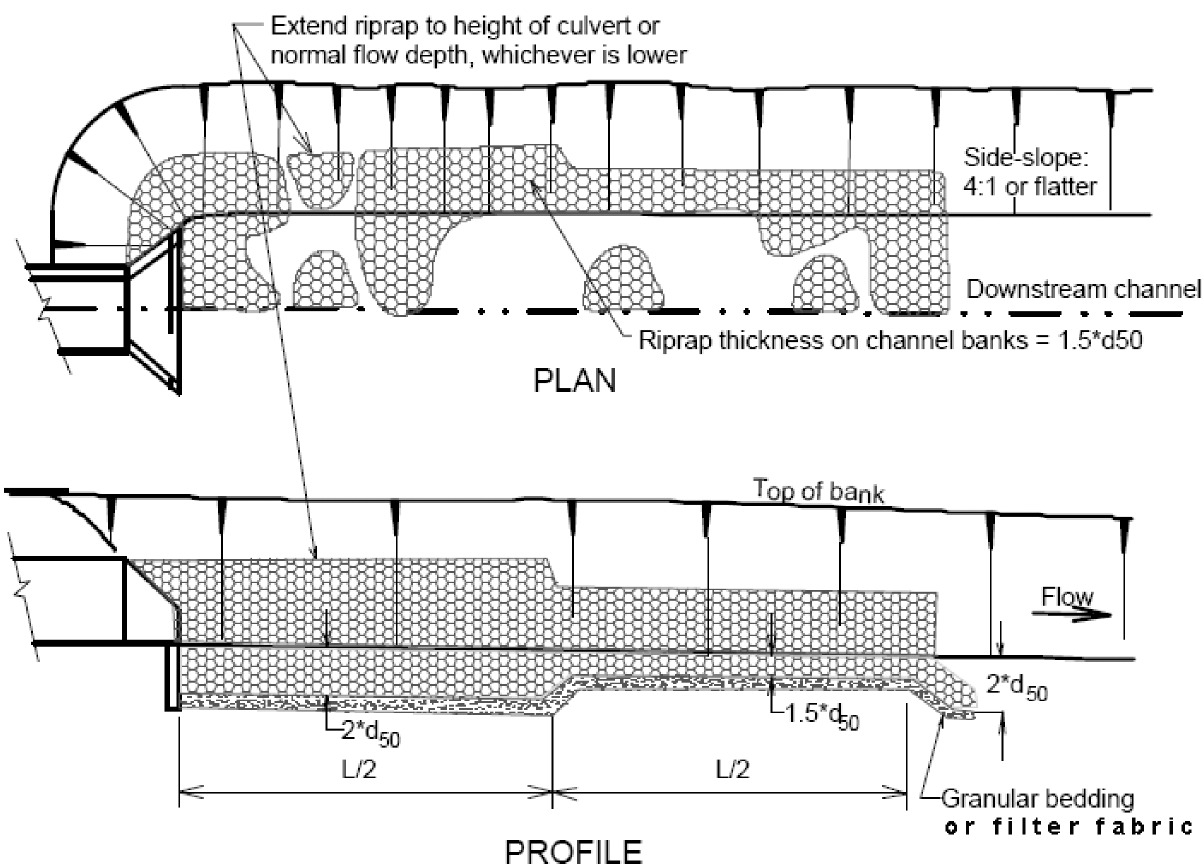
in which the maximum value of H_a shall not exceed H, and

where:

D_a = parameter to use in place of D_c in Figure 11.2 when flow is supercritical
D_c = diameter of circular culvert, in feet
H_a = parameter to use in place of H in Figure 11.3 when flow is supercritical
H = height of rectangular culvert, in feet
Y_n = normal depth of supercritical flow in the culvert

Figure 11.1 Culvert and pipe outlet riprap apron

(from UDFCD, 2001)



NOTES: 1. Headwall with wingwalls or flared end section required at all culvert outlets.
2. Cutoff wall required at end of wingwall aprons and end section.
Minimum depth of cutoff wall = 2*d50 or 3-feet, whichever is deeper.
3. Provide joint fasteners for flared end sections.

The designer should use Figure 11.2 and Figure 11.3 to estimate a riprap designation (Type VL, L, M, H, or VH) and corresponding d_{50}. The class of riprap to be specified is that which has a d_{50} greater than or equal to the required minimum size from Table 11.4. The designer should specify a gradation that is available in the area and in conformance with the gradation ranges specified in Table 11.5, which was derived from Table 12.4. Refer to the Riprap Quality and Riprap Layer Characteristics portions of Section 12.3.5.2 for required riprap quality specifications and characteristics. An underlying granular filter blanket, filter fabric, or combination of the two is required for riprap apron installations, in conformance with Chapter 8 of the Hydraulics Manual.

The riprap size may also be determined using the procedure built-in to the HY-8 computer program (FHWA, 2007). The apron configuration shall be designed in conformance with this section.

Table 11.4 Classification and gradation of ordinary riprap

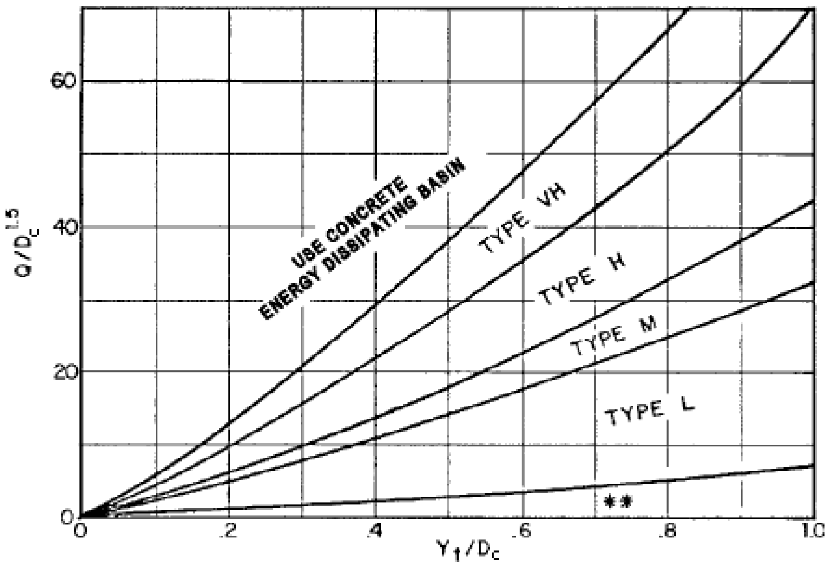
Riprap Classification	D_{50} (inches)
Type VL	6
Type L	9
Type M	12
Type H	18
Type VH	24

Table 11.5 Gradation ranges for ordinary riprap classifications

Percent of Gradation Smaller Than	Range of Stone Size				
	Riprap Classification				
	Type VL	Type L	Type M	Type H	Type VH
100	9.0-10.2	13.5-15.3	18.0-20.4	27.0-30.6	36.0-40.8
85	7.2-8.4	10.8-12.6	14.4-16.8	21.6-25.2	28.8-33.6
50	6.0-6.9	9.0-10.4	12.0-13.8	18.0-20.7	24.0-27.6
15	2.4-3.6	3.6-5.4	4.8-7.2	7.2-10.8	9.6-14.4

Figure 11.2 Riprap protection at circular conduit outlets

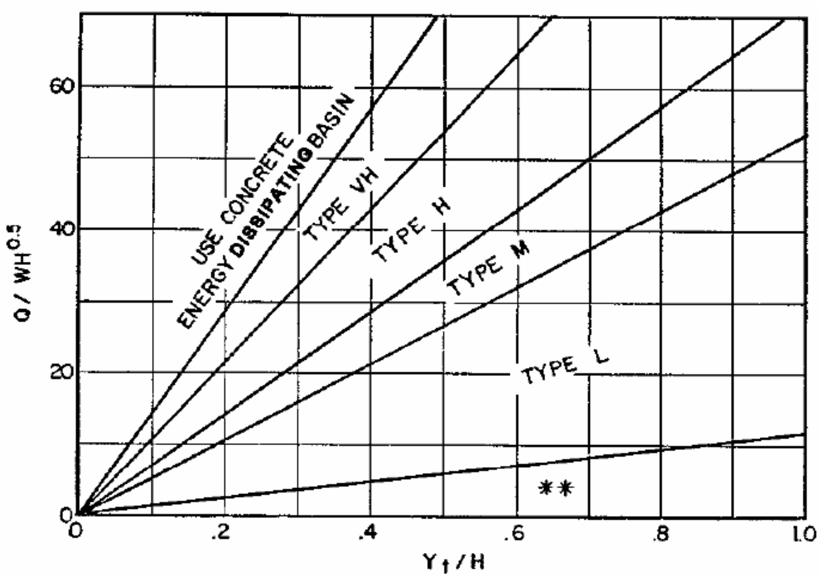
(from UDFCD, 2001)



Use D_a instead of D whenever flow is supercritical in the barrel.
** Use Type L for a distance of 3D downstream.

Figure 11.3 Riprap protection at rectangular conduit outlets

(from UDFCD, 2001)



Use H_a instead of H whenever culvert has supercritical flow in the barrel.
**Use Type L for a distance of 3H downstream.

Figure 11.4 Apron expansion factors for circular conduits

(from UDFCD, 2001)

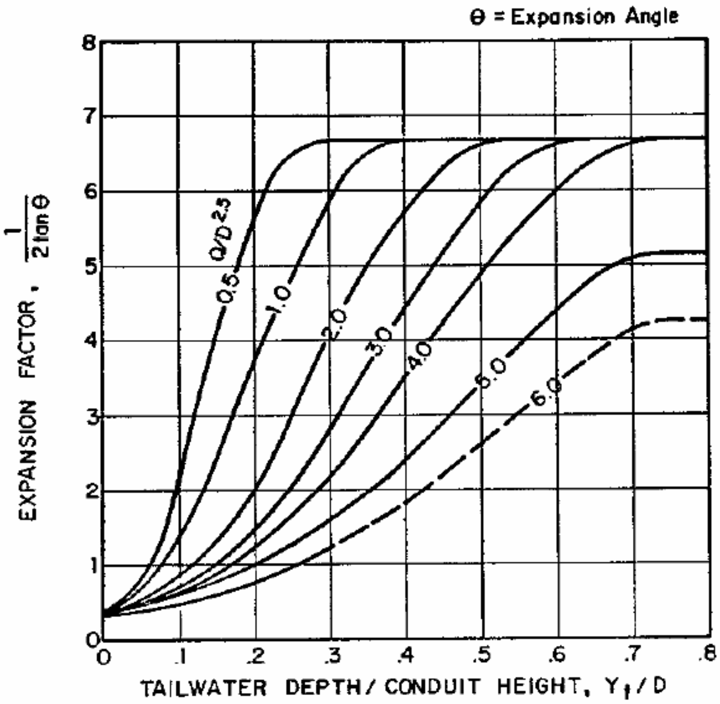
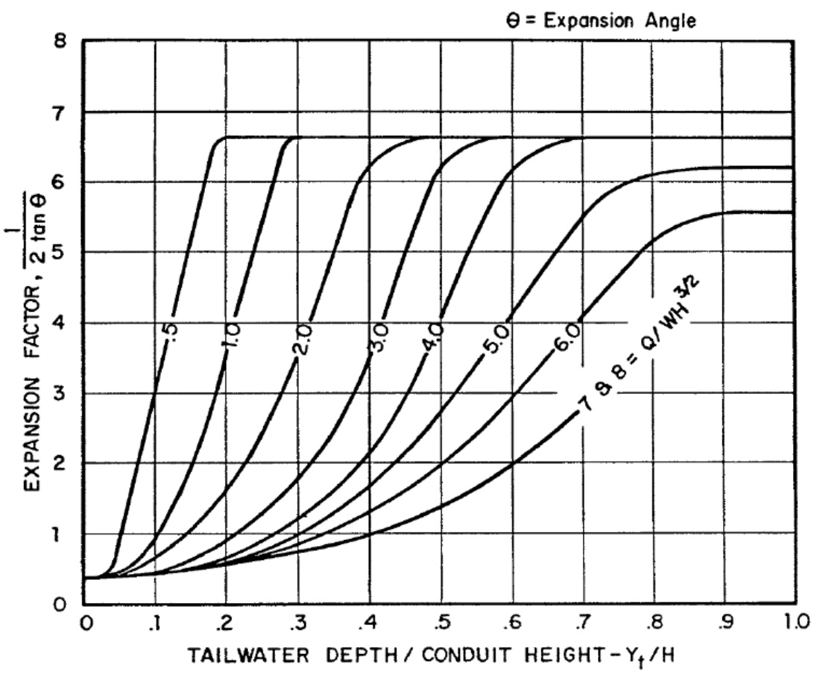


Figure 11.5 Apron expansion factors for rectangular conduits

(from UDFCD, 2001)



Extent of Protection. The length of the riprap protection downstream from the outlet depends on the degree of protection desired. If it is necessary to prevent all erosion, the riprap must be continued until the velocity has been reduced to an acceptable value. For purposes of outlet protection during major floods, the acceptable velocity is set at 3.5 ft/sec for very erosive soils and at 6.0 ft/sec for erosion resistant soils. The rate at which the velocity of a jet from a conduit outlet decreases is not well known. For the procedure recommended here, it is assumed to be related to the angle of lateral expansion, Θ, of the jet. The velocity is related to the expansion factor, ($1/(2\tan\Theta)$), which can be determined directly using Figure 11.4 or Figure 11.5, assuming that the expanding jet has a rectangular shape. The apron length (L), can then be computed using Equation 11.7:

$$L_p = \left(\frac{1}{2\tan\theta}\right)\left(\frac{A_t}{Y_t} - W\right) \qquad 11.7$$

where:

L_p = length of protection, in feet
W = width of the conduit, in feet (use diameter for circular conduits)
Y_t = tailwater depth, in feet
Θ = the expansion angle of the culvert flow

and:

$$A_t = \frac{Q}{V} \qquad 11.8$$

where:

Q = design discharge, in cfs
V = the allowable non-eroding velocity in the downstream channel, in ft/sec
A_t = required area of flow at allowable velocity, in ft²

In certain circumstances, Equation 11.7 may yield unreasonable results. Therefore, in no case should L_p be less than $3H$ or $3D_c$, nor does L_p need to be greater than $10H$ or $10D_c$ whenever the Froude parameter, $Q/WH^{1.5}$ or $Q/D_c^{2.5}$, is less than 8.0 or 6.0, respectively. Whenever the Froude parameter is greater than these maximums, increase the maximum L_p required by ¼ D_c or ¼ H for circular or rectangular culverts, respectively, for each whole number by which the Froude parameter is greater than 8.0 or 6.0, respectively.

Multiple Conduit Installations. The procedures outlined above can be used to design outlet erosion protection for multi-barrel culvert installations by hypothetically replacing the multiple barrels with a single hydraulically equivalent rectangular conduit. The dimensions of the equivalent conduit may be established as follows:

1. Distribute the total discharge, Q, among the individual conduits. Where all the conduits are hydraulically similar and identically situated, the flow can be assumed to be equally distributed; otherwise, the flow through each barrel must be computed.
2. Compute the Froude parameter $Q_i/D_{ci}^{2.5}$ (circular conduit) or $Q_i/W_iH_i^{1.5}$ (rectangular conduit), where the subscript i indicates the discharge and dimensions associated with an individual conduit.
3. If the installation includes dissimilar conduits, select the conduit with the largest value of the Froude parameter to determine the dimensions of the equivalent conduit.
4. Make the height of the equivalent conduit, H_{eq}, equal to the height, or diameter, of the selected individual conduit.
5. The width of the equivalent conduit, W_{eq}, is determined by equating the Froude parameter from the selected individual conduit with the Froude parameter associated with the equivalent conduit, $Q/W_{eq}H_{eq}$ and solving for W_{eq}.

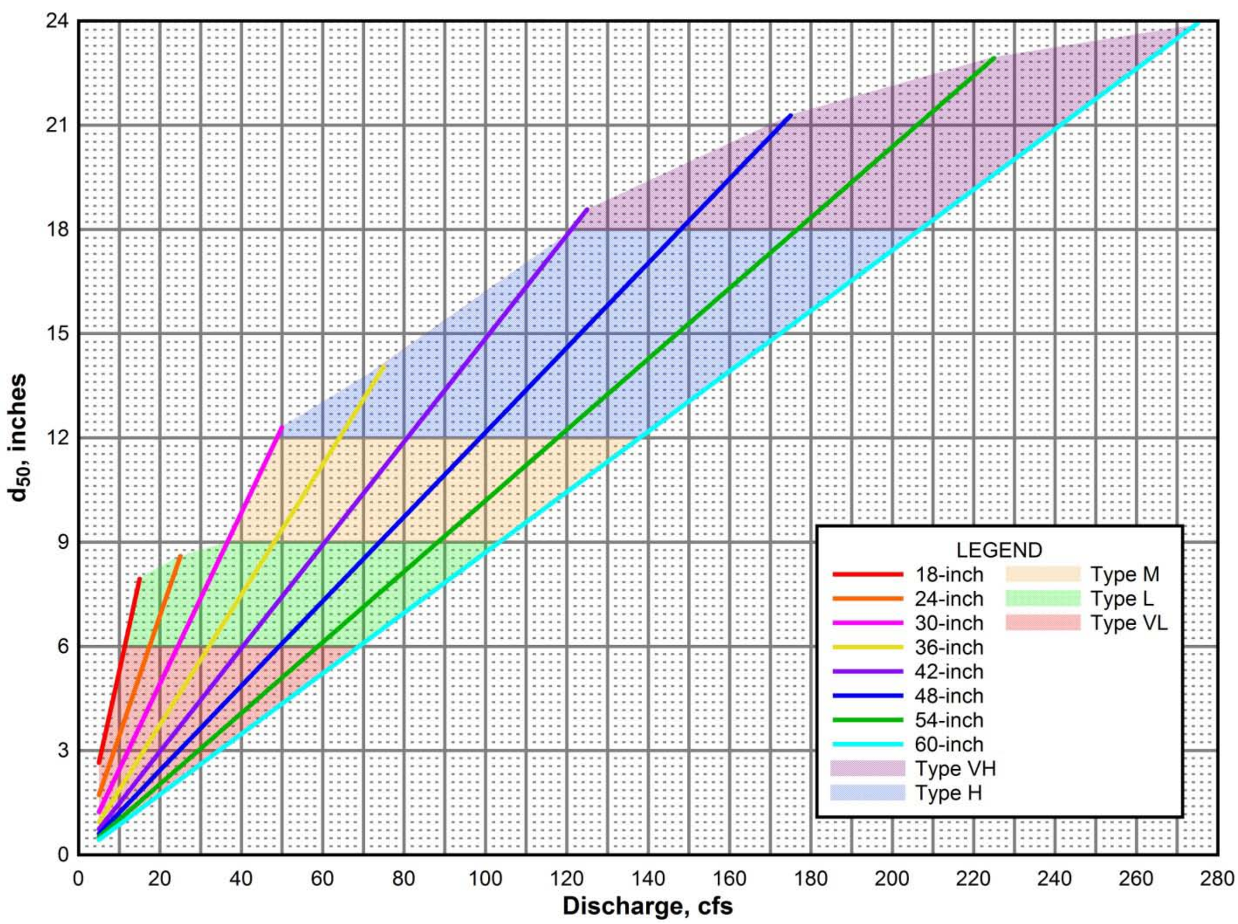
Simplified Riprap Apron Method

Determine the riprap size using Figure 11.6. Determine the apron length using Figure 11.7 or Figure 11.8. The simplified riprap apron method is based on the following:

1. Figure 11.6 is based on the following criteria:
   a. Pipe full flow velocity <= 15 fps.
   b. $Q/D_c^{2.5}$ <= 6.0
   c. $Y_t/D_c = 0.35$; where Y_t = tailwater depth in feet and D_c = culvert diameter in feet.
   d. A single pipe culvert installation. Refer to the Detailed Riprap Apron Method for riprap sizing for multiple conduit installations.
2. Figure 11.6 is derived using the methodology set forth in the Detailed Riprap Apron Method section, which is based on UDFCD (2001).
3. The riprap gradation class from Figure 11.6 may be used, within the limits specified, without detailed sizing computations. For most designs, this will be a conservative approach. If a more economical design is needed, the more detailed procedures defined in the Detailed Riprap Apron Method section shall be used to refine the sizing selection.
4. Where Type VL is acceptable per Figure 11.6, use Type L for a minimum length of 3 times the pipe diameter immediately downstream of the outlet, then use Type VL for the remainder of Length, L.
5. Culvert outlet riprap apron designs with parameters falling outside the color shaded limits shown on Figure 11.6 shall be designed using the Detailed Riprap Apron Method, or using the riprap basin approach from Section 8.4.3 of the Hydraulics Manual.

6. The length (L) on Figure 11.7 or Figure 11.8 is based on a limiting velocity at the end of the apron of 5 fps. If the soils are highly erosive, the Detailed Riprap Apron Method shall be used.

Figure 11.6 Riprap apron sizing chart for circular culvert outlets



The length (L) from Figure 11.1 of the riprap apron can be estimated using Figure 11.7 or Figure 11.8. These charts correspond with the riprap sizing chart shown on Figure 11.6. Riprap materials shall be in conformance with Section 12.3.5.2. Riprap gradations shall conform to Table 11.5. An underlying granular filter blanket, filter fabric, or combination of the two is required for riprap apron installations, in conformance with Chapter 8 of the Hydraulics Manual.

In the absence of a defined channel downstream from the culvert outlet, a trapezoidal apron configuration should be used. The minimum required expansion ratio of the apron sides (Ratio: width to length) is shown on Figure 11.9 and Figure 11.10.

Figure 11.7 Riprap apron length for circular pipes (18-inch - 36-inch)

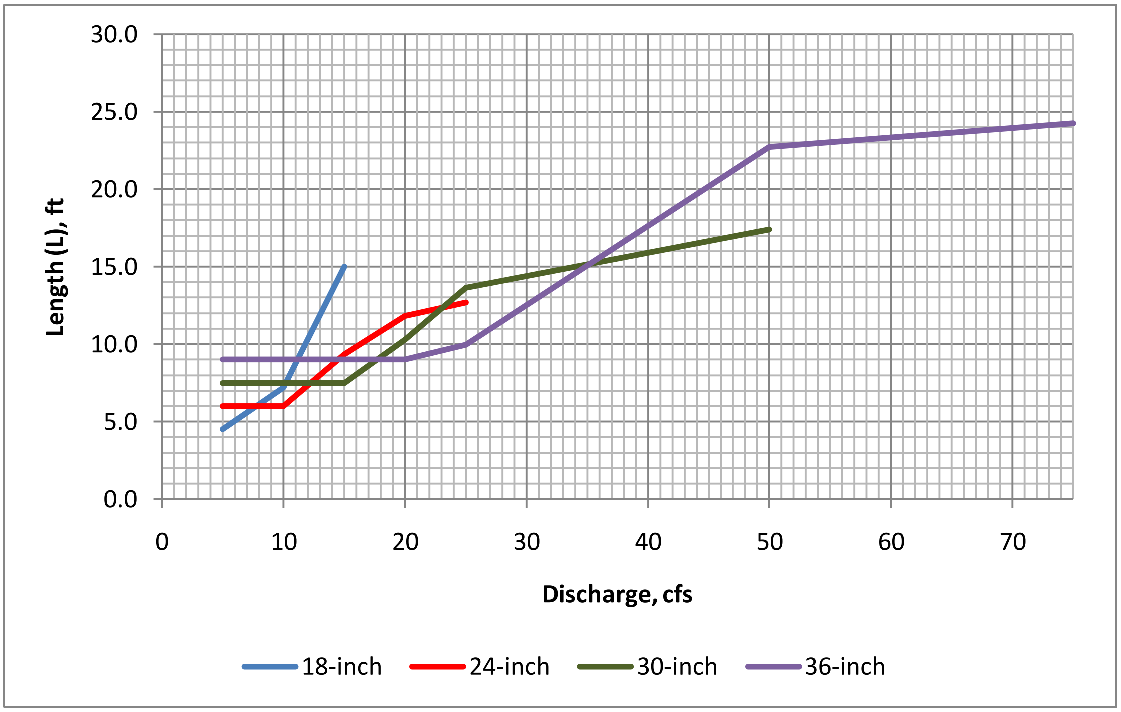


Figure 11.8 Riprap apron length for circular pipes (42-inch - 60-inch)

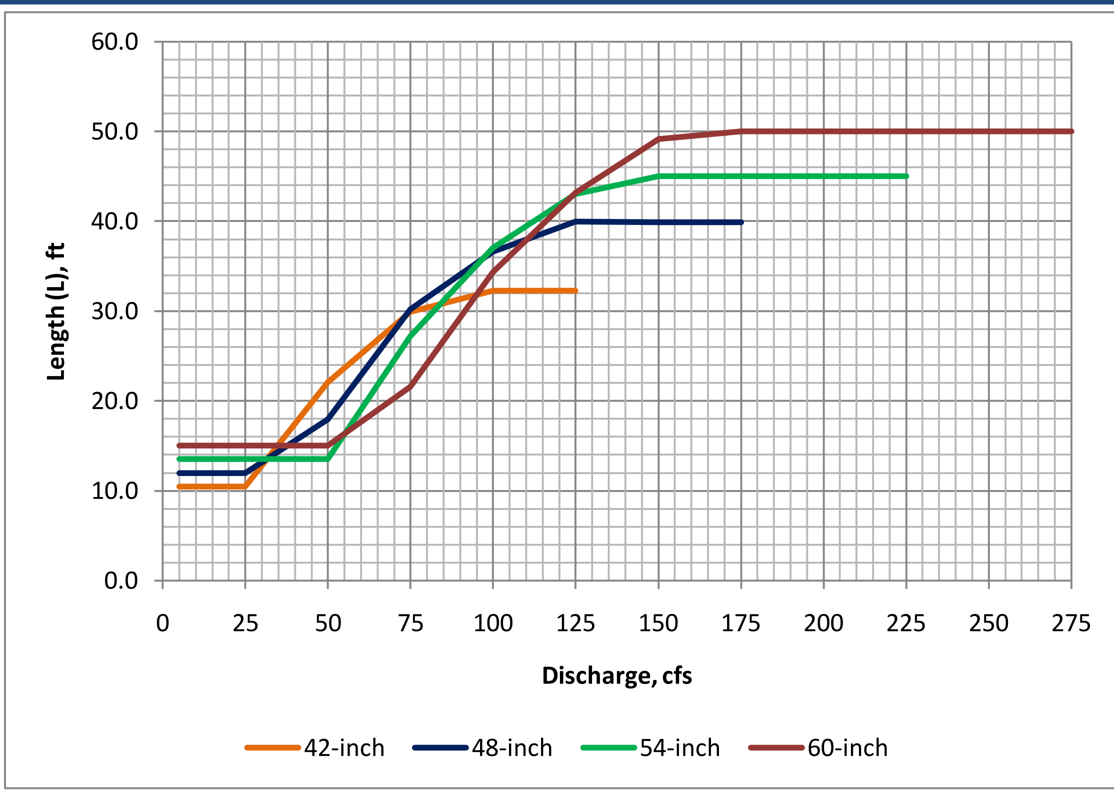


Figure 11.9 Riprap apron side expansion ratio (18-inch - 36-inch)

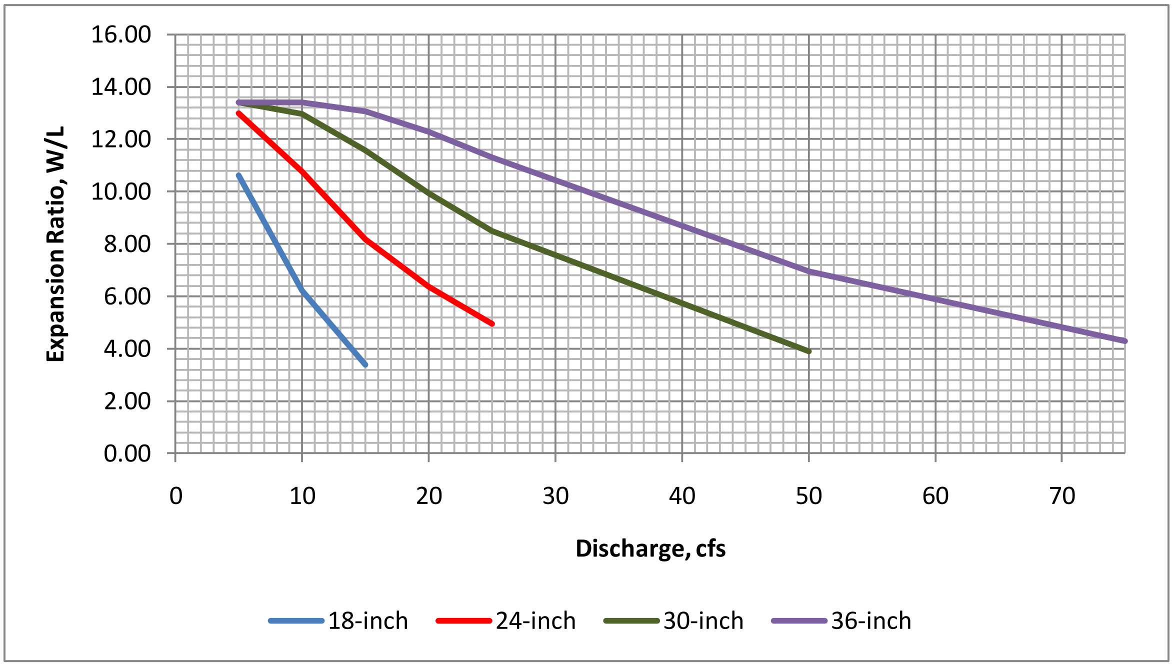
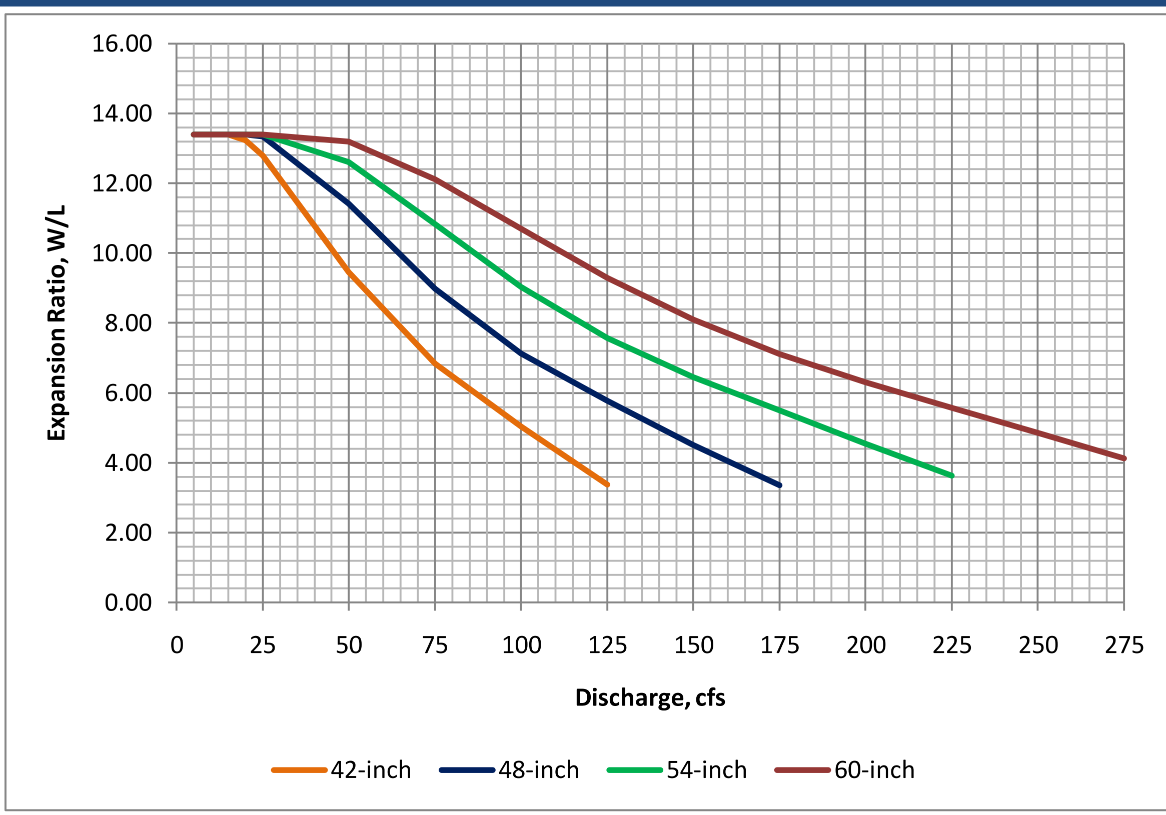


Figure 11.10 Riprap apron side expansion ratio (42-inch - 60-inch)



11.4.13 BRIDGE DESIGN EROSION REQUIREMENTS

If structural erosion protection is proposed, a comprehensive sediment transport analysis that assesses sediment transport in time and space (i.e. dynamic modeling consistent with 3 tier analyses identified in the Sedimentation chapter of the Hydraulics Manual) shall be undertaken to support the design and show that there are no adverse impacts to adjacent properties. The study may also be required to show that use of a similar design for other potential future crossings within limits of a study reach established by Mohave County do not result in cumulative adverse impacts within the study reach.

11.4.14 SUPERCRITICAL FLOW REQUIREMENTS

1. For channels functioning in a supercritical flow regime for the design discharge, there shall be no reduction in cross sectional area at bridges or culverts, or any obstructions (including bridge piers) in the flow path, up to the maximum practical span for the structure type as approved by COUNTY. For cases where bridge piers must be constructed because of maximum practical span considerations, piers shall be placed in the areas of lowest velocity whenever possible.

2. Bridge freeboard below the low chord elevation shall be the greater of 2 feet or the computed velocity head ($\frac{V^2}{64.4}$) using the channel velocity.

12 OPEN CHANNELS

12.1 INTRODUCTION

An open channel is a conveyance system in which water flows with a free surface at the water atmosphere interface. The channel may be either a natural watercourse or an artificial, "engineered" conveyance. Natural streams typically consist of a main flow channel and adjacent floodplains. Artificial channels are used for a wide variety of applications varying in scale from modest roadside ditches to large conveyance facilities that can be up to several hundred feet wide. This chapter provides technical standards for design of artificial channels.

12.2 HYDRAULIC TECHNICAL DESIGN CRITERIA

Technical guidance for design of open channels shall be in accordance with the open channels chapter of the Hydraulics Manual. Addition guidance may be obtained from FHWA (2005b). Specific design standards for Mohave County needed for application of the technical information contained in the Hydraulics Manual are defined herein.

12.3 OPEN CHANNELS STANDARDS

12.3.1 DESIGN WATER SURFACE ELEVATIONS

Design water surface elevations for excavated channels are to be below adjacent natural ground, including design freeboard.

12.3.2 MAXIMUM PERMISSIBLE VELOCITIES

Maximum permissible design velocities in open channels shall be governed by Table 12.1, Table 12.2, and Table 12.3.

Table 12.1 Maximum permissible velocities for unlined channels	
(from FHWA, 1961)	
Soils Type of Lining (Earth, No Vegetation)	**Maximum Permissible Velocity (1), ft/s**
Fine Sand (noncolloidal)	2.5
Sandy Loam (noncolloidal)	2.5
Silt Loam (noncolloidal)	3.0
Ordinary Firm Loam	3.5
Fine Gravel	5.0
Stiff Clay (very colloidal)	5.0
Graded, Loam to Cobbles (noncolloidal)	5.0
Graded, Silt to Cobbles (noncolloidal)	5.5
Alluvial Silts (noncolloidal)	3.5
Alluvial Silts (colloidal)	5.0
Coarse Gravel (noncolloidal)	6.0
Cobbles and Shingles	5.5
Shales and Hard Pans	6.0
(1) For channels multiply permissible velocity by:	0.95 for slightly sinuous; 0.90 for moderately sinuous; and 0.80 for highly sinuous.

Table 12.2 Maximum permissible velocities for grass-lined channels

Channels with Uniform Stand of Various Grass Cover and Well Maintained (1) (2) (adapted from FHWA 1961)

Cover	Maximum Permissible Velocity, fps
Bermuda Grass	6.0
Desert Salt Grass and Vine Mesquite	5.0
Lehman Lovegrass, Big Galleta, Purple Threeawn, Sand Dropseed	3.5

(1) Use velocities over 5 fps only where good covers and proper maintenance can be obtained.
(2) Grass is accepted only if an irrigation system is provided.

Table 12.3 Design criteria for artificial channels

Type of Channel Lining (1)	Maximum Side Slope, H:V (%)	Maximum Velocity, fps (2)
Structural Concrete	Vertical	15
Pneumatically Placed Concrete (3)	1.5:1 (67%) (7)	10
Soil Cement	2:1 (50%)	7(4)
Riprap	3:1 (33%)	9(5)
Grouted Riprap	2:1 (50%)	9(5)
Gabion Baskets	(6)	9(5)
Grass (irrigated & maintained)	4:1 (25%)	2.5 to 6.0
Earth	6:1 (16.7%)	2.5 to 6.0

Table 12.3 Design criteria for artificial channels

Type of Channel Lining [1]	Maximum Side Slope, H:V (%)	Maximum Velocity, fps [2]

(1) The values in this table are for channel sections with the same lining material for bottom and sides. For conditions where the bottoms and sides of the channels are different, the most critical applicable criteria are to be used.
(2) Maximum velocities listed for erodible linings are to be checked in each design to assure that erosion will not occur.
(3) Pneumatically Placed Concrete is allowed, but must be reinforced per a structural concrete design. Fiberglass reinforcement may be used with supporting design calculations.
(4) Higher velocities for soil cement lined channels/drop structures are acceptable upon submittal of a geotechnical analysis that assesses the suitability of the in-situ materials for soil cement applications and presents cement mixture specifications for the in-situ soils for the proposed maximum design velocities. The submittal shall be sealed and signed by a PE. Velocities greater than 15 fps are not recommended. Energy dissipaters may be required.
(5) Guideline only. Strict limits have not been set because this manual recommends that these channels be designed for subcritical flow.
(6) Per manufacturer's specifications.
(7) Channel side slope shall not exceed the soil natural angle of repose.
Note: The criteria listed in this table are boundary values. The designer is responsible for determining the adequacy of criteria for each specific application. For design of lining materials, analyses of soil conditions and subsurface drainage may be required.

12.3.3 CONSTRUCTION PLANS

Construction plans for open channel drainage improvements are to meet the requirements of Section 18.3.

12.3.4 ENCROACHMENT REQUIREMENTS IN FEMA FLOODPLAINS

All channelization and/or floodplain encroachments within FEMA mapped floodplains must be designed so that the cumulative effect of the encroachment does not raise the 100-year water surface (or energy grade line for supercritical flow) above the floodway water surface elevation, or more than 1 foot for FEMA mapped floodplains without a defined floodway. In addition, when determining encroachments of fill or other development, the "equal conveyance from both sides of channel" rule shall apply. The maximum 1 foot rise in water surface may not come from one side of the channel at the expense of the adjacent property owner. Encroachment and/or stabilization on one bank may result in increased erosion potential on the opposite bank. Such adverse effects shall be evaluated and mitigated as a part of the design.

In the event that the rise criteria will be exceeded and the construction of levees are proposed, the levees shall be designed and constructed in accordance with, and certified to meet, FEMA and DISTRICT criteria as a minimum. Although FEMA freeboard height criteria is the minimum standard, levee design freeboard shall be based on risk and uncertainty analysis methodology as established by the USACE. A government agency shall also agree in writing to maintain the levee system. COUNTY/DISTRICT strongly discourages the construction of levees for flood control purposes.

12.3.5 CHANNEL LINING REQUIREMENTS

12.3.5.1 Concrete Lined Channels

Concrete and pneumatically placed concrete lined channels shall be evaluated for the need for continuous reinforcement extending both longitudinally and transversely. Pneumatically placed concrete channels are to be designed to the same structural integrity as concrete channels.

All sloping and flat concrete, pneumatically placed concrete, and soil cement linings are preferred to have roughened surfaces (e.g. embedded rock, grooves, etc.) to discourage inappropriate recreational use.

The lining for channel bottoms that will require maintenance vehicle access must be designed for a minimum of 18 kip axle loads assuming one loading per week for the design life of the channel.

12.3.5.2 Riprap Lined Channels

Common riprap can be an effective lining material if properly designed and constructed. The choice of riprap usually depends on the availability of graded rock with suitable material properties and at a cost that is competitive with alternative lining systems. Riprap design involves the evaluation of five performance areas. These areas include the evaluation of:

1. riprap quality;
2. riprap layer characteristics;
3. hydraulic requirements;
4. site conditions; and
5. river conditions.

In Arizona, site requirements and river conditions are important factors in the protection of bridge structures and flood control channels.

Riprap Quality

Riprap quality determination refers to the physical characteristics of the rock particles that make up the bank protection. Qualities determined to be most important include density, durability, and shape. Requirements for each of these properties are summarized in this section.

Specific Gravity (Density). The design stone size for a channel depends on the particle weight, which is a function of the density or specific gravity of the rock material. A typical value of specific gravity in Mohave County is 2.4. All stones composing the riprap should have a specific gravity equal to or exceeding 2.4, following the standard test American Society for Testing and Materials (ASTM) C127.

Durability. Durability addresses the in-place performance of the individual rock particles, and also the transportation of riprap to the construction site. In-place deterioration of rock particles can occur due to cycles of freezing and thawing, or can occur during transportation to the site. The rock particles must have sufficient strength to withstand abrasive action without reducing the gradation below specified limits. Qualitatively, a stone that is hard, dense, and resistant to weathering and water action should be used. Rocks derived from igneous and metamorphic sources provide the most durable riprap. Laboratory tests should be conducted to document the quality of the rock. Specified tests that should be used to determine durability include: the durability index test and absorption test (see ASTM C127). Based on these tests, the durability absorption ratio (DAR) is computed as follows:

$$DAR = \frac{Durability\ Index}{Percent\ Absorption + 1} \qquad 12.1$$

The following specifications are used to accept or reject material:

1. DAR greater than 23, material is accepted;
2. DAR less than 10, material is rejected;
3. DAR 10 through 23:
    a. Durability index of 52 or greater, material is accepted; and,
    b. Durability index of 51 or less, material is rejected.

Shape. There are two basic shape criteria. First, the stones should be angular when not enclosed in wire-tied or gabion baskets. Angular stones with relatively flat faces will form a mass having an angle of internal friction greater than rounded stones, and therefore will be less susceptible to slope failures. Second, not more than 25 percent of the stones should have a length more than 2.5 times the breadth. The shape of the riprap stone should be cubical, rather than elongated. Cubical stones nest together, and are more resistant to movement. The length is the longest axis through the stone, and the breadth is the shortest axis perpendicular to the length. Angularity is a qualitative parameter which is assessed by visual inspection. No standard tests are used to evaluate this specification. If the engineer is faced with a supply of rounded river rock without a crusher to create angular rock, stone size should be increased 25% and side slopes decreased (USACE, 1994).

Riprap Layer Characteristics

The major characteristics of the riprap layer include: characteristic size; gradation; thickness; and filter-blanket requirements.

Characteristic Size - The characteristic size in a riprap gradation is the d_{50}. This size represents the average diameter of a rock particle for which 50 percent of the gradation is finer, by weight.

Gradation - To form an interlocked mass of stones, a range of stone sizes must be specified. The object is to obtain a dense, uniform mass of durable, angular stones with no apparent voids or pockets. The recommended maximum stone size is 2 times the d_{50} and the recommended minimum size is one-third of the d_{50}. The gradation coefficient, G, should equal 1.5.

$$G = 0.5(\mathrm{d}_{84}/\mathrm{d}_{50} + \mathrm{d}_{50}/\mathrm{d}_{16}) \qquad 12.2$$

Table 12.4 provides a recommended design gradation for riprap. As a practical matter, the designer should check with local quarries and suppliers regarding the classes and quality of riprap available near the site.

Table 12.4 Recommended riprap gradation limits from FHWA (1989)		
Stone Size Range, ft	Stone Weight Range, lb	Percent of Gradation Smaller Than
$1.5d_{50}$ to $1.7d_{50}$	$3.0W_{50}$ to $5.0W_{50}$	100
$1.2d_{50}$ to $1.4d_{50}$	$2.0W_{50}$ to $2.75W_{50}$	85
$1.0d_{50}$ to $1.15d_{50}$	$1.0W_{50}$ to $1.5W_{50}$	50
$0.4d_{50}$ to $0.6d_{50}$	$0.1W_{50}$ to $0.2W_{50}$	15

Thickness - The minimum thickness of riprap linings shall be the greater of d_{100} or 2.0 times d_{50} for hand-placed material, and 3.0 times d_{50} for dumped material in accordance with ADWR (7-1998). Refer to the Hydraulics Manual Chapter 11 for determining stone size requirements.

Where only the channel banks are lined, the riprap at the toe shall be designed to minimze the effect of scour as required in Section 6.5.7 of the Hydraulics Manual. At a minimum, the key-in width and depth shall be 3 feet, extending out from the toe of the channel bank and below the channel bottom.

Filter Blanket Requirements - Filter blankets, filter fabric, or a combination of the two, are required to underly riprap installations. Refer to the Hydraulics Manual Section 6.6.3 for guidance in the design of both granular and filter fabric types.

12.3.5.3 Hydraulic Stability Requirements

Due to safety concerns with excessively high velocities and the high potential for structural failure and unanticipated hydraulic jump locations, the recommended upper limit of Froude Number (F_r) shall be 2.0.

The limiting Froude Number for all types of channel linings designed for the subcritical flow regime shall be $F_r < 0.86$.

For concrete, soil cement, and pneumatically placed concrete lined channels designed to function in the supercritical flow regime, the additional range of $1.13<F_r<2.0$ is allowed, provided a sediment analysis is approved that substantiates that sediment loading will not change the flow regime from supercritical to subcritical. These linings shall be structurally designed to resist the hydrodynamic forces present during the design peak discharge with a safety factor of two (2.0).

At locations where there are to be planned hydraulic jumps, concrete, soil cement, and pneumatically placed concrete lined channels may pass through $0.86>F_r<1.13$.

A 100-year floodplain delineation based on subcritical conditions will be required if a channel designed to be supercritical may change flow regimes unpredictably due to sedimentation issues and flow will exceed the channel banks for the subcritical condition.

12.3.5.4 Curved Channel Radius Requirement

For channels with Froude Numbers less than 0.86, the ratio of the channel radius, r_c, (at the centerline) to the design width of the water surface shall be greater than 3.0.

For channels with Froude Numbers greater than or equal 1.2 and less than 2.0, the minimum radius of curvature should be computed using Equation 12.3.

$$r_{sc} = C\left(\frac{\mathrm{V}^2\mathrm{W}}{0.5\mathrm{g}}\right) \qquad 12.3$$

where:

r_{sc}	= minimum radius of channel centerline curvature in ft,
C	= coefficient,
V	= mean channel velocity in ft/s,
W	= channel width at elevation of centerline water surface in feet, and
g	= acceleration due to gravity in ft/s^2.

Equation 12.3 incorporates an allowance for superelevation and standing waves for supercritical flow in rectangular and trapezoidal channels with rigid lining and using a simple circular curve to define channel horizontal alignment transitions. For these conditions, use a value of C equal to one. This will limit the total superelevation to 0.5 feet. The normally determined freeboard requirements listed in Section 12.3.5.5 are adequate when used in conjunction with Equation 12.3. Equation 12.3 is derived from Section 2-5 of USACE (1994).

Curved channels should not be used when $0.86<F_r<1.2$. Extra care shall be taken in the design of bank protection on both the inside and outside of curves, using estimates of maximum velocity and considering eddies. The guidance in Chapters 8 and 11 of the Hydraulics Manual shall be carefully followed and applied. If a less conservative radius of curvature is needed, refer to guidance in Section 2-5 of USACE (1994).

12.3.5.5 Freeboard Requirements

Freeboard Equation. Required freeboard for both subcritical and supercritical flow conditions is computed according to the following formula:

$$FB = 0.25\left(\mathrm{Y} + \frac{\mathrm{V}^2}{2\mathrm{g}}\right) \qquad 12.4$$

where:

FB	= freeboard in feet,
Y	= depth of flow in feet,
V	= velocity of flow in ft/s; and
g	= acceleration due to gravity in ft/s^2.

Freeboard Requirements. The minimum freeboard value for straight rigid channels shall be 1 foot for subcritical and 2 feet for supercritical flows. The minimum freeboard value for curved rigid channels shall be 1.5 feet for subcritical and 2.5 feet for supercritical flows. If the minimum channel radius requirements of Section 12.3.5.4 are to be reduced as a part of a more detailed design, the freeboard requirements are to be added to the superelevated water surface elevation at channel bends for both subcritical and supercritical flow conditions, as computed using guidance in Sections 2-5 and 2-6 of USACE (1994). Using a smaller freeboard in specific cases requires prior approval by COUNTY. Freeboard exceeding the minimum standard is strongly recommended.

Levees. Although strongly discouraged, levees must meet FEMA and USACE design and freeboard requirements as a minimum. Freeboard design shall be based on USACE risk and uncertainty analysis methodology.

FEMA Floodplains. In all FEMA jurisdictional floodplains, the greater of the above equation or FEMA's freeboard requirement shall prevail for design use in open channels.

Freeboard Allowance for Vegetation. Every constructed channel that is capable of supporting vegetation growth is to be designed for an appropriate range of n-values in conjunction with an approved vegetation maintenance plan. The procedures in the Hydraulics Manual Chapter 7 and Chapter 13 herein shall be followed. The maintenance plan shall include an agreement, approved by the COUNTY/DISTRICT, for perpetual maintenance of the channel. If this is not feasible, then additional freeboard shall be required. For this case, standard freeboard requirements shall be added to the water surface elevation for the design storm hydraulics computed using the expected worst-case roughness condition assuming no on-going maintenance of vegetation.

12.3.6 MODELING HYDRAULIC FLOW SPLITS

12.3.6.1 General

Hydraulic flow splits are locations where a stream splits apart into two or more streams. Examples can include:

1. Distributary flow networks where streams can split and diverge and may or may not rejoin the main channel downstream.
2. Splits in rivers around an island or high ground.
3. Overtopping of a levee.
4. Overtopping of a watershed divide.
5. Diversion structures such as lateral weirs and spillways.
6. Street intersections where low flow patterns are dictated by the gutter but less frequent, higher discharges may be diverted into two or more streets downstream of the intersection.

12.3.6.2 Natural Channels

An example of a natural channel split, which could be a simple distributary flow split or a riverine split around an island, is shown on Figure 12.1. One-dimensional steady flow hydraulic modeling approaches can be used to estimate the percentage of peak discharge in each channel downstream of the junction. If multiple channels are involved, such as for distributary flow networks with unconfined flow, then two-dimensional modeling techniques are recommended. The computation process can be complicated; therefore, it is not recommended that hand computations be used. Instead, the US Army Corps of Engineers HEC-RAS computer program is recommended for situations that can be modeled using one-dimensional techniques. Using HEC-RAS, the optimization can be based on balancing energy or momentum. The energy-based method neglects the effects of the angle of the split. The momentum approach accounts for force effects due to the angle of the bifurcation. The following are the most common analysis situations for modeling flow splits:

- Energy-based, one-dimensional, steady state, subcritical
- Energy-based, one-dimensional, steady state, supercritical

- Momentum-based, one-dimensional, steady state, subcritical
- Momentum-based, one-dimensional, steady state, supercritical

HEC-RAS is also capable of applying a mixed-flow approach for each of the above. For most natural channel situations, the energy-based subcritical approach will be used. The following example describes the process for using HEC-RAS to compute flow split hydraulics using a trial and error procedure. HEC-RAS also has the capability of performing the entire optimization procedure automatically. Refer to USACE (2008b) pg 4-22 and USACE (2008a) Chapter 15.

The energy-based approach is generally applicable when flow is solidly subcritical (USACE, 2008b). Therefore, the energy-based approach should be applied where F_r<0.6 and/or the bifurcation angle is 45 degrees or smaller. For other conditions, the momentum based approaches should be used. The energy-based method should not be used for supercritical flow.

Energy-Based, One-Dimensional, Steady-State, Subcritical

A common natural channel flow split is a single channel diverging into two channels downstream where the flow regime is dominantly subcritical. The following procedure is from USACE (2008b), pgs 4-10 through 4-12. Key assumptions for this approach are:

- Normal depth computations are appropriate.
- Subcritical flow.
- The split flow boundaries are fixed and do not erode or change over time.

Cross sections used to define the split hydraulics should be assigned as close to the junction as is practical. For example, Cross Section A (CS A) on Figure 12.1 is the upstream cross section and contains the total flow entering the junction. CS B and CS C are cross sections assigned to the channels immediately downstream in each leg of the split. The computational procedure is a trial and error approach, involving determining the flow rate in each downstream cross section corresponding to the same energy.

1. Build an HEC-RAS model of the junction using the cross section layout approach shown in Figure 12.1, including additional cross sections upstream and downstream as appropriate based on engineering judgment.
2. Assume an initial flow split at the junction (i.e. Make an initial estimate of peak discharge in each downstream channel).

3. Compare the energy at CS B and CS C. If they differ by a significant magnitude (usually more than 1-2 percent), then the flow distribution is incorrect. Re-distribute the flow by putting more flow into the reach that had the lower energy and reducing the flow in the other.
4. Run HEC-RAS again and compare the energies. If the energy at CS B and CS C still differ significantly, then redistribute the flow again. Repeat this process until the energies converge.

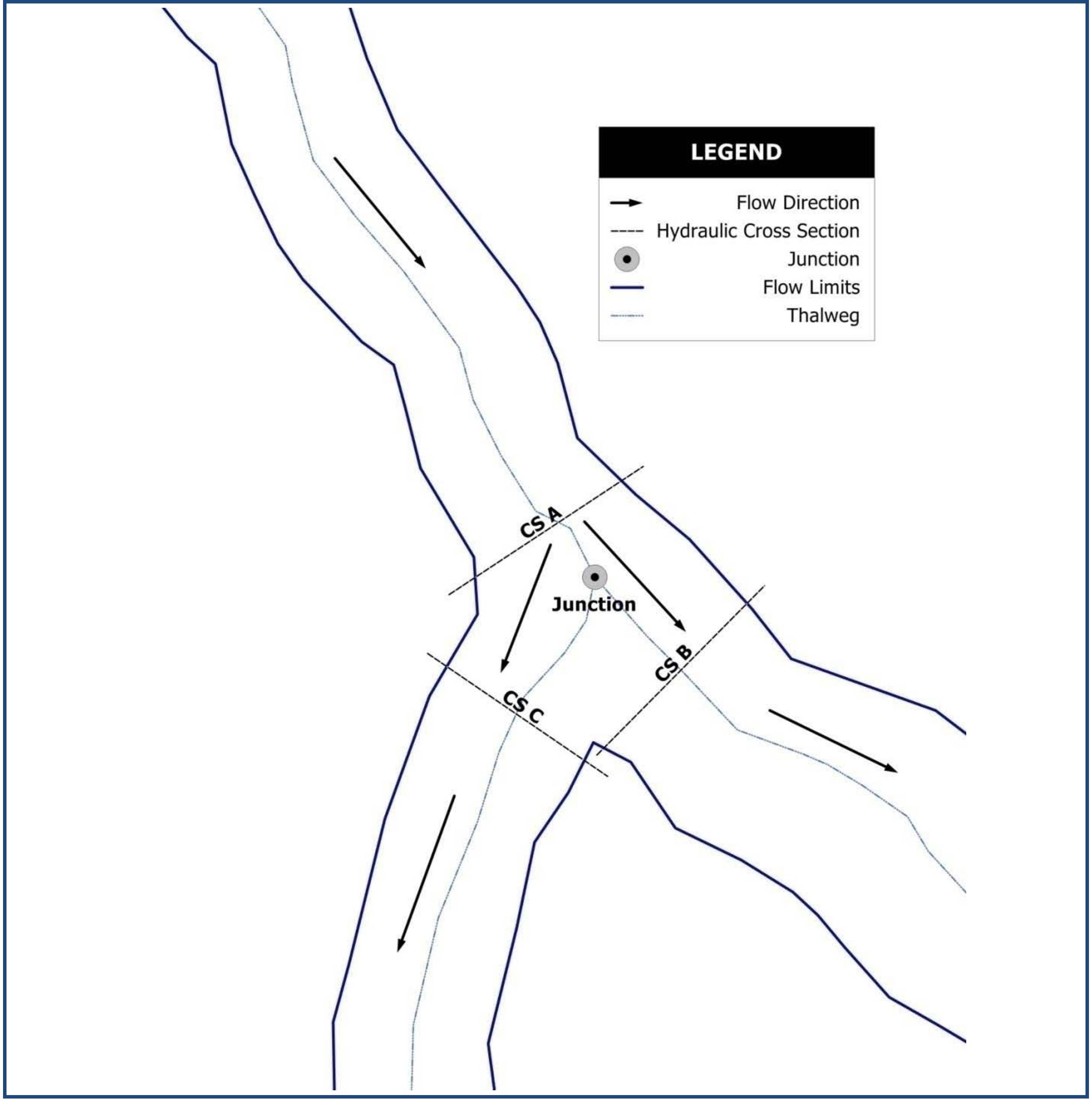
Energy-Based, One-Dimensional, Steady-State, Supercritical

Follow the procedure set forth in USACE (2008b), pg 4-13.

Momentum-Based, One-Dimensional, Steady-State, Subcritical and Supercritical

Follow the procedures set forth in USACE (2008b), pgs 4-15 through 4-19.

Figure 12.1 Hydraulic flow split schematic



12.3.6.3 Diversion Structures

Diversion structures, including lateral weirs and spillways, and even levees, should be modeled as described in USACE (2008a) Chapter 15 and USACE (2008b) Chapter 8.

12.3.6.4 Street Intersection

Conservative overdesign of street conveyance within the subdivision to handle the maximum offsite and onsite flows would eliminate the need to perform detailed flow split hydraulic Analyses. Conservative overdesign is to design the subdivision street cross section hydraulic carrying capacity assuming 100% of the upstream flow entering any intersection can flow in any downstream direction leaving the intersection.

For those cases where the downstream street cross section hydraulic carrying capacity is not designed to handle 100% of the upstream flow entering any intersection, HEC-RAS, or other pre-approved applicable program, shall be used. The momentum approach shall be used rather than the energy approach. For HEC-RAS, follow the procedures set forth in USACE (2008b) pgs 4-15 through 4-19.

12.3.7 IRRIGATION CANALS

Irrigation canals may not be used as an outfall for stormwater runoff without written approval by the agency that owns the facility.

12.3.8 MINIMUM EASEMENT WIDTH REQUIREMENT FOR CONSTRUCTED CHANNELS

A dedicated right-of-way, or privately owned drainage parcel shall be a minimum of the top width of an appropriately sized open channel plus 2 feet contiguous on both sides. For washes with a 100-year design peak discharge of 50 cfs or greater, and vehicular maintenance access is not provided within the channel bottom, add 16 feet of width to the easement on one side to cover a maintenance access road.

12.3.9 MINIMUM LANDSCAPE AND MAINTENANCE GUIDELINES

1. Landscaping and revegetation must not impede access for maintenance.
2. The vegetation must comply with the design intent of the channel in terms of conveyance and freeboard.
3. Landscaped channels must be designed using minimum and maximum expected n-values for the interval between maintenance operations, with minimum freeboard as specified above.

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13 FRICTION LOSSES IN OPEN CHANNELS AND PIPES

13.1 INTRODUCTION

The primary difficulty in using the Manning equation in practice is accurately estimating an appropriate value of the resistance coefficient, n. Selecting a value of *n* actually means estimating the resistance to flow in a channel or pipe, thus accounting for energy loss due to friction. Appropriate assignment of n-values requires an exercise in sound engineering judgment, due to the many intangibles.

This section provides guidance for selection of Manning’s n-values for use when modeling the hydraulics of natural or designed channels and when modeling or designing pipe culverts and storm drains. Manning’s n-values for these situations are provided for various conditions as a convenient reference for the engineer/hydrologist, and for the purposes of providing uniformity and reproducibility of assignment for hydraulic modeling in Mohave County. Assignment of n-values does require sound engineering judgment, so the source references for the values presented should be carefully studied and kept available for use when preparing models and designing drainage facilities. These references provide guidance in application that is not provided in this section. In addition technical guidance for estimation of friction losses shall be in accordance with the Friction Losses chapter of the Hydraulics Manual.

13.2 FRICTION LOSSES STANDARDS

1.1.1.1 Manning’s n-values for Natural Channels

The Manning's roughness coefficient, n, is a measure of the flow resistance of a channel or overbank flow area. The flow resistance is affected by many factors including size of bed material, bed form, cross section irregularities, flow depth, vegetation, channel alignment, channel shape, obstructions to flow, and quantity of sediment being transported in suspension or as bed load. In general, all factors that retard flow and increase turbulent mixing tend to increase n.

Per Phillips and Tadayon (2006), the *n* for a channel can be computed by:

$$n = (n_0 + n_1 + n_2 + n_3 + n_4)m_5 \qquad 13.1$$

where:

n_0 = the base value for a straight, uniform, stable channel.

n_1 = is a value for the effect of surface irregularities,

n_2 = is a value to account for obstructions to flow,

n_3 = is a value for vegetation effects,

n_4 = is a value to account for variations in channel cross section, and

m_5 = is a correction factor to account for meandering of the main channel.

The value for n_o can be selected from Table 13.1. The adjustment factors (n_1, n_2, n_3, n_4, and m_5) can be selected from Table 13.2. Extensive guidance for this method is provided in Phillips and Tadayon (2006), which is included in the Hydraulics Manual as Chapter 7. The engineer/hydrologist is encouraged to study this reference. Table 13.1 and Table 13.2 are based on the assumption of a stable channel bed. For unstable channels bottoms, Phillips and Tadayon (2006) should be consulted for guidance.

For overbank floodplains where agriculture is present, or where the above procedure may be impractical, the composite value of n may be selected from Table 13.3. Composite values of n for stable constructed channels may be selected from Table 13.4.

Table 13.1 Base values of Manning's n for channels considered stable (from Phillips and Tadayon, 2006)

Channel Material	Size of Bed Material		Base Values, n_0	
	Millimeters	Inches	Benson and Dalrymple (1967)	Chow (1959)
(1)	(2)	(3)	(4)	(5)
Firm earth	—	—	0.025–0.032	0.020
Coarse sand	1–2	—	0.026–0.035	—
Fine gravel	—	—	—	0.024
Gravel	2–64	0.08–2.5	0.028–0.035	—
Coarse gravel	—	—	—	0.028
Cobble	64–256	2.5–10.5	0.030–0.050	—
Boulder	> 256	> 10	0.040–0.070	—

Table 13.2 Adjustment factors used to determine Manning's *n* values (from Phillips and Tadayon, 2006)

Channel Conditions	Manning's *n* Adjustment	Example
***n1*, Degree of irregularity (0.000-0.020)**		
Smooth	0.000	Smoothest channel attainable in a given bed material.
Minor	0.001–0.005	Channels with slightly scoured or eroded side slopes.
Moderate	0.006–0.010	Channels with moderately sloughed or eroded side slopes.
Severe	0.011–0.020	Channels with badly sloughed banks; unshaped, jagged, and irregular surfaces of channels in rock.
***n2*, Variation in channel cross section (0.000-0.015)**		
Gradual	0.000	Size and shape of channel cross sections change gradually.
Alternating occasionally	0.001–0.005	Large and small cross sections alternate occasionally, or the main flow occasionally shifts from side to side owing to changes in cross-section shape.
Alternating frequently	0.010–0.015	Large and small cross sections alternate frequently, or the main flow frequently shifts from side to side owing to changes in cross-section shape.
***n3*, Effect of obstructions (0.000-0.060)**		
Negligible	0.000–0.004	A few scattered obstructions, which include debris deposits, stumps, exposed roots, logs, piers, or isolated boulders, which occupy less than 5 percent of the channel.
Minor	0.005–0.015	Obstructions occupy from 5 to 15 percent of the cross-section area and spacing between obstructions is such that the sphere of influence around one obstruction does not extend to the sphere of influence around another obstruction. Smaller adjustments are used for curved, smooth-surfaced objects than are used for sharp-edged, angular objects.
Appreciable	0.020–0.030	Obstructions occupy from 15 to 50 percent of the cross-section area, or the space between obstructions is small enough to cause the effects of severe obstructions to be additive, thereby blocking an equivalent part of a cross section.

Table 13.2 Adjustment factors used to determine Manning's *n* values (from Phillips and Tadayon, 2006)

Channel Conditions	Manning's *n* Adjustment	Example
Severe	0.040–0.060	Obstructions occupy more than 50 percent of the cross-section area, or the space between obstructions is small enough to cause turbulence across most of the cross section.
***n4*, Amount of vegetation (0.000-0.200)**		
Negligible	0.000–0.002	Grass, shrubs, or weeds were permanently laid over during flow.
Small	0.002–0.010	Dense growths of flexible turf grass, such as Bermuda, or weeds growing where the average depth of flow is at least two times the height of the vegetation where the vegetation is not laid over. Trees, such as willow, cottonwood, or salt cedar, growing where the average depth of flow is at least three times the height of the vegetation. Flow depth is about two times the tree height, and the trees are laid over.
Medium	0.010–0.025	Moderately dense grass, weeds, or tree seedlings growing where the average depth of flow is from two to three times the height of vegetation; brushy, moderately dense vegetation, similar to 1- to 2-year-old willow trees growing along the banks. A few 8 to 10-year old willow, cottonwood, mesquite, or palo verde, which blocks flow by approximately 1 to 10 percent, and spheres of influence or turbulence do not overlap.
Large	0.025–0.050	8- to 10-year-old willow, cottonwood, mesquite or palo verde trees (block flow by approximately 10 to 30 percent where the spheres of influence overlap) intergrown with some weeds and brush where the hydraulic radius exceeds 2 feet.
Very large	0.050–0.100	Bushy willow trees about 1-year old intergrown with weeds alongside slopes or dense cattails growing along the channel bottom; trees intergrown with weeds and brush. Moderately dense (blocks flow by approximately 30 to 50 percent and the spheres of influence overlap) 8- to 10-year old trees spaced randomly throughout channel where depth of flow approximates height of vegetation.

Table 13.2 Adjustment factors used to determine Manning's *n* values (from Phillips and Tadayon, 2006)

Channel Conditions	Manning's *n* Adjustment	Example
Extremely large	0.100–0.200	Mature (greater than 10 years old) willow trees and tamarisk intergrown with brush and blocking flow by more than 70 percent of the flow area, causing turbulence across most of the section. Depth of flow is less than average height of the vegetation. Dense stands of palo verde or mesquite that block flow by 70 percent or more and hydraulic radius is about equal to or greater than average height of vegetation.
***m5*, Degree of meandering (1.00-1.30)**		
Minor	1.00	Ratio of the channel length to valley length is 1.0 to 1.2.
Appreciable	1.15	Ratio of the channel length to valley length is 1.2 to 1.5.
Severe	1.30	Ratio of the channel length to valley length is greater than 1.5.

Table 13.3 Values of Manning's *n* for agriculture or overbank areas

(from Phillips and Tadayon, 2006)

Description	Manning's n		
	Minimum	Normal	Maximum
Pasture, no brush			
Short grass	0.025	0.030	0.035
High grass	0.030	0.035	0.050
Cultivated areas			
No crop	0.020	0.030	0.040
Mature row crops	0.025	0.035	0.045
Mature field crops	0.030	0.040	0.050
Shrubs			
Scattered shrubs, heavy weeds	0.035	0.050	0.070
Light shrubs and trees, in winter	0.035	0.050	0.060
Light shrubs and trees, in summer	0.040	0.060	0.080
Medium to dense shrubs, in winter	0.045	0.070	0.110
Medium to dense shrubs, in summer	0.070	0.100	0.160
Trees			
Dense willows, mesquite, saltcedar	0.110	0.150	0.200
Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
Same as above, but heavy growth of sprouts	0.050	0.060	0.080
Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
Same as above, but with flood stage reaching branches	0.100	0.120	0.160

13.2.1.2 Manning's n-values for Constructed Channels

Table 13.4 Composite values of *n* for stable constructed channels

[excerpt from: Simon, Li and Associates (1981). Adapted from: Chow (1959), Aldridge and Garret (1973), FHWA (2005b), and Phillips and Tadayon (2006)]

Description	Manning's n		
	Minimum	Normal	Maximum
A. Lined or built-up channels			
a. Concrete			
1. Finished	0.011	0.013	0.015
2. Float finish	0.013	0.015	0.016
3. Unfinished	0.014	0.017	0.020
4. Shotcrete, good section	0.016	0.019	0.023
5. Shotcrete, wavy section	0.018	0.022	0.025
b. Soil cement	0.018	0.020	0.025
c. Gravel mulch (1-inch, flow depth 0.5-3.3 ft)	0.031	0.033	0.040
d. Gravel mulch (2-inch, flow depth 0.5-3.3 ft)	0.038	0.042	0.056
e. Cobble (2.5"<=d_{50}<=5", flow depth <= 2 ft)	0.040	0.045	0.049
f. Riprap (5"<=d_{50}<=22", flow depth <= 1-6 ft)	0.040	0.070	0.100
g. Grouted riprap	0.028	0.030	0.040
h. Gabions	Same as for cobble and riprap linings		
i. Gravel bottom with sides of			
1. Formed concrete	0.017	0.020	0.025
2. Random stone in mortar	0.020	0.023	0.026
3. Dry rubble or riprap	0.023	0.033	0.036
B. Evacuated or dredged channels			
a. Earth, straight and uniform			
1. Clean, after weathering	0.018	0.022	0.025
2. Gravel, uniform section, clean	0.022	0.025	0.033
b. Earth, winding and sluggish			
1. Earth bottom and rubble sides	0.028	0.030	0.035
2. Stony bottom	0.025	0.035	0.040
3. Cobble bottom and clean sides	0.030	0.040	0.050
c. Rock cuts			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050

13.2.1.3 Manning's n-values for conduits

Manning's n-values for hydraulic conveyance structures such as culverts and storm drains are listed in Table 13.5 for concrete pipes and box culverts, and in Table 13.6 for corrugated steel pipes and pipe arches. For guidance in proper application of these tables, refer to Chow (1959), AISI (1980), AISI (2007), Mays (1999), and Mays (2001). The publications of the American Concrete Institute should also be reviewed.

Table 13.5 Manning's *n* for concrete pipe flowing partly full

(from Chow, 1959)

Description	Manning's n		
	Minimum	Normal	Maximum
Culvert, straight and free of debris	0.010	0.011	0.013
Culvert with bends, connections, and some debris	0.011	0.013	0.014
Finished	0.011	0.012	0.014
Sewer with manholes, inlet, etc., straight	0.013	0.015	0.017
Unfinished, steel form	0.012	0.013	0.014
Unfinished, smooth wood form	0.012	0.014	0.016
Unfinished, rough wood form	0.015	0.017	0.020

Table 13.6 Values of *n* for corrugated steel pipe

[from AISI (1980)]

Corrugations	Annular	Helical														
	2⅔ in	1½ x ¼ in.	2⅔ x ½ in.													
Flowing:	**Diameters**	**8 in.**	**10 in.**	**12 in.**	**15 in.**	**18 in.**	**24 in.**	**30 in.**	**36 in.**	**42 in.**	**48 in.**	**54 in. and larger**				
Full Unpaved	0.024	0.012	0.014	0.011	0.012	0.013	0.015	0.017	0.018	0.019	0.020	0.021				
Full 25% Paved	0.021						0.014	0.016	0.017	0.018	0.020	0.019				
Part Full Unpaved	0.027			0.012	0.013	0.015	0.017	0.019	0.020	0.021	0.022	0.023				
Flowing:	**Pipe-Arch**				**17 x 13**	**21 x 15**	**28 x 20**	**35 x 24**	**42 x 29**	**49 x 33**	**57 x 38**	**64 x 43 and larger**				
Full Unpaved	0.026				0.013	0.014	0.016	0.018	0.019	0.020	0.021	0.022				
Part Full	0.029				0.018	0.019	0.021	0.023	0.024	0.025	0.025	0.026				
	Annular	**Helical**														
	3 x 1 in.	**3 x 1 in.**														
Flowing:									**36 in.**	**42 in.**	**48 in.**	**54 in.**	**60 in.**	**66 in.**	**72 in.**	**78 in. and larger**
Full Unpaved	0.027								0.022	0.022	0.023	0.023	0.024	0.025	0.026	0.027
25% Paved	0.023								0.019	0.019	0.020	0.020	0.021	0.022	0.022	0.023
	Annular	**Helical**														
	5 x 1 in.	**5 x 1 in.**														
Flowing:											**48 in.**	**54 in.**	**60 in.**	**66 in.**	**72 in.**	**78 in. and larger**
Full Unpaved	0.025										0.022	0.022	0.023	0.024	0.024	0.025
25% Paved	0.022										0.019	0.019	0.020	0.021	0.021	0.022
All pipe with		All Diameters														
smooth interior*		0.012														

* Includes full paved, concrete lined, spiral rib and double wall pipe. Per deGrout and Boyd (1983) modified for lower values of n for helical pipe.

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14 HYDRAULIC STRUCTURES

14.1 INTRODUCTION

Hydraulic structures are used in drainage and flood control works to control water flow characteristics such as velocity, direction and depth. Structures may also be used to control the elevation and slope of a channel bed, as well as the general configuration, stability and maintainability of the waterway. The use of hydraulic structures can increase the capital cost of drainage facilities while lowering O&M costs. The use of hydraulic structures should be limited by careful and thorough hydraulic engineering practices to locations and functions justified by prudent planning and design. On the other hand, use of hydraulic structures can reduce initial and future maintenance costs by changing the characteristics of the flow to fit the project needs, and by reducing the size and cost of related facilities.

Hydraulic structures include channel drop structures, spillways, grade control structures, energy dissipaters, bridges, transitions, chutes, bends and many other specific drainage works. This chapter is oriented toward control structures for drainage channels, outlets for storm drains and culverts, and spillways for non-jurisdictional dams.

14.2 HYDRAULIC TECHNICAL DESIGN CRITERIA

Technical guidance for design of hydraulic structures shall be in accordance with the hydraulic structures chapter of the Hydraulics Manual. Specific design standards for hydraulic structures needed for application of the technical information contained in the Hydraulics Manual are defined herein.

14.3 HYDRAULIC STRUCTURES STANDARDS

The following standards shall be utilized in the design of hydraulic structures:

14.3.1 TRASH RACK CLOGGING FACTOR

A clogging factor of 50 percent of the rack area shall be used in the hydraulic analysis of all trash racks.

14.3.2 SIPHONS

The use of siphons for stormwater conveyance is strongly discouraged. A siphon may be allowed provided it is demonstrated there is no other feasible option and adequate provisions for on-going maintenance are in-place. Approval by the MOHAVE COUNTY ENGINEER is required for the use of siphons.

14.3.3 DROP STRUCTURES

At drop structures, a hydraulic jump analysis shall be conducted for a range of flows, since flow characteristics at the drop may vary with discharge. This analysis is to be used to support the design of the structure and erosion control measures.

Due to a high failure rate and excessive maintenance costs, drop structures having loose riprap on a sloping face are not permitted.

Open channels are recommended in lieu of pipes for conveyance of low flows through drop structures. Pipes, if approved by COUNTY/DISTRICT for conveying low flows through drop structures, shall be no smaller than 24 inches in diameter.

14.3.4 CONSTRUCTION PLANS

Construction plans for hydraulic structure drainage improvements are to meet the requirements of Section 18.3.

15 STORMWATER STORAGE

15.1 INTRODUCTION

Land development can convert natural pervious areas into impervious or otherwise altered surfaces. These activities may cause an increase in runoff volume and/or peak discharge. The temporary storage of stormwater runoff can decrease downstream peak discharges and associated impacts to drainage infrastructure. Two types of stormwater storage approaches are considered for use in Mohave County. The primary method, which shall be used as the standard method, is stormwater retention. Stormwater retention is a basin or reservoir where water is stored for regulating a flood; however, it does not have significant gravity-flow outlets for discharging stored runoff. The stored water is disposed by other means such as infiltration into the soil, evaporation, injection (or dry) wells, low flow outlets, or pumping systems. The low flow outlets have a relatively constant discharge rate under ponded conditions (much less than existing peak discharges) and are intended to drain the basin between 24 and 36 hours after the storm ends. The design intent for retention basins is to capture the runoff volume for the design storm frequency and duration.

A detention basin uses gravity-flow outlets for discharging the stored runoff. Detention facilities do not reduce the volume of runoff, they do however lengthen the time flow will be present in the watercourse downstream of the facility. Due to the longer duration of flow downstream of detention basins, their use requires greater analysis to verify that peak discharges are not increased downstream. Care must be taken not to size the outlet too large, and a range of flood frequency events shall be considered in the analysis. The design intent for the outlet is for post development peak outflows to be equal or less than pre-development flows for the design storm event(s).

The following are COUNTY/DISTRICT policies related to stormwater storage:

15.2 HYDRAULIC TECHNICAL DESIGN CRITERIA

Technical guidance for design of stormwater storage facilities shall be in accordance with the stormwater storage chapter of the Hydraulics Manual. Specific design policies and standards for Mohave County needed for application of the technical information contained in the Hydraulics Manual are defined herein.

15.3 STORMWATER STORAGE POLICIES

15.3.1 STORMWATER RETENTION FOR DEVELOPMENTS

All development (residential and non-residential subdivisions, and single non-residential parcels) shall make provisions to retain stormwater runoff falling within its boundaries.

15.3.2 ON-LOT STORAGE

On-lot retention is permitted (but not encouraged) only if the lots are greater than or equal to one (1) acre in gross area. On-lot storage is not allowed for residential subdivisions with a lot size less than one gross acre without a variance approved in writing by the Board of Supervisors.

15.3.3 MULTI-USE FEATURES

The designers of stormwater storage areas in residential subdivisions are encouraged to incorporate multi-use features and to design the basin grading with varying side slopes/land features that are aesthetically pleasing while accommodating safety features. Aesthetics as well as functionality are to be considered in the design of stormwater storage and conveyance facilities.

Siting recreational facilities, particularly playground equipment for children, at the very bottom of stormwater storage basins is to be avoided. It is recommended these basins be designed with tiers or gentle slopes to allow for the collection of nuisance water and conveyance around fields and play areas to keep them safe from inundation during the more frequent rainfall events, such as the one- or two-year storm.

15.3.4 DRAINAGE OF STORAGE FACILITIES

Storage facilities shall be designed to drain in accordance with the procedures in the Hydraulics Manual and Section 15.4.1.4. All stormwater storage facilities shall be designed to drain to appropriate outfall facilities.

15.3.5 UNDERGROUND STORAGE FACILITIES

Underground storage facilities are allowed but not encouraged. Such facilities must be designed in accordance with Section 15.4.1.

15.4 STORMWATER STORAGE STANDARDS

15.4.1 STORMWATER RETENTION

15.4.1.1 Minimum Design Storm

All new developments, regardless of lot size, shall make provisions to retain the entire stormwater runoff from a 100-year, 2-hour duration storm falling within its boundaries.

15.4.1.2 Guidelines for Retention within Parking Lots

The maximum depth of ponded water within any parking lot location is recommended to be 1 foot. Parking lot retention areas should not be adjacent to buildings and not be sited in travel lanes. No more than 25% of the parking lot area should be used for stormwater storage. The minimum longitudinal slope recommended within parking lot storage facilities is 0.005 ft/ft, unless concrete valley gutters are provided. With concrete valley gutters, a minimum longitudinal slope of 0.002 ft/ft is recommended.

15.4.1.3 Underground Storage

Underground storage is allowed but not encouraged. It shall meet the drain time requirements.

15.4.1.4 Drain Time

All stormwater storage facilities shall be designed so that the stored runoff is emptied completely from the facility within 36 hours after the runoff event has ended. The drainage system for retention facilities shall be accomplished using one of the following methods, listed in order of preference

1. Percolation.
2. Dry wells or a combination of percolation and dry wells.
3. Gravity bleed-off to the existing surface drainage system.
4. Pumping to an approved facility.

Percolation. Procedures used to determine the design percolation rate shall be one of the following two methods, listed by order of preference:

<u>Method 1</u>. ASTM D 3385-03, Double Ring Infiltrometer. If the soils present are outside the accepted range for application of ASTM D 3385-03, then method 2 shall be applied. Soils

outside the acceptable range for ASTM D3385-03 are typically very pervious or very impervious with a saturated hydraulic conductivity greater than about 14 inches/hour or less than about 0.0014 inches/hour. Very impervious soils that are outside the range of applicability for ASTM D3385-03 are not suitable for stormwater percolation disposal system applications. Dry wells may be a better choice for these conditions. If there is a question regarding the applicability of this method for the soils at a particular site, ASTM D 3385-03 should be applied and the results checked against the acceptable range of values of hydrologic conductivity. ASTM D 3385-03 may also not be applicable for dry or stiff soils that will fracture when the rings are installed, or gravels that do not allow penetration by the rings.

Method 2. EPA Falling Head Percolation Test Procedure from Design Manual - Onsite Wastewater Treatment and Disposal Systems (EPA, 1980). An adaptation of this procedure is outlined in Table 15.1.

Table 15.1 Falling head percolation test procedure

Step	Description
Number and Location of Tests	A minimum of two percolation tests shall be performed within the area proposed for an absorption system. Test holes are to be spaced uniformly throughout the area proposed for percolation, as defined in Table 15.2. If soil conditions are highly variable, more tests will be required with quantity and location based on engineering judgment.
Preparation of Test Hole	The diameter of each test hole are to be a uniform dimension of 12 inches, dug or bored to the proposed depth of the absorption system or to the most limiting soil horizon. Each test hole shall have a minimum depth of 18-inches. To expose a natural soil surface, the sides of the hole are to be scratched with a sharp pointed instrument and the loose material removed from the bottom of the test hole. Two inches of 1/2 to 3/4 inch gravel are to be placed in the hole to protect the bottom from scouring action when the water is added.
Soaking Period	The hole is to be carefully filled to a depth of 12 inches with clear water. This depth of water shall be maintained for at least 4 hours and preferably overnight if clay soils are present. A funnel with an attached hose or similar device may be used to prevent water from washing down the sides of the hole. Automatic siphons or float valves may be employed to automatically maintain the water level during the soaking period. It is extremely important that the soil be allowed to soak for a sufficiently long period of time to allow the soil to swell to obtain accurate results. In sandy soils with little or no clay, soaking is not necessary. If, after filling the hole twice with 12 inches of water, the water seeps completely away in less than ten minutes, the test can proceed immediately.

Table 15.1 Falling head percolation test procedure

Step	Description
Measurement of the Percolation Rate	Except for sandy soils, percolation rate measurements should be made 15 hours but no more than 30 hours after the soaking period begins. Any soil that sloughed into the hole during the soaking period is to be removed and the water level adjusted to 6 inches above the gravel (or 8 inches above the bottom of the hole). At no time during the test should the water level be allowed to rise more than 6 inches above the gravel. Immediately after adjusting the depth to 6-inches, the water level is to be measured from a fixed reference point to the nearest 1/16 inch at 30 minute intervals. The test shall be continued until two successive water level drops do not vary by more than 1/16 inch. At least three measurements are to be made. After each measurement, the water level is to be readjusted to the 6 inch level. The last water level drop shall be used to calculate the percolation rate. In sandy soils or soils in which the first 6 inches of water added after the soaking period seeps away in less than 30 minutes, water level measurements are to be made at 10 minute intervals for a 1 hour period. The last water level drop shall be used to calculate the percolation rate.
Calculation of the Percolation Rate	The percolation rate is calculated for each test hole by dividing the magnitude of the last water level drop by the time interval used between measurements. The percolation calculation results should be in terms of inches per hour (in/hr). Example: If the last measured drop in water level after 30 minutes is 5/8 inch, the percolation rate = (5/8 in) /(0.5 hrs) = 1.25 in/hr) To determine the percolation rate for the area, the lowest rate obtained from all tests in the basin shall be selected.

Number of Tests (each test includes one soil log hole and one percolation test):

- A minimum of two (2) tests is required per retention basin.
- Each soil log boring hole shall extend at least 10-feet below the bottom of the proposed basin. A soil horizon log shall be prepared for each boring to obtain the approximate soil texture of each soil layer (horizon) observed and to identify soil horizons that may impede percolation.
- Additional tests shall be performed based on proposed basin floor percolation area as set forth in Table 15.2.

Method 2 may be applied using a 12-inch diameter bore hole where it is not practical to excavate a pit for performing the test at the proposed bottom of retention basin. The same procedures shall be applied as set forth in Table 15.1, except that measurements shall be taken with a water level sounder with a measuring tape that meets or exceeds federal specification US GGG-T-106E, with a vertical accuracy of at least 0.008%. The measuring tape shall be able to be accurately read to 0.01 foot. In the event the bore hole is unstable and must be lined, a pit shall be excavated to facilitate use of Method 1 or Method 2.

Table 15.2 Minimum quantity of soil log hole / percolation tests required

Retention Basin Bottom Area, sf	Minimum Number of Tests Required
<10,000	2
≥10,00 and <20,000	3
≥20,000 and <30,000	4
≥30,000 and ≤43,560	5
>43,560	A minimum of 5. Additional percolation tests may be required if the soil borings indicate variation in soil texture within the proposed percolation area.
The tests should be distributed evenly throughout the retention basin using engineering judgment. For example, when 5 tests are required, the typical distribution assuming a square basin would be a test in each corner and one in the middle.	

Field percolation test values should be reduced by a safety factor when designing any percolation facility (Stahre and Urbonas, 1990). This is necessary because soils will tend to clog with time, which has proven to be a significant cause for basin failure to drain within 36-hours in Maricopa County and other locations throughout the United States. The Method 2 percolation test includes measurement of sidewall infiltration. The desired design percolation rate must be for the basin bottom only. Therefore, the measured percolation rate must be adjusted to negate the sidewall infiltration. This is done by applying a sidewall correction factor. For a 12-inch diameter test hole with a 6-inch water depth, the sidewall correction factor is determined using Equation 15.1.

$$CF_{sw} = \frac{1}{A_b / (A_b + A_{sw})} \tag{15.1}$$

$$CF_{sw} = \frac{1}{(3.1416)(6^2) / \left((3.1416)(6^2) + (3.1416)(12)(6)\right)}$$

$$CF_{sw} = \frac{1}{0.333} = 3.0$$

where:

CF_{sw}	=	Sidewall correction factor,
A_b	=	Area of the bottom of the test hole, in in^2, and
A_{sw}	=	Area of the test hole sidewalls for a 6-inch water depth, in in^2.

The design factor to be applied shall be selected from Table 15.3 for the percolation test method used, and the subsurface conditions identified by the soil boring holes. The measured percolation rate shall then be adjusted for design using Equation 15.2. The tests shall be performed by a testing laboratory, and the results sealed by a civil engineer, licensed to practice in the State of Arizona. Stormwater disposal by percolation is not allowable if the percolation rate, after application of the design factor, is less than 0.5 inches per hour. Stormwater disposal by percolation is also not allowable if groundwater or an impermeable layer is encountered within 4-feet below the bottom of the basin.

Table 15.3 Percolation design factors for design

<table>
<tr><th rowspan="3">Condition</th><th colspan="4">Design Factor</th></tr>
<tr><th rowspan="2">Method 1 (ASTM) Design Factor</th><th colspan="3">Method 2 (EPA, 1980)</th></tr>
<tr><th>Sidewall Correction Factor</th><th>De-rating Factor</th><th>Design Factor (3)*(4)</th></tr>
<tr><th>(1)</th><th>(2)</th><th>(3)</th><th>(4)</th><th>(5)</th></tr>
<tr><td>No groundwater or impermeable layer is encountered within 10-feet below the bottom of the basin, and the soils are of similar texture to those where the percolation test is taken. The geotechnical engineer may specify a higher de-rating factor based on analysis of the soil conditions below the basin bottom.</td><td>2</td><td>3</td><td>2</td><td>6</td></tr>
<tr><td>Groundwater or an impermeable layer is encountered within 4-feet to 10-feet below the bottom of the basin</td><td>4</td><td>3</td><td>4</td><td>12</td></tr>
</table>

$$P_d = \frac{P}{D_r} \tag{15.2}$$

where:

P_d	=	Design percolation rate, in inches/hour,
P	=	Lowest measured percolation rate, in inches/hour, and
D_r	=	Design factor from Table 15.3.

Basin drain time is estimated by using Equation 15.3.

$$T_d = \frac{V}{A_p \frac{P_d}{12}} \tag{15.3}$$

where:

T_d	=	Retention basin drain time in hours,
A_p	=	Percolation area (basin bottom), in acres
P_d	=	Design percolation rate, in inches/hour, and
V	=	Retention basin design storage volume, in acre-feet.

Only the bottom area of the retention basin may be used for computing the basin drain time by infiltration/percolation. The side slope areas shall not be used in the drain time computation unless the basin configuration is "V" shaped without a flat bottom. For a "V" shaped basin without a flat bottom, the bottom area assumed available for percolation shall be computed using Equation 15.4.

$$A_p = \frac{2 * \left({}^{D}/_{3}\right) * SS * L}{43{,}560} \tag{15.4}$$

where:

A_p	=	Percolation area (approximate), in acres,
D	=	Design ponding depth, in feet,
SS	=	Basin side slope, in feet/foot, and
L	=	Length of retention basin, in feet.

Dry Wells. Drywells shall be designed, operated, and maintained in conformance with the most current ADEQ guidelines. The above Method 1 or Method 2 percolation test procedures may be used for estimating initial design percolation rates for dry wells. The final design rate shall be based on a constant-head percolation test performed on each completed well at the site. The test results for each well shall be de-rated based on the in-situ soil conditions. A de-rating factor of 2 shall be applied for coarse-grained soils (cobbles, gravels and sands). A de-rating factor of 3 shall be applied for fine grained soils (silts and loams). A de-rating factor of 5 shall be applied for clay soils. These de-rating factors are required to compensate for deterioration of the percolation capacity over time in addition to providing a factor of safety for silting and grate obstruction. The accepted design disposal rate for a dry well, after application of the de-rating factor, shall not be less than 0.1 cfs per well. The maximum allowable rate, after application of the de-rating factor, shall not exceed 0.5 cfs per drywell in any case for design purposes. It shall be the owner's, or owner's representatives', responsibility to clean and maintain each dry well to ensure that each remains in proper working order. Under no condition shall the regular maintenance schedule exceed 3-years. Drywells that cease to drain a retention basin with 36-hours shall be replaced or refurbished by the owner or his representative. Maintenance requirements shall be written in the CC&R's for subdivisions where dry wells are used to drain retention basins. In accordance with ADEQ requirements, the installation of any subsurface drainage structure must be located into a permeable porous strata at least 10-feet above saturated soils and 100-feet away from any water supply well.

Gravity Bleed-off. Where bleed-off pipes are to be used as the primary means of draining a retention-type stormwater storage basin, the pipe shall drain the 100-year (design) stormwater storage volume within 36 hours, but in no less than 24 hours.

As a part of the design of the bleed-off system, the design engineer shall evaluate and show that discharge flow rate post-development times of concentration do not adversely affect downstream peak discharges.

Retention systems using a bleed-off method shall meet the first flush requirements of Chapter 6.

The proposed diameter of a basin drain pipe should be rounded up to the nearest standard size made by pipe manufacturers. The minimum allowable pipe size for primary outlet structures is 18-inches in diameter. A permanently attached, hinged orifice plate shall be used to meter the outflow, in conformance with Figure 8.5 of the Hydraulics Manual. Bleed-off time shall be calculated by the Modified Puls storage routing method. Refer to the Hydraulics Manual for example computations.

The required basin drain time may be extended, with prior approval by the COUNTY/DISTRICT, for major storage basins (> 50 acre-feet). Vector control provisions will be one of the requirements for approval of an extended drain time.

Field investigations shall be performed and shall include soil borings and percolation tests taken at the bottom of the proposed basin to obtain percolation rates for use in the design of the stormwater storage facility.

15.4.2 STORMWATER DETENTION

15.4.2.1 Allowable Use

The use of a detention basin in lieu of a retention basin is not allowed without approval in writing by the MOHAVE COUNTY ENGINEER. In the special case where approval is granted by the MOHAVE COUNTY ENGINEER to waive the requirement to retain the 100-year 2-hour runoff volume, the stormwater quality requirements (Chapter 6) must still be met.

Possible special cases where the retention requirement may be waived in favor of detention are as follows:

1. A major drainageway or watercourse is available to accept runoff from the subject site that has sufficient hydraulic capacity to safely convey the 100-year pre-development peak discharge. To be approved: 1) watershed timing issues must be studied and determined to not be an issue for downstream properties, 2) system sediment balance must not be significantly affected, and 3) cumulative impacts of applying such a policy throughout the watershed must not be detrimental to public safety or property.
2. An approved Area Drainage Master Plan for the area states application of detention basins is acceptable.
3. Sensitive riparian vegetation in a downstream watercourse would be adversely affected by application of the retention basin requirement.

15.4.2.2 Minimum Design Storm Criteria

Post-development peak discharges shall not exceed pre-development peak discharges for the 2-, 10-, 50-, and 100-year storm events for the design of detention basins.

15.4.3 STORMWATER STORAGE BASIN DESIGN

15.4.3.1 Basin Depth

Stormwater retention basins should typically have a maximum water depth of 3 feet for the 100-year, 2-hour storm event. Deeper water depths for the design event shall address safety issues.

15.4.3.2 Adjacent to Streets

The required stormwater retention volume shall not intrude upon the public road right-of-way and shall be set back a minimum of 5-feet from the right-of-way (R/W).

15.4.3.3 Berms

Berms are not to be placed closer than 13 feet from the back of the curb, or 8 feet from the back of the sidewalk. Berms are not to be higher than 2-feet above grade on the downhill side. Berms must have a minimum top width of 8 feet. A overflow area (emergency spillway) shall be provided.

15.4.3.4 Side Slopes

Side slopes of stormwater retention facilities are to be no steeper than 3:1 unless prior approval is received for a steeper slope, considering safety issues and erosion control. Stormwater retention basin sides, edges, or top of slopes should be varied to provide an esthetically pleasing appearance.

15.4.3.5 Protective Measures

Protective measures shall be required for all constructed drainage basins located in developed areas, with side-slopes steeper than 3:1 or depths exceeding three (3) feet, unless provisions are made for safe exit from the facility during flooding conditions, or other deterrents to access during unsafe conditions are provided. Determining the type of protection measures shall be based on sound engineering judgment for the intended application and must be sealed by an Arizona registered civil engineer.

15.4.3.6 Sediment Storage Requirement

Sedimentation basins, which may be required, are to be located at the upstream (inlet) portions of stormwater storage facilities. The sediment settling basins shall be easily accessible by maintenance equipment (such as backhoes) and should have a minimum storage volume equivalent to the 2-year watershed sediment yield, in addition to suitable storage volume and conveyance capacity to ensure transfer of the 100-year 2-hour runoff volume and the 100-year peak discharge into the retention facility, or the 100-year design storm volume and peak discharge into the detention facility.

15.4.3.7 Emergency Spillway Requirement

Emergency spillways shall be provided for all stormwater storage basins. For basins with all the design storage volume situated below existing grade (i.e. without a berm/dam), the spillway may be nothing more than grading to ensure that basin overflow will follow the downstream predevelopment drainage pattern in a safe manner.

Emergency spillways shall be designed to safely convey the peak discharge from the storm listed in Table 15.4, exclusive of the attenuation effects of the basin.

Table 15.4 Emergency spillway design capacity requirements

For an embankment berm/dam that is not regulated by ADWR	
Berm/Dam Height	**Spillway Design Capacity**
H < 6 ft.	Unattenuated 100-year inflow
6 ft. <= H < 25 ft.	1/2 Probable Maximum Flood

where:

> Berm/Dam height is the vertical distance from the lowest point along the downstream slope to the crest of the emergency spillway.
>
> 100-year inflow is the unattenuated peak discharge from the pre- or post-development 100-year 6-hour or 24-hour storm, whichever is larger.
>
> Refer to Section 2.4 for information regarding dams regulated by ADWR.

Emergency spillways shall be designed to convey the design peak discharge and erosion protection shall be provided in accordance with the Hydraulics Manual.

Down-gradient properties are to be protected from flow depths and velocities in excess of pre-development conditions.

A 1 foot minimum freeboard is required between the berm crest and the water surface elevation of the 100-year peak discharge in the emergency spillway (without attenuation from basin storage), except where the berm crest is designed to function as the emergency spillway.

The finished floor elevation of adjacent structures must be at least 1.0 feet above the 100-year peak water surface elevation of the flow passing through the emergency spillway.

15.4.3.8 Landscaping

Landscaping components should not adversely affect the basin hydrologic and hydraulics functions.

15.4.3.9 Ownership and Maintenance Requirements

Regional: Publicly-owned COUNTY/DISTRICT Maintained.

Commercial. Drainage easement attached to the deed for single parcels. Privately maintained.

Commercial or Residential Subdivision . Parcel recorded on the Final Plat. Privately maintained.

Single Family Residential (>1 acre): Drainage easement attached to the deed. Privately maintained.

15.4.3.10 NPDES Requirement

Discharges from stormwater facilities must be in compliance with 40 CFR 122, the NPDES, and the AZPDES.

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16 EROSION CONTROL DURING CONSTRUCTION

16.1 INTRODUCTION

Construction activity disturbs the land surface, thereby exposing native soils to increased rates of erosion by wind and rain. Airborne soil poses detrimental health risks and reduces visibility. Erosion of soil from construction sites by storm water increases the rate of siltation of drainageways, which can exacerbate flooding and increase the cost of on-going maintenance. Appropriate erosion control measures are required at construction sites.

16.2 TECHNICAL DESIGN CRITERIA

Technical guidance for erosion control during construction shall be in accordance with the Flood Control District of Maricopa County Erosion Control Manual (FCDMC, 2009).

16.3 EROSION CONTROL DURING CONSTRUCTION POLICIES

16.3.1 STORMWATER CONVEYANCE DURING CONSTRUCTION

Stormwater conveyance is to be provided at all times during construction in a manner that does not increase flood depths, sedimentation, or erosive velocities above pre-construction levels for the areas adjacent to, and downstream of, construction projects.

16.3.2 STORM WATER POLLUTION PREVENTION PLANS

Storm Water Pollution Prevention Plans (SWPPP) or Storm Water Management Plans (SWMP) that incorporate best management practices are required of new development/construction. See NPDES Storm Water construction requirements for full details.

16.3.3 PERMITTING

There are a myriad of federal and state permits that may be required prior to the start of construction of a project (see Chapter 2). It is not Mohave County's responsibility to ensure that the plans for a proposed project satisfy state and federal permit requirements. This notwithstanding, COUNTY may not approve engineered grading projects nor recommend grading permit issuance until the applicant documents that all of the applicable state and federal permits have been obtained.

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17 SEDIMENTATION

17.1 INTRODUCTION

Sedimentation and the fluvial processes associated with sediment transport play an important role in the long-term conveyance capacity of a drainage system as well as the on-going cost of maintenance. Sedimentation is a very complex subject and not all drainage designs need to consider it as a primary design criteria. The policies and standards in this chapter define the conditions where sedimentation should be considered as an important design parameter.

17.2 SEDIMENTATION TECHNICAL CRITERIA

Technical guidance for defining and addressing erosion/sedimentation hazards shall be in accordance with the sedimentation chapter of the Hydraulics Manual. Specific design policies and standards for Mohave County needed for application of the technical information contained in the Hydraulics Manual are defined herein.

17.3 SEDIMENTATION POLICIES

The designer of drainage facilities should undertake the appropriate level of erosion and sedimentation analysis commensurate with the risk of undesirable consequences expected to the health, safety, and welfare of the general public. Erosion zones consistent with ADWR (1996b) may be required for all development meeting the criteria defined in Section 17.4 in which the watercourses are to be left in an undisturbed state. ADWR (1996b) specifies a three level approach. Each level must be performed by a registered civil engineer licensed to practice in Arizona. Level I is a simplified approach, requires the watershed area, is only applicable to watersheds with an area less than 30 square miles, and in general provides a conservative estimate for the erosion setback. Engineering analysis is minimal for the Level I approach. Depending on the geomorphic conditions of the area, if the erosion limits are estimated by the COUNTY/DISTRICT to exceed those estimated using a Level I analysis, a more detailed engineered analysis may be required as defined in ADWR (1996b), using a Level II or Level III approach.

A Level II approach is only applicable for watersheds less than 30 square miles in area, and is normally applied if the developer believes the Level I setback is overly conservative, or the COUNTY/DISTRICT suspects the Level I setback is insufficient.

A Level III approach is required if:

1. the watershed area is greater than 30 square miles, or
2. significant shifting of the river channel has been observed in the past, or
3. the area is undergoing channel filling (aggradation) to a significant degree, or
4. local river mining, channelization, or other modifications could result in flow redirection unanticipated in the development of the Level I or Level II setback.

Level III erosion hazard analysis shall be done in conformance with the guidelines set forth in ADWR (1996b) and Hydraulics Manual Chapter 11 Sedimentation.

If erosion hazard guidelines (ADWR, 1996b) are determined to apply, and the developer/homeowner elects to not use the erosion setback approach, structural mitigation measures are required in conformance with Section 17.4.

17.4 SEDIMENTATION STANDARDS

17.4.1 CRITERIA DEFINING NEED FOR SEDIMENTATION ANALYSES

Recognizing that sedimentation and sediment transport is either supply or transport control and that storm water runoff may produce sedimentation or erosion, the following standards shall be applied to:

1. Watercourses with drainage areas equal to or greater than 0.25 sm or a 100-year peak discharge estimate of more than 500 cfs, as estimated using the procedures in Chapter 7 Hydrology.
2. Structures that could fail or incur significant damage as a result of erosion or deposition.
3. Proposed structures that, if built, could result in adverse impacts to adjacent properties.

17.4.2 TECHNICAL REFERENCES

Erosion control improvements shall be designed and constructed in accordance with guidance provided in ADWR (1998) and Hydraulics Manual Chapters 5, 8 and 11. Chapter 11 of the Hydraulics Manual lists and summarizes numerous technical references that are appropriate for use in Mohave County for erosion and sediment transport modeling and design of erosion/sedimentation facilities.

17.4.3 SEDIMENTATION BASIN REQUIREMENTS

Where sediment transport analysis is required, sedimentation basins and/or structures shall be designed and constructed as an integral part of storm water storage and/or conveyance facilities unless a sediment transport analysis of the system demonstrates that sediment is conveyed through the system during frequent events including the 2-year storm event.

Sedimentation basins and/or structures shall be designed to hold a minimum of two years of watershed sediment yield using an annual sediment yield of 0.25 ac-ft/sq. mi./year or the site-specific sediment yield based upon in-situ geomorphic and engineering analyses meeting or exceeding those methods identified in the Hydraulics Manual Chapter 11.

Sediment basins and/or structures shall be designed with minimum 6:1 side slopes and 16-foot wide access ways on opposing sides. Sediment basins and/or structures shall be designed to slow the passage of runoff but not prevent the passage of runoff.

Sediment check structures shall have low flow outlets with inverts set equal to the invert of the drainageway and shall be no higher than 18 inches. All outlets shall be designed for protection from scour per the Hydraulics Manual.

17.4.4 ROADWAY CROSSING REQUIREMENTS

For County Highways, arterial roads, and all-weather access roads crossing a distributary flow area or alluvial fan, the following minimum standards shall apply for the design of culverts or bridges, where a low water crossing approach is not taken:

1. The natural wash bottom width should be completely spanned where possible.
2. A minimum height of 4 feet should be provided (5 feet high is preferred), and the structure flowline set to equilibrium grade (inverts may be buried a maximum of 6-inches for sediment continuity, but the minimum clear opening above the channel invert shall be a minimum of 4 feet) .
3. Culverts shall be sized so that the sediment transport capacity of flow does not vary more than 5% from the existing condition.

For adjacent watercourses, separated by less than 100 feet, where the natural grade of the watercourses at the culvert inlets are within 12 inches vertically, an equalizer ditch should be placed on the upgradient side of the road, between culverts. The bottom width shall be no narrower than 5 feet with side slopes no steeper than 25% (4:1).

17.4.5 CHANNEL CONSIDERATIONS

Encroachment and/or erosion control on one bank of a channel may result in increased erosion potential on the opposite bank. Such adverse effects shall be evaluated and mitigated as a part of the design.

18 DOCUMENTATION REQUIREMENTS

18.1 INTRODUCTION

An important part of any drainage design is complete and accurate documentation. This helps review of the design proceed more quickly, and provides information that will be needed in the future when maintaining, modifying, or extending the improvements. Following an orderly report format also helps ensure that critical items are not missed and make it easier to locate needed information. The following sections identify the preferred outline for preparation of hydrology and hydraulics reports for projects in Mohave County.

18.2 HYDROLOGY AND HYDRAULICS REPORTS

18.2.1 FLOOD INSURANCE STUDIES

Hydrology and hydraulics reports documenting floodplain delineation studies for approval by the DISTRICT and/or FEMA shall be prepared in accordance with ADWR State Standard 1-97 (ADWR, 1997). Checklist E.5 should be used and a completed copy of both provided with the submittal.

18.2.2 PRELIMINARY DRAINAGE DESIGN REPORT

Preliminary Drainage Reports for subdivisions, preliminary design of roadway drainage, or preliminary design of flood control facilities shall, as a minimum, include the following information:

1. Documentation for new and revised hydrology and hydraulic models.
2. Assumptions and parameters that are to be used for design of each drainage system component.
3. Identification of all areas where flooding may affect proposed building pads and other structures. These are the areas where minimum building pad and finished floor elevations for areas within floodplains and backwater ponding from structures or roadway embankments must be specified in the Final Drainage Design Report.
4. Parameters proposed for design of retention basins and location and approximate sizes of proposed retention structures.
5. If a variance from stormwater retention criteria is being requested, a Stormwater Quality Plan identifying proposed permanent stormwater quality features including First Flush provisions shall be provided.

The Table of Contents must be sealed by a Civil Engineer licensed to practice in the State of Arizona. The Preliminary Drainage Report should be organized to include sections as follows (as a minimum):

TABLE OF CONTENTS

1.0 Completed Hydrology and Hydraulics Report General Checklist

2.0 Introduction/Purpose

3.0 Location

4.0 Site Description and Proposed Development

5.0 FEMA Floodplain Classification

6.0 Survey and Mapping

7.0 Off-site Drainage Description

7.1 Background

Describe drainage studies that are of record for the off-site watershed, methods being used to handle drainage, and the anticipated future development conditions. Identify where off-site flow enters and exits the subject project, including multiple frequency discharges, depth, velocity, and extent of floodplain on the subject project and a minimum of 200 feet upstream and downstream of the subject project.

7.2 Proposed Offsite Flow Management Plan

Describe how off-site flows will be addressed for the subject project. The description shall include guidelines to be used for design of the project drainage facilities to ensure that drainage is handled in accordance with the DDM and that there are no adverse impacts on adjacent, upstream, or downstream properties.

8.0 Proposed On-site Drainage Plan Description

Describe the proposed methods for handling on-site drainage. Provide the criteria and guidelines that will be used for design of the project drainage facilities. Provide the preliminary proposed location, size, type and configuration of retention facilities, culverts, channels, storm drains, erosion protection, sedimentation basins and other facilities. Identify proposed flow directions for streets, channels and site grading and drainage. Identify existing floodplain and/or floodway limits, and delineate new floodplain and floodway limits for previously unstudied washes, in accordance with the DDM. Describe how the proposed project will impact the floodplain and/or floodway, and mitigation measures to be taken for those impacts. If a CLOMR or LOMR is proposed, or required by COUNTY/DISTRICT, provide the time table for preparation of the design plans, TDN, and submittals to COUNTY/DISTRICT and FEMA.

9.0 Hydrology

This section shall document all hydrology calculations made for both the on-site and off-site watersheds and set forth in Sections 7.0 and 8.0.

9.1 Methodology

Describe the hydrologic methodology used and any numerical modeling approaches applied, including software used and software versions.

9.2 Parameters

Document all hydrologic modeling parameters including:

9.2.1 Rainfall
9.2.2 Rational Method Parameters
9.2.3 Rainfall Losses
9.2.4 Unit Hydrograph Parameters
9.2.5 Channel Routing
9.2.6 Storage Routing
9.2.7 Transmission Losses

9.3 Results

Present and discuss the hydrologic modeling results.

9.4 Confidence Checks, Calibration and Sensitivity Analyses

Document confidence checks made in conformance with DDM Section 7.11, any calibration efforts, and any sensitivity analyses conducted for critical parameters.

9.5 Unique Conditions and Problems

Document any unique watershed conditions and modeling problems encountered and describe how each was addressed. This includes documenting all modeling warning and error messages in the computer model output and describing the significance of the messages and, if necessary, what has been done to address them.

10.0 Hydraulics

This section shall document all preliminary hydraulic calculations made for both the on-site and off-site watersheds and set forth in Sections 7.0 and 8.0.

10.1 Methodology

10.1.1 Floodplain Delineations

Describe the hydraulic methods used for floodplain/floodway delineations, including the numerical modeling approaches applied, software used and software versions. Describe the methods used to delineate floodplain boundaries and the compute floodway limits. Describe the cross section placement, orientation, and how cross sections were obtained.

10.1.2 Project Drainage Conveyance Facilities

Describe the methods used for preliminary sizing of proposed culverts, channels, storm drains, flow transition facilities, and other stormwater conveyance facilities. List software used including software versions.

10.2 Parameters and Modeling Considerations

Document all hydraulic modeling parameters including:

10.2.1 Floodplain Delineations

10.2.1.1 Flow Regime
10.2.1.2 Frequencies for which Profiles were Computed
10.2.1.3 Roughness Coefficients
10.2.1.4 Expansion and Contraction Coefficients
10.2.1.5 Hydraulic Jump and Drop Analysis
10.2.1.6 Bridges and Culverts
10.2.1.7 Levees and Dikes
10.2.1.8 Islands and Flow Splits
10.2.1.9 Ineffective Flow Areas
10.2.1.10 Supercritical Flow
10.2.1.11 Floodway Modeling

10.2.2 Project Drainage Conveyance Facilities

10.2.2.1 Roadway Drainage
10.2.2.2 Storm Drains
10.2.2.3 Culverts and Bridges
10.2.2.4 Open Channels
10.2.2.5 Hydraulic Structures

10.3 Results

Present and discuss the hydraulic modeling results.

10.3.1 Floodplain Delineations
10.3.2 Project Drainage Conveyance Facilities

10.4 Calibration and Sensitivity Analyses

Document any calibration efforts made, and any sensitivity analyses conducted for critical parameters.

10.4.1 Floodplain Delineations
10.4.2 Project Drainage Conveyance Facilities

10.5 Unique Conditions and Problems

Document any unique hydraulic conditions and modeling problems encountered and describe how each was addressed. This includes documenting all modeling warning and error messages in the computer model output and describing the significance of the messages and, if necessary, what has been done to address them.

10.5.1 Floodplain Delineations
10.5.2 Project Drainage Conveyance Facilities

11.0 Stormwater Retention and First Flush Requirements

Document the proposed location, size and design criteria and guidelines for proposed retention facilities. Identify if first flush facilities will be necessary and if so, where they are to be located.

12.0 Minimum Finished Floor Elevation Requirements

Identify where minimum finished floor elevations are anticipated to be necessary and specified in the final construction documents and on the Final Plat.

13.0 Sedimentation and Erosion Hazards Discussion
Discuss sedimentation issues and erosion hazards that must be addressed during the design phase. Provide design criteria and guidelines for use by the design engineer.

15.0 Stormwater Permits Requirements (401/404, Floodplain, Right-of-Way, Stormwater Quality, and other permit requirements)
Identify stormwater, drainage, and floodplain permits that will be necessary for the project.

16.0 Conclusions and Recommendations

17.0 References

FIGURES

Figure 1	Area Location Map
Figure 2	Site Aerial Photo Map
Figure 3	FIRM Map
Figure 4	Off-site Watershed Map
Figure 5	On-site Watershed Map
Figure 6	On-Site Drainage and Grading Plan
Figure 7	Floodplain/Floodway Delineation Work Study Maps
Figure 8	Floodplain Profiles

APPENDICES

Appendix A	Offsite Hydrology Documentation
Appendix B	On-Site Hydrology Documentation
Appendix C	Channel Design and Floodplain Hydraulics Documentation
Appendix D	Street Capacities & Storm Drain Analysis Documentation
Appendix E	Stormwater Storage and First Flush Documentation
Appendix F	Stormwater Quality Documentation
Appendix G	Sediment and Erosion Hazard Documentation
Appendix H	Digital Data/Model Input and Output Files

If a report section does not apply, the section should still be included and identified as not applicable. The outline numbering scheme should be maintained. Add subsections as necessary to describe methodologies used and results. Include supporting calculations and computer printouts in the appropriate appendix.

18.2.3 FINAL DRAINAGE DESIGN REPORT

The Final Drainage Design Report should, as a minimum, include the following information:

1. Documentation for new and revised hydrology and hydraulic models that have changed since the Preliminary Drainage Report.
2. Design assumptions and parameters for each drainage system component.
3. Minimum building pad and finished floor elevations for areas within floodplains and backwater ponding from structures or roadway embankments.
4. Retention basin design parameters and rating curves.
5. If a variance from stormwater retention criteria is being requested, a Stormwater Quality Plan documenting permanent stormwater quality features including First Flush provisions shall be provided.
6. It is also recommended that a Stormwater Pollution Prevention Plan (SWPPP), as filed with ADEQ, documenting recommended BMP's and recommended BMP locations for the various phases of the construction process, be included as an appendix of the Final Drainage Design Report.

The Table of Contents must be sealed by a Civil Engineer licensed to practice in the State of Arizona. The Final Drainage Design Report should be organized to include the same sections and sub-sections as the Preliminary Drainage Report. The report shall document all changes made since the Preliminary Drainage Report and all design computations and results for the project drainage facilities under the appropriate report section.

If a report section does not apply, the section should still be included and identified as not applicable. The outline numbering scheme should be maintained. Various designed components should be included under the appropriate section. Add subsections as necessary to describe methodologies used and results. Include supporting calculations and computer printouts in the appropriate appendix.

18.2.4 DRAINAGE DESIGN REPORT CHECKLISTS

Each report should contain the applicable hydrologic and hydraulic analysis checklists shown in Appendix E, completed as appropriate for the proposed project.

18.2.5 ADDITIONAL REPORT REQUIREMENTS

1. Drainage Design Reports shall include, but not be limited to, the following items:
   a. Professional engineer seal, signed and dated, on the Title page and Table of Contents.

b. A drainage map that shows the discharges at points of concentration and clearly identifies the existing drainage system. Minimum scale will be 1 inch equals 500 feet. Where drainage areas are large, a smaller scale may be approved.

c. Detailed street hydraulic analysis and storm drain analysis (where required).

d. Calculations for the proposed stormwater retention facilities showing storage volume required and retention volume provided, and First Flush calculations. If more than one facility is proposed, calculations must be separated for each area, and each tributary area referenced to its respective stormwater storage facility. Analysis confirming that each basin will drain within 36 hours of the end of the design precipitation event is required.

e. If adjacent land drains into or is diverted around the development, the adjacent contributory drainage area must be shown and quantified. Size of the adjacent drainage area and slope of the land information shall be shown.

f. A schematic line drawing of the proposed drainage system in plan view showing design flow and capacity.

g. Sufficient information to determine the path of the water entering and leaving the project property under pre-development and post-development conditions. Sufficient information to show that proposed conditions do not pond water on adjacent properties or change the historical flow path and pre-development hydrologic and hydraulic characteristics of stormwater leaving the property.

h. Typical cross sections of all street classifications.

i. FEMA floodplains in and adjacent to the project area as an exhibit or figure.

j. Summary of previously prepared drainage reports pertinent to the subject area.

18.3 GENERAL CONSTRUCTION DRAWING REQUIREMENTS

18.3.1 CONSTRUCTION DOCUMENTS

Construction documents shall comply with requirements in the Mohave County Engineering Design Standards, Specifications and Details for items to be installed or constructed in public rights-of-way or easements.

18.3.2 PREPARATION BY LICENSED PROFESSIONAL

All plans for engineered drainage improvements shall be prepared under the direction of a Civil Engineer licensed to practice in the State of Arizona, and sealed, dated and signed by that engineer.

18.3.3 PLAN REQUIREMENTS FOR Q_{100}<50 CFS

Engineered drainage improvements designed for flows less than 50 cfs may be shown in plan view with spot elevations, flow direction arrows, and typical sections. The plan shall show the horizontal alignment and dimensions as well as the type and extent of the proposed work. Other elements from Section 18.3.5 may be required.

18.3.4 PLAN REQUIREMENTS FOR 50 CFS≤Q_{100}<500 CFS

All drainage improvement plans may be required to contain a plan and profile as well as adequate cross sections to describe geometry.

The profile, if required, shall show the following: proposed invert, estimated water surface profile, energy grade line, hydraulic jump location and length, original ground at channel center line, top of slope, all utilities and structure crossings, and if necessary, top of proposed embankment and fill including freeboard as required. Other elements from Section 18.3.5 may be required.

18.3.5 PLAN REQUIREMENTS FOR Q_{100}≥500 CFS.

The following are general requirements for drainage improvement plans:

1. Information to determine drainage patterns, including direction of flow for ditches, channels, natural drainageways, etc.
2. The type of work on existing facilities shall be clearly indicated.
3. Temporary construction and permanent easements and rights-of-way lines shall be clearly shown and labeled.
4. Inlet and outlet elevations for all drainage facilities.
5. Type and thickness of drainage facility linings.
6. Type, dimensions, and details of elements providing scour and erosion protection, including aprons, cut-off walls, and bank protection, shall be clearly shown and identified.
7. Existing ground line profile shown as a dashed line and finished grade line as a solid line.
8. Profile line of drainage facilities such as ditches and drainage channels with the slope labeled as a decimal in ft/ft.
9. Information to determine that an adjacent property drainage pattern will not be adversely affected.
10. A HEC-RAS analysis for designed channels and existing washes shall be provided. The model characteristics and results shall be submitted in plan and profile at a scale not to exceed 1”=100’. The plan view shall show existing and proposed ground contours, depict the exact location of the beginning and end point locations of each cross section, the left

and right bank station alignments, the limits of defined reaches, and 100-year floodplain limits. Profiles shall include the existing ground, design water surface, and the energy gradeline. This information is to be provided with the design data sheet(s) from the hydrology/hydraulics report. The following data shall also be included in addition to the HEC-RAS standard output tables:

a. Delta water surface elevation change between cross sections.

b. Left bank freeboard.

c. Right bank freeboard.

d. Velocity distribution for each cross section.

11. Profiles of storm drains, culverts, box culverts, and catch basins and connector pipes shall be provided. These profiles shall show gutter elevation, top of curb elevation, catch basin type, depth, size and cross-section, connector pipe invert at the catch basin and at the inlet to the main line storm drain (as well as any grade breaks), connector pipe size and slope in ft/ft, and the location and size of existing and proposed utilities along the profile and in the vicinity of the catch basin. Each catch basin profile shall be labeled by road centerline station or main storm drain stationing if different. Profiles shall also include:

    a. Pipe material and dimensions labels.

    b. Inlet and outlet structures and wingwalls, if any, clearly identified and labeled.

    c. The finished street elevation over the storm drain pipe.

    d. The pipe profile and size.

    e. The design peak discharge (cfs) in each storm drain pipe segment.

    f. The velocity (fps) in each storm drain pipe segment.

    g. Appropriate stationing.

12. On the storm drain plan sheets, the engineer shall show the rim and invert elevations at all existing sanitary sewer manholes.

13. In plan and profile, existing and proposed underground utilities shall be labeled according to size and type. Corresponding alphanumeric labels shall be shown for each utility and depicted in the legend. If the utility is an underground conduit, give all the details such as number of ducts and whether or not the conduit is encased in concrete. Any utilities to be constructed prior to the project shall be shown and so indicated. Conflicts between existing utilities and proposed construction are to be identified. Utilities that are abandoned or to be abandoned shall be indicated as well as those designated to be relocated or removed. The engineer shall contact the appropriate utility if any questions arise about types or locations of underground facilities. Existing and proposed underground tanks shall also be shown.

14. The minimum vertical clearance between a proposed storm drain and all existing utilities shall be 1 foot unless otherwise required by the given utility.

15. Below ground utilities shall be dimensioned from the road center or monument line.

16. Above ground utilities such as power poles, light poles, guys and anchors, irrigation structures, utility pedestals, transformers, switching cabinets, gas regulators, waterline back-flow prevention units, and other features shall be called out including size and pad

elevation, shown in plan view, and stationed relative to the adjacent road monument line or centerline from the street side face of the utility (e.g. 12+33 R 32').

17. When below ground appurtenances (utilities, monuments, tanks, valve boxes, and other features) depicted on As-Built or "Record" drawings cannot be field located, they shall be shown and labeled as "not found".

18. The following items shall be shown on storm drain plan and profile sheets:

    a. New storm drain pipe

    b. Manholes/Junction structures

    c. Catch basins

    d. Connector pipe

    e. Pipe collars

    f. Prefabricated pipe fittings

    g. Other drainage appurtenances (headwalls, trashracks, drop inlets, hand rails, pipe supports, etc.).

19. Where new street paving work joins existing side streets, pavement crown and gutter elevations are required to be displayed and shall be shown in plan view for a minimum of 100 feet beyond the curb return on the side street intersections. Where new street paving work joins an existing street linearly, the existing pavement crown and gutter elevation shall be a minimum of 300 feet beyond the new work to ensure proper drainage and a smooth ride for vehicular traffic.

20. All storm drain plans shall have the following format:

    a. Storm drain designs shall be depicted on single plan/profile sheets.

    b. Main line storm drain plans shall be 1 inch=20 feet horizontal and 1 inch=2 feet vertical, unless otherwise approved.

    c. Scales for connector pipe/catch basin profiles shall be 1 inch=5 feet horizontal and 1 inch=5 feet vertical, unless otherwise approved.

    d. Profile slopes shall be shown in feet per foot dimensions to four significant figures.

    e. Grade breaks shall be stationed with elevations shown. Station and elevations shall also be shown at sheet match lines and at the beginning/end of the storm drain.

    f. Centerline stationing shall be shown on plan and profile. Stationing shall run from the low point, or outfall, and increase toward the high point or inflow. Where the storm drain is being installed in conjunction with a paving project (i.e. depicted on corresponding paving plans), the stationing shall be correlated with the paving project stationing.

    g. Final plan sheets shall be 24 inch x 36 inch. As-built plans shall be 24 inch x 36 inch ink on mylar.

    h. Letter size on full size drawings shall be 14 point minimum.

    i. Title blocks shall be located in the lower right-hand corner of the plans and shall include the title "Grading and Drainage Plans".

j. Storm drain diameters shall be shown in plan and profile without reference to material type.

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19 REVISION PROCESS

Those seeking changes to policies or standards contained in this manual must make a formal submittal to the MOHAVE COUNTY ENGINEER stating the present policy/standard, identifying the proposed change(s), and providing comprehensive justification for the change. A review committee, appointed by the MOHAVE COUNTY ENGINEER, will convene periodically to review requested changes. If proposed changes are found appropriate by the review committee, the manual will be revised in draft form, posted on the Mohave County Public Works Drainage Design Manual web page (http://www.co.mohave.az.us/FloodControlDrainageDesignManual), and notices send out to holders of the manual soliciting review and comments. A notice regarding the availability of the new draft document for review and comment and the review period will be posted on the Mohave County Public Works web page. Public review comments received will be carefully considered and changes made if appropriate. The revised manual will be forwarded to the Flood Control District Board of Directors and the County Board of Supervisors for approval.

Amendment application forms are available from Mohave County Public Works. Six copies of the completed application and supporting documents should be delivered to Mohave County Public Works. Upon review and certification of a complete submittal, a date will be assigned at which time the committee will review requested amendments.

The current adopted Drainage Design Manual for Mohave County will be posted on the Mohave County Public Works web page

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20 REFERENCES

Aldridge, B.N., and Garrett, J.M., 1973, *Roughness coefficients for stream channels in Arizona*, U.S. Geological Survey Open-File Report, 87 p.

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American Steel and Iron Institute (AISI), 1980, *Modern Sewer Design (4th Ed 1999)*,

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21 GLOSSARY

100-year Storm / Flood

A storm or flood with a one (1) percent chance of occurring or being exceeded within any given year.

Adverse Impacts

Any physical action performed in a floodplain that adversely impacts the property and rights of others. An adverse impact can be measured by an increase in flood stages, flood velocity, flows, the potential for erosion and sedimentation, degradation of water quality, or increased cost of public services.

All Weather Access

Each lot within a subdivision shall have at least one vehicular access route which, regardless of street width design classification, provides access to and from the lot for private and emergency vehicles during flood events. Such routes are referred to as “All Weather Access” routes.

Base Flood Elevation

The water surface elevation of a flood having a one percent (1%) chance of being equaled or exceeded in any given year.

Best Management Practices (BMPs)

Schedules of activities, prohibitions of practices, maintenance procedures, and other management practices to prevent or reduce the discharge of pollutants to stormwater discharges. BMPs also include treatment requirements, operating procedures, and practices to control site runoff, spillage or leaks, sludge or waste disposal, or drainage from outdoor storage areas.

Bridge

Structures designed to span a watercourse, including bridges for vehicular roadways and pedestrian-only uses.

Conditional Letter of Map Amendment (CLOMA)

A letter from FEMA stating that a proposed structure that is not to be elevated by fill would not be inundated by the 100-year flood if built to the proposed finished floor elevation.

Conditional Letter of Map Revision (CLOMR)

A letter from FEMA commenting on whether a proposed project, if built as proposed, would justify a map revision.

Conditional Letter of Map Revision Based on Fill (CLOMR-F)

A letter from FEMA stating that a parcel of land or proposed structure that is to be elevated by fill would not be inundated by the 100-year flood if fill is placed on the parcel as proposed or the structure is built as proposed.

Culvert

Buried pipe or box hydraulic conveyance structure designed to convey stormwater from one side of a roadway, embankment, or service area to the other side.

Erosion

The wearing away of the earth's surface by running water.

Design Storm

The storm frequency and duration that a drainage facility is designed for, as a minimum.

FEMA 100-year Floodplain

An area of ground described by a FEMA/FIRM map as a potential flood hazard, or the delineation of a potential flood hazard.

FEMA Regulatory Floodway

A "Regulatory Floodway" means the channel of a river or other watercourse and the adjacent land areas that must be reserved in order to discharge the base flood without cumulatively increasing the water surface elevation more than a designated height. In Arizona, that height is one (1) foot.

Lowest Floor Elevation

The elevation of the lowest floor of the lowest enclosed area of a structure, including the basement.

First Flush

The initial or early stages of stormwater runoff from a storm event which commonly delivers a disproportionately large amount of previously accumulated pollutants due to the rapid rate of runoff. The first flush is defined as the first one-half (1/2) inch of direct runoff from the contributing drainage basin.

Letter of Map Amendment (LOMA). A letter from FEMA stating that an existing structure or parcel of land that has not been elevated by fill would not be inundated by the 100-year flood.

Letter of Map Revision (LOMR)

A letter from FEMA officially revising the current FIRM to show changes to floodplains, floodways, or flood elevation. Physical changes include watershed development, flood control structures, etc.

Letter of Map Revision Based on Fill (LOMR-F)

A letter from FEMA stating that an existing structure or parcel of land that has been elevated by fill would not be inundated by the 100-year flood.

Pollutant

Fluids, contaminants, toxic wastes, toxic pollutants, dredged spoil, solid waste, substances and chemicals, pesticides, herbicides, fertilizers and other agricultural chemicals, incinerator residue, sewage, garbage, sewage sludge, munitions, petroleum products, chemical wastes, biological materials, radioactive materials, heat, wrecked or discarded equipment, rock, sand, cellar dirt and mining, industrial, municipal and agricultural wastes or any other liquid, solid, gaseous or hazardous substances.

Physical Map Revision (PMR)

A reprinted FIRM incorporating changes to floodplains, floodways, or flood elevations. Because of the time and cost involved to change, reprint, and redistribute a FIRM, a PMR is usually processed when a revision reflects increased flood hazards or large-scope changes.

Regulatory Flood Elevation

Regulatory Flood Elevation means an elevation one foot (1') above the Base Flood Elevation for a Watercourse for which the Base Flood Elevation has been or shall be as determined by the criteria developed by the Director of ADWR (ADEM or other designated agency) for any other and/or all other Watercourses.

Regulatory Floodway (or Floodway)

Regulatory Floodway (or Floodway): channel of a river or other Watercourse and the adjacent land areas that must be reserved in order to discharge the Base Flood without cumulatively increasing the Water Surface Elevation more than one (1) foot.

Scour

Localized erosion in a channel resulting from a physical obstruction to flow.

Sedimentation

The depositing of sediment by fluvial processes.

Subdivision

The definition from the current version of the Mohave County Land Division Regulations is used for the purposes of this document.