Town of Prescott Valley

Uniform Drainage Policies and Standards Manual

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PRESCOTT VALLEY

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LIST OF ACRONYMS

Term	Description
1D	One-dimensional
2D	Two-dimensional
AAC	Arizona Administrative Code
ACPA	American Concrete Pipe Association
ADEQ	Arizona Department of Environmental Quality
ADMP	Area Drainage Master Plan
ADMS	Area Drainage Master Study
ADOT	Arizona Department of Transportation
ADWR	Arizona Department of Water Resources
AF	Adjustment Factor
AISI	American Iron and Steel Institute
ALTA	American Land Title Association
ADA	Americans with Disabilities Act
APP	Aquifer Protection Permit
ARS	Arizona Revised Statue
AZPDES	Arizona Pollutant Discharge Elimination System
BFE	Base Flood Evaluation
BMP	Best Management Practice
CADD	Computer-aided Design and Drafting
CFR	Code of Federal Regulations
cfs	Cubic Feet Per Second
CGP	Construction Activity General Permit
CIPP	Cast In-Place Pipe
CLOMA	Conditional Letter of Map Amendment
CLOMR	Conditional Letter of Map Revision

Term	Description
CLOMR-F	Conditional Letter of Map Revision Based on Fill
CMP	Corrugated Metal Pipe
CRS	Community Rating System
CSP	Corrugated Steel Pipe
CSPI	Corrugated Steel Pipe Institute
CWA	Clean Water Act
DFIRM	Digital Flood Insurance Rate Map
DTHETA	Volumetric Soil Moisture Deficit
EPA	Environmental Protection Agency
FB	Freeboard
FCDMC	Flood Control District of Maricopa County
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
FIRM	Flood Insurance Rate Map
FIS	Flood Insurance Study
fps	Feet Per Second
GIS	Geographic Information System
HDPE	High Density Polyethylene
HVAC	Heating, Ventilation, and Air Conditioning
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
LP3	Log-Pearson Type 3
MAG	Maricopa Association of Governments
MAP	Mapping, Assessment, and Planning

Term	Description
MDP	Master Drainage Plan
MSGP	Multi-Sector General Permit
MS4	Municipal Separate Storm Sewer System
NEXRAD	Next-Generation Radar
NFIP	National Flood Insurance Program
NOAA	National Oceanic and Atmospheric Administration
NOI	Notice of Intent
NOT	Notice of Termination
NOV	Notice of Violation
NPDES	National Pollution Discharge Elimination System
NRCS	Natural Resource Conservation Service
NSS	National Streamflow Statistics
NWP	Nationwide Permit
PFDS	Precipitation Frequency Data Server
PSIF	Wetting Front Capillary Suction
PMR	Physical Map Revision
RFE	Regulatory Flood Elevation
RGRCP	Rubber Gasket Reinforced Concrete Pipe
RTIMP	Percent Impervious
RUSLE	Revised Universal Soil Loss Equation
SFHA	Special Flood Hazard Area
SIC	Standard Industrial Classification
SS	State Standard
SSA	State Standard Attachment
SSURGO	Soil Survey Geographic Database
STATSGO	Soil Survey Geographic
SWMP	Stormwater Management Plan

Term	Description
SWPP	Surface Water Protection Program
SWPPP	Stormwater Pollution Prevention Plan
TDN	Technical Data Notebook
USACE	United States Army Corps of Engineers
USBR	United State Bureau of Reclamation
USFS	United States Forest Service
USGS	United States Geologic Survey
VCD	Vegetation Cover Density
WCMP	Watercourse Master Plan
WOTUS	Waters of the United States
XKSAT	Effective Saturated Hydraulic Conductivity
YCFCD	Yavapai County Flood Control District

1 Introduction

1.1 Purpose

It is the intent of the Town of Prescott Valley (Town) to have a comprehensive drainage management program that protects the health, safety, and welfare of its citizens, their property, and the environment. This document outlines the Town's philosophy on planning for drainage facilities, the federal and state regulations pertaining to such facilities, and the Town's drainage regulations, policies, and standards. This document is intended to be used in concert with the Hydraulics and Erosion Control Drainage Design Manuals for Maricopa County (referenced in this document as the Maricopa County Hydraulics Manual and the Maricopa County Erosion Control Manual), the Arizona Department of Transportation (ADOT) Highway Drainage Design Manual, Hydrology (referenced in this document as the ADOT Hydrology Manual), and the Drainage Design Manual for Yavapai County (referenced in this document as the Yavapai County Flood Control District (YCFCD) Drainage Design Manual). The objective of the Drainage Design Manuals for Maricopa County, the ADOT Highway Drainage Design Manual, Hydrology, and YCFCD Drainage Design Manual is to provide technical guidance for storm drainage facilities in the Town. These manuals provide a convenient source of analytical and design information. The Town may create design manuals specific to Prescott Valley in the future. In the meantime, the most current versions of the Maricopa County Hydraulics Manual, Maricopa County Erosion Control Manual, the ADOT Hydrology Manual and YCFCD Drainage Design Manual shall serve as the basic references for the Prescott Valley Uniform Drainage Policies and Standards Manual.

The Town reviews and approves drainage reports and plans for construction projects for general conformance with the Town's policies and standards. This notwithstanding, the Town does not assume liability for insufficient design or improper construction. The Town’s reviews and approvals do 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, promote public safety, and use stormwater as a resource to establish long-term water security. Compliance with the regulatory elements, policies, and design standards does not imply a guarantee that properties will be free from flooding or flood damage. The Town, its officials, or employees assume no liability for information, data, or conclusions prepared by private engineers, and makes no warranty expressed or implied in its review/approval of drainage projects or studies including stormwater quality submittals.

1.2 Application

Policies and standards set forth in this document apply to private development projects, to projects funded entirely by the Town, and projects funded in cooperation with the Town and/or other agencies. In an advisory capacity, these policies and standards also apply to state and state and federally-funded projects sponsored by the Town. It is understood that there may be exceptions to the policies that may be granted by the Town. The standards are minimum standards. More stringent standards may be required in the event that public health, safety, welfare, and long-term water security could be adversely affected by application of the minimum standard.

1.3 Background

In 2003, the Town entered into a contract with Claycomb/ Rockwell Associates, Inc., to assist the Town in developing a drainage criteria manual for the Town. It was recognized at that time that the Town did not have the resources to develop a totally stand-alone document that would be sufficiently extensive to meet the needs of the rapidly growing area. The decision was reached to use the extensive drainage design manuals that have already been developed by the Flood Control District of Maricopa County (FCDMC) and the ADOT as references and generate specific Uniform Drainage Policies and Standards for the Town.

The ADOT Hydrology Manual is already widely used as a reference by entities throughout Arizona, being the basic reference for the YCFCD Drainage Design Manual.

In 1998, Maricopa County started a collaborative effort with the City of Phoenix to meld their respective drainage design manuals. Various technical aspects of both the City and County's manuals required updating due to advances in the engineering science and further experience with applications unique to Maricopa County. Also, the "uniform policies and standards" identified in previous versions of the Maricopa County Hydraulics and Maricopa County Erosion Control Manuals were removed to allow the City of Phoenix and all other municipalities within Maricopa County the opportunity to have their own stand-alone policies and standards that address the unique conditions in their respective communities. The new manuals now only provide comprehensive technical methodologies for definition of flood and erosion hazards and for design of drainage facilities within both the unincorporated and incorporated areas of Maricopa County. Maricopa County's intent is that the Manuals be adopted as a part of each separate Uniform Drainage Policies and Standards Manual prepared and adopted by individual municipalities. Thus, the Town can avail itself of the extensive technical design information in the Manuals by adopting its own Uniform Drainage Policies and Standards, which references the Maricopa County Manuals. The user of these manuals is encouraged to routinely check the web-based version for updates since addenda will be issued as needed by this means. Maricopa County's website is www.maricopa.gov.

In 2024 the Town entered into a contract with WEST Consultants to update the manual to include the following:

- Updates to federal and state policies and standards typically found in drainage manuals throughout the state.
- Update referenced documents including new and relevant references.
- Modify manual language clearly identify state requirements versus suggestions.
- To remove barriers to best management practices.
- Used results from the recent Area Drainage Master Plan to set hydrology standards.
- Set standards regarding the use of two-dimensional models.

Information from *Area Drainage Master Plan, Hydrology and Hydraulic Modeling, Technical Support Data Notebook* was used to develop standards related to the use of the Green and Ampt Method for determining rainfall losses specific for the Town (See Section 6.3.4) (WEST Consultants, Inc., 2024).

As such, this manual update provides drainage policies and standards specific for the Town. The latest edition of the Drainage Design Manual for Maricopa County, Arizona, Volume II, Hydraulics; the Drainage Design Manual for Maricopa County, Arizona, Volume III, Erosion Control; the ADOT Highway Drainage Design Manual, Hydrology, and YCFCD Drainage Design Manual are incorporated into this manual by reference. Throughout this manual these reference documents are referred to respectively as the Maricopa County Hydraulics Manual, Maricopa County Erosion Control Manual, ADOT Hydrology Manual and YCFCD Drainage Design Manual.

1.4 Scope

The Town's Uniform Drainage Policies and Standards Manual is divided into seven chapters that address the major administrative areas of drainage and stormwater management. The intent of this Manual is to provide regulatory guidance for the design of drainage and stormwater facilities. Chapter 2, Planning, stresses the Town's vision for drainage and stormwater management while providing guidance for the planning process. The drainage and stormwater management policies provided in Chapter 3 build upon this vision and are supported by the Town's floodplain and drainage regulations. Federal and state regulatory requirements are outlined in Chapter 4. Town specific regulations are provided in Chapter 5. The standards, provided in Chapter 6, identify specific criteria for the design of drainage and stormwater facilities in conformance with the more general policies. Finally, Chapter 7 identifies the procedures for modifying policies and standards.

2 Planning

2.1 Purpose

Drainage and stormwater runoff facilities are an integral part of public infrastructure systems and should be planned as such. Drainage planning must be included in the formulation of both site-specific and regional drainage plans and all urban planning should be coordinated from the beginning with drainage as an integral part. Drainage master plans need to be carefully produced 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 urban considerations. Drainage planning should not be done after all the other decisions are already made as to the layout of a new subdivision, commercial or industrial area. It is this latter approach which creates drainage problems which are costly to correct.

Good planning results ultimately in the construction of adequately sized drainage facilities and a better community. Natural drainage ways and street drainage patterns should be coordinated to achieve the policies and design criteria standards presented in this Manual update. The construction and/or long-term maintenance costs for drainage and flood control measures are high when planning is poor. The quality of the planning significantly impacts the costs to the developer and Town. Furthermore, inadequate planning potentially affects residents and other infrastructure systems in terms of flood damages.

Supplemental or complimentary benefits and uses from drainage facilities should be considered. Both passive and active recreational uses are examples. Any effort made toward increasing local and community-wide benefits is appropriate and is encouraged. Also, the drainage facilities should be compatible with other Town plans, such as shade and recreation.

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

2.2 Planning Philosophy

Planning of drainage facilities should incorporate natural waterways into the development of a desirable and aesthetic design, rather than attempting to superimpose drainage works on a development after it is laid out. Channels and stormwater storage facilities that are designed as a focal point of the community minimize misuse (e.g., dumping) and encourage proper maintenance. This is also the best time to add green stormwater infrastructure. Planning considerations are included in Appendix B.

Drainage should be considered on the basis of two design phases. The first is the planning phase where master drainage plans are developed. The second is the final design phase, which encompasses detailed engineering using the first phase as the basis for the final design. The first phase is a more global view, as discussed herein, and results in the conceptualization of an overall drainage solution. The second phase is an extension of the first and it is here that the engineering details for the localized infrastructure are worked out.

A well-planned system will protect the development area from extensive property damage and loss of life episodes from flooding. This system is generally designed for the more severe and less frequent stormwater runoff, such as the 100-year return period. 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 flow downhill whether development and people are in its way or not.

2.3 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 at this point of the development process where the greatest impact can be made on the cost of drainage facilities. When flood hazards are involved, the planner must 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 and park areas, and more recreational opportunities.
5. Development of otherwise undevelopable land.
6. Opportunities for lower building construction cost.
7. Avoidance of flood damage claims and resultant litigation.
8. Avoidance of fines and fees levied for non-compliance with Federal and State National Pollution Discharge Elimination System (NPDES) Stormwater regulations.
9. Proper watershed function.
10. Groundwater recharge promoting long-term water security.
11. Passive irrigation of vegetated landscape treatments.

2.4 Types of Drainage Plans

Drainage plans can be divided into two types: master drainage plans and final drainage plans. Master drainage plans deal with the broad assessment of existing drainage conditions and development of conceptual solutions to drainage problems, either existing or induced by a project. Final drainage plans provide engineered solutions and details to support the final design of a project.

The Town completed an Area Drainage Master Plan (ADMP) in 2024 (WEST Consultants, Inc., 2024). The purpose of the plan is to examine the existing conditions of the study area for flooding areas of concern and to develop conceptual level mitigation solutions that leverage more sustainable and conservation-oriented solutions that support and/or compliment other water conservation objectives for the Town.

2.4.1 Master Drainage Plan (Regional)

On a watershed basis, regional master drainage plans, often called Area Drainage Master Studies & Plans (ADMSs & ADMPs), are prepared to identify areas of existing flooding problems and present potential alternative solutions. Solutions typically include an array of stormwater conveyance and storage alternatives, and non-structural or regulatory solutions. These plans are an excellent source for hydrology as sub-basin hydrographs are typically provided. The ADMS typically includes detailed hydrologic and hydraulic analyses and identification of flooding and erosion hazards within a major watershed area. Alternative solutions are identified, evaluated, and classified. The ADMP is typically a more detailed study, providing detailed analysis of selected alternatives recommended in the ADMS, and thorough evaluation of a final recommended alternative. The ADMP also provides guidelines for development within the study area with a focus on watershed management to implement a public safety strategy, and watershed components of any Watercourse Master Plans (WCMP) completed in the study area. A WCMP is similar to an ADMP, except that a WCMP has more of a focus on the management of a particular watercourse and associated flood and erosion hazard zones. No WCMPs have been completed within the Town at the time of adoption of this Manual.

2.4.2 Master Drainage Plan (Land Development)

Master drainage plans are required for land development 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 both the proposed development and the existing development. Master drainage plans for developments are required for large parcels of land, but the principles remain valid for all parcels regardless of size. The keys to master drainage plans for land developments are:

1. Determining the magnitude of flow both entering the property and created on the property.
2. Developing an approach to intercept this flow.
3. Identify a workable means of conveying the flow through the project.
4. Discharging to the downstream drainage network (whether natural or man-made) in a manner that matches existing conditions. Master drainage plans for land developments shall also identify locations for detention basins.

2.4.3 Final Drainage Plan

Final drainage plans provide engineered solutions and details to support the final design of a project. Here, the hydrology and hydraulics of the selected approach from the master drainage plan is further refined to apply to 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 primary difference between master drainage plans and final drainage plans is that master drainage plans are more conceptual and may assess more than one potential solution. The master drainage plan becomes a building block for the final drainage plan. The final drainage plan addresses details such as depth of flow in roadways and storm sewer geometry that are not generally fully defined at the master drainage plan level. Master drainage plans do not require detailed grading plans in order to develop conceptual drainage solutions whereas final drainage plans are prepared in concert with grading plans.

2.5 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 master drainage plan. Table 2-1 highlights some of the information available.

Table 2-1. Types of Available Drainage Information

Item	Source	Description
Flood insurance studies	Federal Emergency Management Agency (FEMA), Arizona Department of Water Resources (ADWR)	Watershed peak discharges, flood elevations, flood risk.
Area Drainage Master Plans & Studies (ADMP & ADMS)	Town, County	Watershed hydrographs and peak discharges, conceptual storage, and conveyance solutions.
Watercourse Master Plans (WCMP)	Town, County	Management of a particular watercourse and its associated flood and erosion hazards.
Studies & plans from existing flood control projects	Town, U.S. Army Corps of Engineers (USACE), U.S. Bureau of Reclamation (USBR), Natural Resources Conservation Service (NRCS)	Examples: Navajo Wash, Mission Lane Outfall, Downtown PV Master Drainage Study.
Transportation Plans & Studies	ADOT, Yavapai, County, Town	Corridor studies address existing and proposed drainage conditions. Plans depict drainage improvements.
Land use zoning maps	Town, Yavapai County	Provides insight to future runoff characteristics. Zoning may limit type of drainage solution.
Soil maps	NRCS & US Forest Service (USFS)	Identifies runoff characteristics and engineering limitations.
Aerial Photography	Town, County, public & private	Identifies watershed and existing land use characteristics.
Topographic mapping	Town, County, public & private	Used to determine watershed boundaries, slopes, and water-course hydraulic characteristics.
American Land Title Association (ALTA) Surveys	Yavapai County Recorder's Office	Land ownership, boundary & utility easements (if available).
Drainage plans from adjacent developments	Town, County, Land Developer, Homeowners Assoc.	Depicts existing or proposed conditions for adjacent properties that may affect the site under study.

2.6 Master Drainage Planning Process

2.6.1 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 and 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. Ultimate design should minimize long-term maintenance requirements.

2.6.2 Waters of the United States

Plans that impact Waters of the United States (WOTUS) must be permitted through the USACE. For most areas under study, jurisdictional waters exist. The professional undertaking a master drainage plan must have knowledge of Section 404 of the Clean Water Act (CWA) requirements (See Chapter 4, Regulations) to apply to the planning objective or have the jurisdictional waters delineated prior to delving too far into the master drainage process. It is likely that the jurisdictional waters will have a significant impact on the overall drainage plan and on-going maintenance activities.

The Town would like to see long continuous stretches of jurisdictional waters undisturbed as opposed to a checkerboard or series of undisturbed/disturbed/undisturbed portions of jurisdictional waters. It will be easier for the Town to manage these required undisturbed areas, keeping them continuous.

2.6.3 Regulations, Policies, and Standards

All master drainage plans must be done in conformance with Town regulations (Chapter 5), policies (Chapter 3), and standards (Chapter 6). These set the guidelines for all infrastructure projects, be they public or private.

2.6.4 Linear Open Space

The Town encourages drainage facilities that combine flood control with recreation uses and open space. Natural or semi-structural drainage corridors can be developed with landscaping and multi-use paths incorporated into the drainage design to provide recreation opportunities. This concept can be applied to new drainage channels and stormwater basins that are utilized for recreation uses as well as existing open channels or natural watercourses that currently do not provide recreation opportunities. The multi-use paths should be located above the channel banks to avoid impacting WOTUS, minimize effects of erosion, minimize interaction with nuisance flows, and to minimize maintenance requirements. The Town stresses the establishment of natural or semi-structural drainage facilities (native vegetation lined flatter sloped channels - not riprap or concrete lined) when planning linear open space drainage corridors. Examples of natural or semi-structural drainage facilities are included in Appendix B. Utilizing natural corridors to accommodate stormwater is the Town 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. Native 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.
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.
9. Decreasing the cost of maintenance.

2.6.5 Stormwater Storage/Detention

All developments, except single-family residential or multi-family residential of 4 units or less on individually platted lots or located in areas where regional detention has been approved through the adoption of a master plan, shall be required to provide on-site detention of storm water.

Wherever possible, storm water detention should be implemented on a regional basis. The storm water detention program should utilize regional detention based on a watershed-wide assessment of the effects of urbanization and planning and development of facilities at the most effective locations to minimize those effects. Such a watershed-wide assessment should include an evaluation of the cumulative effects of urbanization such that the implementation of the storm water detention program addresses both localized increases in runoff and regional effects to the extent possible. Where such a plan can be implemented, on-site retention should be avoided.

There may be an occasion to utilize a storm water storage basin that is not designed for detention purposes. For example, a small open basin located at the inlet of a box culvert underneath a street where the inlet flow into the small basin is a narrow open channel or piped flow through a constricted right-of-way. Such a basin can be constructed to temporarily reduce velocity (allowing sediment and pollutants to settle out, improving the water quality of the ultimate discharge runoff) as well as provide access for maintenance. Such a basin could also provide a local area of open space for the neighborhood. These basins are encouraged as an active part of the Town's Stormwater Management Plan (SWMP) process (refer to Section 4.10).

In the planning process drainage corridors and stormwater storage/retention basins should be combined with open space, parks, and paths to create focal points for the community rather than isolated tracts. These combined uses should augment the Town parklands whenever feasible. The benefits of this approach are an enhanced sense of community, increased open space with landscape amenities, and decreased crime. The Town encourages combined use of drainage and recreation facilities on both public and private lands. It is recommended that these drainage facilities be non-geometrically (more natural shape with smoother curves and not square with regular slopes) shaped. The design of storm water detention facilities should also be coordinated with the Town to assure compliance with water quality standards (see Chapter 4).

Given the demand for open space and organized sports fields such as soccer and ball fields, basins should be used for more than one purpose. Avoid siting recreational facilities at the very bottom of stormwater basins. These basins should be designed with tiers, gentle slopes or interior channels to allow for the collection of nuisance water and conveyance to allow for dry open space or field areas under normal and lower flow conditions.

The desired location for stormwater basins is adjacent to parks to increase the open space. The acreage of stormwater basins inundated by the 10-year or smaller storm event runoff may not be included in determining whether the park site meets the minimum acreage standards for Town park classifications. Integrating non-geometric basins into park design is encouraged for both active and passive recreation purposes, subject to meeting Town aesthetic and safety standards. All final determinations of whether or not any stormwater facility may be considered park or open space acreage will be based on Parks & Recreation Commission and Town Council acceptance and approval.

2.6.6 Zoning

Zoning often dictates the nature of watercourse development and outlines 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, 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 an 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 master 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. Generally zoning requirements will take precedence over other stormwater regulations when they are more restrictive in nature.

2.6.7 Design Hydrology and Hydraulics

At the master plan level, the drainage engineer shall concentrate on quantifying off-site flows that may impact the project and determine the means for conveying the flow through the project site. Review of the Town ADMS and/or ADMP that encompasses the project area, if available, provides the design team with valuable information pertaining to the magnitude of stormwater discharges and volumes affecting the project. The FEMA Flood Insurance Rate Maps (FIRM) should also be reviewed to establish if regulated floodplains cross the project.

An accurate estimate of the design peak discharge is necessary to approximate the channel or drainage structure capacity and size. The improvements presented in a master drainage plan shall not adversely impact adjacent property owners. In other words, flood stage levels, peak flows, and velocity cannot be increased or relocated off- site.

For the master drainage plan, on-site hydrology is typically performed to estimate storm water detention requirements and the approximate layout of storm sewers, street runoff and open channels. Here, drainage divides are oftentimes set consistent with existing topography since the desired final grades have yet to be determined. This is a reasonable assumption since earthwork costs become significant to appreciably change direction of slope from the existing direction.

2.6.8 Flood Hazards

Master 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 must be considered. In addition, the lateral migration of watercourses may threaten public health and welfare, unless proper erosion hazard zones are provided, prohibiting development in these areas. The Town policies regarding definition of erosion hazard zones in riverine areas, and accompanying residential development guidelines, are to 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 FCDMC's Piedmont Flood Hazard Assessment for Flood Plain Management for Maricopa County, Arizona (Hjalmarson, 2003), most current version and Alluvial Fan Flooding (National Research Council, 1996) and FEMA guidelines (see Section 3.7.4) when master planning on alluvial piedmonts. Finally, ponding areas upgradient of elevated roads, railroads, and irrigation canals must be considered during the development of the master drainage plan.

2.6.9 Safety

A basic tenet of any capital improvement project is the promotion of public safety. Public safety must be a consideration taken throughout the development of a master drainage plan. Excessive stormwater depth and/or velocity poses a threat to safety and public health. See Section 6.2 for more information.

2.6.10 Cost

During the development of a master plan, initial capital costs and long-term maintenance costs must 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 Town's policies and standards.

2.7 Master Drainage Plan Considerations

2.7.1 Open Channel Conveyance

The alignment of a drainage system is often set by following the natural watercourse flow line or low flow channel. In these cases, the alignment is a more straightforward matter, and essentially need only be defined on mapping. In many areas about to be urbanized natural channels 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 minimize or possibly eliminate the need for an underground storm drain system. Where WCMPs 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 the Maricopa County Hydraulics Manual, Chapter 11. Detailed lateral migration and long-term erosion analysis would be performed as part of final design in those circumstances.

In many urbanized areas, there is no existing well-defined watercourse, or the watercourse has been filled and built upon. In this instance, the master plan must establish channel alignment based first on site geomorphology, and second on site development considerations.

The master 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 upon Town Standards, good hydraulic practice, environmental design, 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 already exists and can be effectively utilized with considerations to erosion protection, setbacks, and the 100-year flooding limits.

A more natural approach is preferred. The ideal channel is an undisturbed one. The benefits of such a channel are:

- Velocities are usually lower, resulting in a longer time of concentration and lower downstream peak flows.
- Natural channel and overbank floodplain storage tends to decrease peak flows.
- Maintenance needs are usually less than artificial channels because it generally is in dynamic equilibrium with the natural erosion/sedimentation process.
- The channel provides desirable open space and recreational area adding significant social benefits. The closer an artificial channel character can be made to that of a natural channel, generally the better will be the artificial channel.

For a master plan, the level of analysis necessary to establish channel widths varies. If the channel is for a watercourse with a 100-year peak discharge of 500 cubic feet per second (cfs) or greater, a detailed floodplain analysis may be required. This is also dependent upon the existing/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 master plan level is the transitioning of flow into and out of a proposed channel. As identified herein, a key Town policy 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 existing conditions. Interceptor channels may be required to funnel offsite flow into an onsite or in-line channel. Similarly, spreading basins or 4:1 channel expansions may be necessary to transition from an artificial channel to the existing downstream floodplain.

2.7.2 Storage/Detention

The master plan is where decisions need to be made on the use of stormwater storage/detention basins/infiltration and their location. The sitting of basins 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 helps to improve the quality of water downstream, however clear water flow can exacerbate erosion.

For preliminary sizing of stormwater basins, a storage per unit area relationship along with a safety factor can be utilized to derive an approximate stormwater volume for storage. The storage per unit area is primarily dependent upon the land use of the proposed project within the contributing drainage area and on the design rainfall depth for the area in question.

2.7.3 Environmental Protection

As explained in the Regulations Section (Chapter 4), there are numerous federal, state, and municipal regulations that must be adhered to during plan development and implementation. At the federal and state level, Section 404 (WOTUS) and Section 401 (water quality) of the Federal CWA, 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 Town, the plan must comply with 40 Code of Federal Regulations (CFR) §122, stormwater quality as well as this Uniform Drainage Policies and Standards Manual. Specific requirements may also be found in Section 402 (p)(3)(B) of the Federal CWA and Arizona Administrative Code (AAC) Title 18, Chapter 9, Article 9 & 10 of the Arizona Stormwater Regulations. Taking the requirements of these regulations into account during the development of the master drainage plan will streamline the final design and implementation process. For example, recognition of the trigger points in Section 404 of the CWA permitting will provide guidance in developing mitigation plans. The Town strongly endorses minimizing disturbances to natural watercourses to lessen the impacts on ecology. See Policies 3.3.1 and 3.3.3.

2.8 Final Drainage Plan Considerations

With the drainage system conceptualized in the master drainage plan, attention to the remainder of the project area can be given relative to localized drainage concerns. For land development projects, maintaining watershed boundaries defined during the master planning effort minimizes earthwork and storm sewer expenditures. Such an approach also supports the basis for preliminary stormwater storage/detention design.

The master drainage plan serves as the framework for final design. A thorough master 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, if needed. 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 Town. During final design, the design engineer applies the policies and standards of the Town to minimize long-term maintenance of the drainage improvements while accommodating safety concerns.

A Stormwater Pollution Prevention Plan (SWPPP) is developed and submitted with the final design to the Town for approval. Prior to the start of construction, the design engineer or developer is responsible to submit a Notice of Intent (NOI) based on the SWPPP to Arizona Department of Environmental Quality (ADEQ) while a Notice of Termination (NOT) to ADEQ is required at the completion of construction, all in accordance with current ADEQ policies and standards.

Any Section 404 of the CWA permit application to the USACE or Letter of Map Amendment (LOMA)/ Letter of Map Revision (LOMR) application to FEMA necessary for the approval of any drainage construction must be copied in full to the Town per Town standards for development infrastructure submittals.

Terminology and labeling of features within and between the master drainage plan, final drainage plan and construction plans must be consistent. This consistency must also hold between any digital and hard copy models and reports. This is necessary for an efficient and accurate review.

3 Policies

3.1 Purpose

It is the intent of the Town to have a comprehensive stormwater drainage program that protects the health, safety and welfare of its citizens, their property, and the environment. The Town's stormwater drainage program documents include the latest revisions of the following:

- Drainage Regulations for the Town of Prescott Valley
- Prescott Valley Municipal Code, Chapter 12, Flood Damage Prevention
- Prescott Valley Stormwater Management Plan
- Prescott Valley Uniform Drainage Policies and Standards Manual
- Highway Drainage Design Manual -Hydrology, ADOT
- Drainage Design Manual for Maricopa County -Hydraulics
- Drainage Design Manual for Maricopa County -Erosion Control
- Drainage Design Manual for Yavapai County

The Town has adopted floodplain management and stormwater drainage policies that set forth guiding principles for the stormwater program. These drainage policies fall under the following categories:

- Planning
- Drainage Character
- Hydrology
- Water Quality
- Floodplain Management
- Erosion Hazard Management
- Street Drainage
- Conveyance Facilities
- Storage Facilities
- Maintenance
- Erosion Control
- Permitting

These policies, together with the storm water program documents listed above, define the criteria and procedures to be used for stormwater management and drainage design in the Town.

3.2 Planning

Proper planning and design of drainage facilities is as important as that for water, wastewater, streets, and other infrastructure needs in a growing community. The following are the Town policies related to drainage planning.

A Master Drainage Plan/Final Drainage Plan, as described in Chapter 2, is required for all developments 40 acres in area and larger. This Master Drainage Plan shall be coordinated with the Town Master Drainage Plan, any ADMPs or WCMPs prepared by the YCFCD that also address the development or are located close enough to be a factor, and any other master drainage plan of an adjacent or nearby property.

For developments smaller than 40 acres the Master Drainage Plan/Final Drainage Plan approach is highly recommended. However, the Master Drainage Plan (MDP) portion may be omitted provided the Final Drainage Plan has incorporated those MDP Planning steps required by the Town.

3.3 Drainage Character

The provision for facilities to convey stormwater runoff is a necessary part of land development activity. In the natural environment, stormwater runoff will determine its own course. Land development may require alteration of the natural alignment of a drainage system. This may result in realigned flow paths, larger peak discharges, greater volume of runoff, higher water surface elevations, increased flow velocities and other drainage modifications which can adversely impact adjacent property owners.

Policy 3.3.1: Historic Drainage Patterns. Historic drainage patterns shall be maintained, to the extent possible, within practical and economical constraints.

Policy 3.3.2: Drainage Entering and Exiting a Property. Drainage improvements shall not adversely change water surface elevations, flow velocities and locations where runoff enters and exits a property being developed, such as the concentration of sheet flows or braided washes, without Town approval. Such drainage activities shall require an engineering report that substantiates there are no adverse impacts.

Policy 3.3.3: Disturbances to Natural Watercourses. Disturbances to natural watercourses shall be minimized in order to lessen the impacts to riparian vegetation, wildlife habitat, and jurisdictional areas (as described in the document National Flood Insurance Program (NFIP) Floodplain Management Requirements, FEMA 480, Unit 1, Section A, Natural and Beneficial Floodplain Functions) (FEMA, 2005). Section 404 of the CWA states that jurisdictional areas to be left undisturbed shall be congregated together in a continuous length as much as possible.

Policy 3.3.4: Town of Prescott Valley Standards. Any facility or structure that will be located within a watercourse, drainageway, or other means of conveying or storing stormwater shall be designed and constructed to Town Standards, as well as other Federal, state, and local regulations, including local acts, codes, laws, regulations, ordinances, standards, and policies.

3.4 Hydrology

Hydrology addresses surface water and the estimation of peak discharges, volumes, and time distributions, which result from precipitation.

Policy 3.4.1: Estimation of Peak Discharges and Runoff Volumes. The following is the preferred order of hierarchy for obtaining peak discharges and runoff volumes for various floodplain and drainage design purposes:

1. For the first choice the developer should use the accepted peak discharges and runoff volumes of record from *The Town of Prescott Valley Master Drainage Plan*, ADMS/ADMPs, or flood insurance studies where available and if approved by the Town for use on the project. All such information from these various sources should be evaluated to determine if the assumptions made are still valid and appropriate for the intended purpose, and accepted by the design engineer of record, per Section 1.1 of this document. 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 this Manual.

2. The second choice is the drainage plans and design report from adjacent properties. This information may be used where available and if approved by the Town.

3. If choices 1 and 2 are not available, or are deemed inappropriate, then peak discharges and runoff volumes shall be calculated in accordance with the procedures in this Manual.

3.5 Stormwater Quality

Stormwater quality programs are being implemented at the federal and state levels requiring the Town to regulate and manage a stormwater system to limit, to the maximum extent practicable, the discharges of pollutants from the Town's storm drain system. The Town's SWMP was developed and implemented to fulfill requirements of stormwater discharges from a small Municipal Separate Storm Sewer System (MS4) in accordance with Section 402 (p)(3)(B) of the Federal CWA, and Arizona Stormwater Regulations (AAC Title 18, Chapter 9, Articles 9 and 10). This SWMP was developed to comply with the requirements and conditions of Parts IV and VI of the general permit: Arizona Pollutant Discharge Elimination System (AZPDES) permit AZG2002-002.

Policy 3.5.1: Discharge of Pollutants. No person or entity may cause the discharge of pollutants into a natural drainage system, or a public storm sewer system or facility.

Policy 3.5.2: Pollutants on the Land Surface. Pollutants released to the land surface that subsequently become a constituent of stormwater runoff are considered a discharge of pollutants [1].

[1] As of 11/30/01, excludes certain activities such as not-for-profit washing of vehicles, non-agricultural irrigation water discharges, fire hydrant/potable water system flushing, dust control watering, and discharge of residential evaporative cooler/air conditioning condensate. Since the federal regulations pertaining to this matter change periodically, the practitioner should review the Town Regulations for revision.

Policy 3.5.3: Soil as a Pollutant. Soil is considered a pollutant when it is entrained in stormwater runoff from construction sites in quantities greater than natural conditions.

Policy 3.5.4: Erosion Control. Erosion control measures for new developments shall be in conformance with the Maricopa County Erosion Control Manual.

Policy 3.5.5: Stormwater Pollution Prevention. Stormwater Pollution Prevention is to be addressed through the use of Best Management Practices (BMP) to the maximum extent practicable to comply with federal, state, county or local regulation or ordinances.

Policy 3.5.6: First Flush. The minimum level of control for new development at which stormwater pollution prevention practices must be put in place is called the "first flush." This consists of retaining or treating the first 0.5 inches of direct runoff from a storm event. Normally, this minimum level of control is met by following the Town's requirement. Any alternative system that is proposed for reducing stormwater pollutants must be approved by the Town.

The Town encourages the use of green infrastructure and low impact development techniques in concert with these stormwater policies to improve water-harvesting potential, improve water quality, and reduce the impacts of increased runoff downstream. To support this goal Appendix B Green Stormwater Infrastructure includes designs and site guidance considerations. These green infrastructure techniques may be used to meet the first flush volume requirements if equivalent volume and performances are achieved.

3.6 Floodplain Management

The Town participates in the NFIP which provides flood insurance to its citizens, and 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 the Town to continue as a member of the NFIP.

The Town 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 Federal regulations governing floodplains and drainage design. Erosion and sedimentation hazard management is an integral part of floodplain management. Policies are also established to manage erosion and sedimentation hazard areas in a uniform and consistent manner.

3.6.1 FEMA

FEMA is an agency of the federal government under the Department of Homeland Security. 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.

Policy 3.6.4.1: Best Available Technical Information. New or updated information for FEMA defined floodplains and floodways is constantly being prepared, both by the Town and by others. It is the Town's policy, in conformance with FEMA Guidelines, to use this information for regulatory purposes and to provide it to the public as the "Best Available Technical Information." However, until the effective FIRM is revised, the requirements from the effective FIRM will also be used. Examples of "Best Available Technical Information" are as follows:

1. New studies that have not yet been submitted to FEMA. This information is usually from studies that are in progress but could also be completed studies that are being held pending further investigations such as completion of an ADMS, ADMP or WCMP. This information may be shared with the public if appropriate and approved for release by the Town. It will be stamped preliminary, and the recipient will be notified that the information is subject to change and is used only at risk. This information may be used for regulatory purposes, particularly if the floodplain and/or floodway widths or 100-year water surface elevations exceed those of the effective FEMA Flood Insurance Study (FIS).
2. New studies that have been submitted to FEMA but not yet approved. The same conditions from item 1 apply here. The effective FEMA FIS will be used for regulatory purposes for all other cases.
3. Floodway delineation in a new study prior to submittal to FEMA.
4. The effective FEMA FIS will be used for regulatory purposes for all other cases.

Policy 3.6.1.2: Development in FEMA Floodways. No development shall be allowed in a FEMA Regulatory Floodway, including, but not limited to fill, new construction, substantial improvements, and other development, 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 one (1) foot.

Policy 3.6.1.3: Changes or Proposed Improvements within Floodplains. Any change or proposed improvements, including, but not limited to, public and private roads, within a mapped FEMA floodplain or special flood hazard area (SFHA) shall be required to follow FEMA submittal procedures for map amendments or revisions. An approval of a LOMA or a LOMR from FEMA is required before any change to or improvements within a regulatory FEMA floodplain or SFHA will be authorized by the Town Also, an approval of a Conditional Letter of Map Amendment (CLOMA) or Conditional Letter of Map Revision based on Fill (CLOMR-F) from FEMA is required prior to a grading permit will be issued by the Town for development.

Policy 3.6.1.4: Subdivisions and Other Proposed Developments Greater than 50 lots or 5 acres, Whichever is Lesser: Subdivisions and other proposed developments greater than 50 lots or 5 acres, whichever is lesser, and planning to submit a Conditional Letter of Map Revision (CLOMR) for modification of a FEMA-designated floodplain and/or floodway, must receive Town approval and submit the CLOMR request to FEMA before a grading permit will be issued by the Town for the development.

Subdivisions and other proposed developments greater than 50 lots or 5 acres, whichever is lesser, that have submitted a CLOMR to FEMA for modification must receive FEMA-approved LOMR (the effective date contained on the LOMR) before final approval by the Town is granted for building occupancy for the development. LOMRs are to be submitted within six months following completion of the development (44 CFR §65.3).

Policy 3.6.1.5: Scour Protection for Utilities. Underground transmission lines (example: electrical, Natural Gas, Gasoline, Oil, fiber optic, cable, water, sewer) shall be protected against scour within the SFHA. The scour depth is to be designed by a civil engineer registered in the State of Arizona.

Policy 3.6.1.6: Watercourse Master Plan Requirements. Where a WCMP has been completed, the adopted plan for erosion setbacks, structural and non-structural measures, existing and/or future condition floodplain and floodway requirements, and development guidelines shall be followed. Where a WCMP is in progress, or is slated to be undertaken within one (1) year, as identified by the Town, watercourses shall remain undisturbed within the 100-year floodplain limits until the WCMP is completed and adopted by the Town.

3.6.2 Non-FEMA

There are many flood prone areas in the Town that do not have regulatory floodplain, floodways, or SFHA established by FEMA.

Policy 3.6.2.1: Lowest Floor Elevation Requirement. Lowest floor elevation for houses and other buildings shall be elevated at least one (1) foot above the 100-year water surface elevation. More stringent requirements may be in place in the Development Guidelines from an ADMS/ADMP/MDP for a particular area.

Policy 3.6.2.2: No Increase in Runoff. New developments shall not increase runoff onto streets outside of the development.

Policy 3.6.2.3: Lot Grading. Lots shall be graded so as not to adversely affect adjacent property owners. Runoff may drain onto or through adjacent property if a dedicated right-of-way(s) or privately owned drainage tract(s) is provided, and such grading schemes promote undisturbed watercourses or use historic runoff patterns. A legal description and exhibit drawing of every easement, sealed by a land surveyor registered in the State of Arizona, must be included as a part of the recorded documents.

Policy 3.6.2.4: Requirement to Delineate 100-year Base Flood Elevation. In locations where a FEMA regulatory base flood elevation (BFE) does not exist and the 100-year discharge exceeds 500 cfs, a BFE shall be established using the standards and procedures in the ADOT Hydrology and Maricopa County Hydraulics Manuals and shall require Town approval.

Policy 3.6.2.5 Variances. Any variances to the Town's floodplain policies shall require approval and meet the requirements per Prescott Valley Floodplain Management Ordinance Article 12-06.

3.7 Erosion Hazard Management

3.7.1 Riverine Areas

Policy 3.7.1: Riverine Erosion Hazard Zones. The guidelines set forth in ADWR, State Standard for Watercourse System Sediment Balance, State Standard (SS) 5-96 (ADWR, 1996) shall, as a minimum, apply to all watercourses identified by FEMA as part of the NFIP, all watercourses with drainage areas more than ¼-square mile, or a 100-year peak discharge estimate of more than 500 cfs, as estimated using the procedures in the ADOT Hydrology and Maricopa County Hydraulics Manuals and all watercourses identified by the Town as having significant potential flood hazards. At a minimum, the guidelines apply to:

- Structures that could fail or incur significant damage as a result of erosion or deposition.
- Proposed structures that, if built, could result in adverse impacts to adjacent properties.
- Watercourses that do not have erosion hazard zones approved by the Town.
- Watercourses within existing or proposed subdivisions, including residential and non-residential.

Watercourses identified by the Town as having significant potential flood hazards. Erosion setbacks consistent with SS 5-96 ADWR, which is a three-level approach, will be required for all properties developed in which the watercourses are to be left in an undisturbed state. Level I is a simplified approach and is only applicable to watersheds with an area of less than 30 square miles. The Level 1 approach in general provides a conservative estimate for erosion setback. Depending on the geomorphic conditions of the area, if the erosion limits are suspected by the Town to exceed those estimated using a Level I analysis, as defined in SS 5-96 ADWR, then a Level II or Level III analysis may be required. 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 Town 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 SS 5-96 ADWR and Maricopa County Hydraulics Manual Chapter 11 Sedimentation.

3.7.2 Distributary Flow Areas

Policy 3.7.2.1: Watercourse Stability Analysis. Stability of the watercourse divergence point(s) and divergent wash(es) should be determined prior to the approval of a proposed structure.

Policy 3.7.2.2 Proposed Watercourse Alterations. Proposed modification should not disturb the natural divergence location(s), especially if upstream, downstream, or adjacent parcels may be adversely impacted.

Policy 3.7.2.3 Erosion Hazard Zones. Erosion hazard guidelines (see Chapter 11 of the Maricopa County Hydraulics Manual) should be applied to all divergent watercourses adjacent to the proposed structure.

3.7.3 Sheet Flow Areas

Policy 3.7.3.1: Sheet Flow Area Erosion Hazard Zones. The guidelines set forth in ADWR, State Standard for Identification of and Development Within Sheet Flow Areas (SS 4-95) (ADWR, 1995) shall, as a minimum, apply to all watercourses identified by FEMA as part of the NFIP, all watercourses with drainage area more than ¼- square mile or a 100-year estimate flow of more than 500 cubic feet per second.

Policy 3.7.3.2: Vegetation Removal and Flow Concentration. Erosion potential directly relates to vegetation removal and concentration of flows. Proposed development should limit vegetation removal and concentration of flow to a minimum, especially in undisturbed natural conditions.

Policy 3.7.3.3: Single-lots. Flows will not be concentrated beyond the typical shallow swale around the structure. These swales should daylight and broaden to the original sheet flow conditions on the downstream side of proposed structures. Erosion protection may be required.

Policy 3.7.3.4: Subdivisions. The subdivision drainage design should focus on limiting the concentration of flows to the absolute minimum condition. Where flows are concentrated, appropriate scour protection should be applied to the channelized reach. Concentrated flows shall be returned to the natural sheet flow condition prior to exiting the property.

3.7.4 Alluvial Fan Areas

Policy 3.7.4.1: Alluvial Fan Area Erosion Hazard Zones. FEMA's most current guidelines on alluvial fans shall be followed to identify the active alluvial fan areas.

Policy 3.7.4.2: Erosion Hazards Zone Identification. The identified active alluvial fan areas are erosion hazards zones. For the inactive alluvial fan areas, the lateral-erosion hazard zone procedures in Maricopa County Hydraulics Manual shall be followed to identify the erosion hazard zones.

Policy 3.7.4.3: Deposition Hazard Zone Identification. The identified active alluvial fan areas are also deposition hazards zones. The active alluvial fans are subject to both erosion and deposition hazards due to the great uncertainty of flow paths.

Policy 3.7.4.4: Active Alluvial Fan Hazard Mitigation. The possible mitigation methods for active alluvial fan hazard zones are detention basins at the fan apex, open channels as floodway corridors, diversion channels, etc.

3.8 Street Drainage

The primary purpose of streets is to serve transportation needs. Accommodation of street drainage is provided so that motorists and emergency vehicles have a reasonable level of access and safety during storm events. Stormwater flowing within or across a street shall be managed in accordance with the following Town policies. Street design may also consider green stormwater infrastructure practices including stormwater harvesting basins, sediment traps, and curb openings. See Appendix B for more information.

Policy 3.8.1: No Adverse Impact. Street design shall not increase peak discharge and flow velocities onto adjacent properties. Street design shall be such that adjacent property drains into the street.

Policy 3.8.2: Safety. Streets shall be designed to convey stormwater runoff so as to provide motorists, pedestrians, cyclists, and emergency vehicles access and safety during a storm event. See Standard 6.4.3

Street Capacity regarding allowable width of spread and depth of flow. Also see Minimum Design Storm Criteria which includes the requirement to retain the stormwater from a 100-year, 2-hour duration storm.

Policy 3.8.3: Standards. Streets shall be designed to convey stormwater in conformance with Town Drainage Standards.

Policy 3.8.4: Velocity. Street flow velocities in excess of those established in the Town Drainage Standards shall require approval.

Policy 3.8.5: Inverted Crowns. Inverted crown streets or alleys shall not be permitted.

Policy 3.8.6: Local Streets. Arterial and collector streets shall not direct drainage onto local streets.

Policy 3.8.7: Culverts and Bridges. Culverts or bridges shall be provided for all streets which cross open channels or drainage ways. The depth of flow on or across a street at the location of a culvert or bridge shall be in accordance with Standard 6.4.3.

3.9 Conveyance Facilities

Stormwater conveyance facilities (drainage ways) can be open channels, undisturbed watercourses, ditches and swales, streets, culverts, or storm drains. These drainage ways are classified as follows:

- Lot to Lot Drainages: Serve very small watershed areas up to 2.5 acres or serving 6 lots of a subdivision.
- Small Drainage:Serve watershed areas from 2.5 acres to 40 acres and normally include drainage conveyances associated with subdivision development.
- Minor Drainages:Serve watershed areas from 40 acres to 160 acres and normally include the larger drainage conveyances associated with subdivision development.
- Major Drainages: Serve watershed areas from 160 acres to 10 square miles. These conveyances include natural and man-made channels, conduits, and washes.
- Regional Drainages:Serve watersheds greater than 10 square miles. These conveyances include natural and man-made channels, washes, and rivers.

Policy 3.9.1: Review. Drainage ways shall be reviewed for conveyance capacity and erosion/ sedimentation considerations in accordance with the Town Drainage Standards and the ADOT Hydrology, Maricopa County Hydraulics and Maricopa County Erosion Control Manuals.

Policy 3.9.2: Hydraulic Structures. All hydraulic structures shall be constructed in conformance with Chapter 6 of this document. 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.

Policy 3.9.3: Acceptance of Existing Structures/Facilities. Prior to the acceptance by the Town, to incorporate existing structures and/or facilities for maintenance, such structures and/or facilities shall be refurbished for the intended life cycle and constructed or reconstructed in accordance with the Town

Standards. 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.

Policy 3.9.4: Erosion/Sedimentation Analyses. The designer of drainage facilities shall undertake the appropriate level of erosion/sedimentation analysis commensurate with the risk of undesirable consequences expected to the health, safety, and welfare of the general public from the capital improvement. Design water surface elevations plus freeboard (FB) shall be at or below adjacent natural ground.

Policy 3.9.5: Levees and Berms. Levees or berms shall not obstruct side or interior drainage to a channel. The use of levees and berms are only allowed as public projects, with maintenance oversight of a governmental agency.

Policy 3.9.6: Irrigation Canals. Irrigation canals shall not be used as an outfall for stormwater runoff.

Policy 3.9.7: Dedicated Tract. A Town-owned property, dedicated right-of-way, or privately-owned drainage tract shall be provided and must accommodate access for maintenance for all drainage facilities maintained by the Town.

Policy 3.9.8: Siphons. Siphons are not allowed.

Policy 3.9.9: Landscape Character. All channels shall be designed to blend into the surrounding landscape to the greatest reasonable extent possible. Native wildflower/grass revegetation of channels, with or without turf reinforcement mats, and with side slopes 3:1 or flatter is preferred over riprap channels. Concrete or shotcrete lined channels are discouraged.

Policy 3.9.10: Floodwater Conveyance During Construction. Flood water conveyance must be provided at all times during construction in such a manner as to not increase flood depths, sedimentation, or erosive velocities above pre-construction levels for the areas adjacent to, and downstream of, construction projects.

3.10 Stormwater Storage/Detention Facilities

Land development can convert natural pervious areas to impervious or otherwise altered surfaces. These activities may cause an increase in runoff volume and/or peak discharge. The temporary storage/detention of storm water runoff can decrease downstream peak discharges and associated impacts to drainage infrastructure. Storage, in this section, refers to both detention as well as in channel storage areas not designed or constructed as formal detention but as water quality facilities per Section 2.6.5. Technical guidelines regarding the design of stormwater storage facilities is included in Section 6.9. The facilities may consider green stormwater infrastructure within the design and shown in Appendix B.

Policy 3.10.1: Stormwater Detention for Developments. All developments, except single-family residential or multi-family residential of four units or less on individually platted lots or located in areas where regional detention has been approved through the adoption of a master drainage plan, shall be required to provide on-site detention of stormwater outfitted with recharge enhancement technology in accordance with the Town's Standards. Stormwater detention is not required, but not prohibited, for single (un-subdivided) residential parcels equal to or greater than one (1) acre in area.

Policy 3.10.2: Regional Stormwater Detention. Wherever possible, stormwater detention should be implemented on a regional basis.

Policy 3.10.3: Maintenance and Ownership. Aesthetic maintenance (weeds, trash, etc.) of regional stormwater storage areas shall be the responsibility of the underlying property owner. Maintenance of the constructed regional facility's functionality shall be the responsibility of the Town when formally dedicated to the Town. All maintenance (weeds, trash, etc. as well as ensuring complete draining within 36-hours) of individual detention facilities on private property shall be the responsibility of the underlying property owner.

Policy 3.10.4: Multi-Use Features Requirement. Stormwater storage areas in residential developments shall incorporate multi-use features and be graded with varying side slopes/land features to be aesthetically pleasing while accommodating safety features per the Town Standards. Stormwater storage and conveyance facilities must consider multiple use opportunities. A multi-use plan must be approved by the Town prior to the issuance of a grading permit. Aesthetics as well as functionality must be considered in the design of stormwater storage and conveyance facilities. While combining uses -- such as flood protection and stormwater management with passive/active open space -- is encouraged and considered beneficial to the community, placing constructed recreational facilities (particularly playgrounds for children) at the low point of stormwater storage basins should be avoided. If multiple use and recreation opportunities are part of the overall project goals, these basins should be designed with terraces, a defined low flow, and gentle side slopes to allow for the collection of nuisance water and conveyance around fields, while keeping designated play areas safe from inundation during higher frequency rainfall events, such as the one or two-year storm.

Policy 3.10.5: Landscape Character. All stormwater storage facilities may be designed to blend into the surrounding landscape to the greatest reasonable extent possible.

Policy 3.10.6: Facilities Maintained by the Town. Stormwater storage facilities to be maintained by the Town will be regional in nature and shall be designed to Town Standards and shall be dedicated to the Town.

Policy 3.10.7: Public Health, Safety and Water Quality Enhancement. Stormwater storage facilities shall be designed with public health and safety in mind.

Policy 3.10.8: Regional Flood Control Facilities. New regional flood control facilities located in previously developed neighborhoods shall have flood control as their primary objective but to the extent practice multi-use features should be addressed.

Policy 3.10.9: Drainage of Storage Facilities. Storage facilities shall be designed to drain in accordance with the procedures in the Maricopa County Hydraulics Manual and this Manual. All stormwater storage facilities shall be designed to drain to appropriate drainage facilities within 36-hours.

Policy 3.10.10: Basin Geometry. Depth and side slopes of stormwater storage facilities shall be in accordance with the procedures in the Maricopa County Hydraulics Manual and this Manual. Non-geometric shapes are encouraged whenever practical. ADWR regulations may also apply, see Section 4.7. The basin shall be designed with an emergency spillway. See Standard 6.9.10 regarding the design of the emergency spillway.

Policy 3.10.11: Flows in excess of basin capacity. Flows in excess of basin capacity shall be directed to adjacent streets or drainage facilities.

Policy 3.10.12: Discharge to Town-Owned Facilities. The discharge of stormwater to Town-owned drainage facilities shall require approval. See Standard 6.9.1 Minimum Design Storm Criteria for requirements related to stormwater basin design and Standard 6.9.10 for requirements related to emergency spillway design.

Policy 3.10.13: Pump Station Use. The use of pump stations is only allowed for public projects, with maintenance oversight of a governmental agency.

Policy 3.10.14: Stormwater Discharge Pumping Requirements. The use of a stormwater discharge pump is prohibited without a Temporary Discharge Permit issued by the Town prior to the discharge. Laboratory analyses may be required by the Town prior to any discharge of water. All costs associated with the laboratory analyses will be the responsibility of the owner. Other restrictions may also apply such as flow rates and monitoring. If approved, the pump shall be maintained and operated by the owner, including Homeowners Associations.

Policy 3.10.15:On-Lot Storage. All developments, except single-family residential or multi-family residential of two units or less on individually platted lots or located in areas where regional detention has been approved through the adoption of a master plan, shall be required to provide on-site detention of stormwater. Water harvesting by individual lot owner is allowed, this is not to be used as part of the required detention volume for the development.

Policy 3.10.16: Offsite Flows. Offsite flows may not be routed through a stormwater storage facility without Town approval. Offsite flows shall not be co-mingled with onsite flows.

Policy 3.10.17: FEMA Special Flood Hazard Area. Stormwater storage facilities shall not be placed within a FEMA SFHA without Town approval.

3.11 Sand and Gravel Mining Floodplain Use Permits

All sand and gravel mining operations with watercourses in the Town must have an approved Floodplain Development Permit prior to commencing operation. Guidelines for obtaining such a permit are contained in Appendix A, Sand and Gravel Mining Guidelines.

3.12 Maintenance

It is essential that maintenance be considered during the planning, design, and construction of drainage facilities. Maintenance is provided so that the facilities can function as they were originally designed and constructed, and so that the service life of the facility is maximized. Common maintenance problems associated with drainage facilities include growth of undesirable vegetation, debris accumulation, sedimentation, erosion, scour, soil piping, soil settlement and structural damage. Drainage facilities are to be designed to avoid impacts to existing sediment transport conditions. Additional maintenance considerations regarding green stormwater infrastructure is included in Appendix B.

Policy 3.12.1: Permanent Accessibility. Provision for permanent drainage facility accessibility, including access for maintenance equipment into channels and culverts, is necessary for regularly scheduled maintenance activities. All drainage facilities shall be accessible for the appropriate maintenance equipment, with special consideration given to access during flood emergencies.

Policy 3.12.2: Consideration of Operation and Maintenance Cost During Design. All drainage facilities shall be designed and constructed with consideration to the cost of ongoing operation and maintenance, including maintenance related to stormwater quality.

Policy 3.12.3: Tracts for Town-Maintained Facilities. All drainage facilities that are to be functionally maintained by the Town shall be constructed within a designated Town- owned property, right-of-way or easement and clearly shown on the recorded plat.

Policy 3.12.4: Tracts for Privately-Maintained Facilities. All drainage facilities that are to be privately maintained shall be encompassed within a platted drainage tract with said tract clearly identified as private property. All drainage facilities owned and/or operated by private entities, including Homeowner's Associations, shall be properly maintained to promote performance of the drainage facilities consistent with the original design intent, including stormwater quality.

Policy 3.12.5: Tracts/Covenants, Conditions and Restrictions Requirement. The underlying property owner is responsible for aesthetic maintenance (weeds, trash, etc.) of all drainage tracts, right-of- way and easements. Where the Town is responsible for functional maintenance of the constructed facilities (Town-Maintained Facilities) this division of responsibilities shall be clearly shown on the recorded plat.

Policy 3.12.6: Alteration of Privately-Owned Facilities. Drainage features and facilities that are the responsibility of entities other than the Town (i.e., Homeowner's Associations, developers, management companies, private owners, etc.) may not be altered in form or function that detrimentally impacts the performance of the feature without a proper permit.

Policy 3.12.7: Trash Racks and Access Barriers. Trash racks at entrances and access barriers at outlets shall be provided for stormwater conduits as specified in Chapter 6 (Standards).

Policy 3.12.8: Section 404 Permits. Where required, Section 404 of the CWA permits shall be obtained prior to the start of maintenance activities.

3.13 Erosion Control During Construction

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 stormwater increases the rate of siltation of drainageways, which can exacerbate flooding and increase the cost of on-going maintenance.

Policy 3.13.1: Requirement. Appropriate erosion control measures shall be required at construction sites.

Policy 3.13.2: **Stormwater Pollution Prevention Plans**. SWPPP that incorporate BMPs shall be required of new developments that are equal to or greater than one acre as specified in the Town Drainage Standards. See Stormwater NPDES (see Section 4.6) and Town Stormwater Management (see Section 4.10) construction requirements for full details.

Policy 3.13.3: **Standards**. Erosion control shall be in accordance with the Maricopa County Erosion Control Manual and meet all ADEQ AZPDES Stormwater Management objectives and requirements.

3.14 Permitting

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 4). It is not the Town's responsibility to ensure that the plans for a proposed project satisfy state and federal permit requirements. This notwithstanding, Town has developed some policies associated with permitting.

Policy 3.14.1: Drainage and Grading Permit Requirement. The Town will not issue a Drainage and Grading Permit until the applicant documents that all of the applicable state and federal permits have been obtained.

4 Regulations

4.1 Purpose

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, and policies. Although these regulations are constantly changing, the following discussion provides some guidance as to the areas where governmental agencies exercise control over drainage related activities.

Any documents referenced in this chapter are intended to be the latest adopted versions. If newer versions exist other than what is included in this document, the user should refer to the current version.

4.2 Water and Cultural Resource Agency Contact List

The list that follows identifies the various agencies the user 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.

General Information

Environmental Protection Agency (EPA) Public Information Center
(415) 947-8000
(866) EPA-WEST
www.epa.gov/aboutepa/epa-region-9-pacific-southwest

Arizona Department of Environmental Quality (ADEQ)
(602) 771-2300, Main Number
(602) 771-2288, Ombudsman
(602) 771-2330, Emergency Response Line
www.azdeq.gov

Arizona Department of Water Resources (ADWR)
(602) 771-8500
1110 W Washington Street, Suite 310
Phoenix, AZ 85007
www.azwater.gov

Floodplain Information

Federal Emergency Management Agency (FEMA)
(510) 627-7100 (Oakland)
(202) 646-2500 (Washington D.C.)
www.fema.gov

Yavapai County Flood Control District (YCFCD)
(928) 771-3197
1120 Commerce Drive,
Prescott, AZ 86305
www.yavapaiaz.gov/Resident-Services/Flood-Control

National Pollutant Discharge Elimination System (NPDES) Permit
EPA (415) 972-3496
ADEQ (602) 771-2300

State Species of Concern
Arizona Game & Fish Department
(602) 942-3000
www.azgfd.com

Aquifer Protection Permits
ADEQ (602) 771-2300

Groundwater & Other Water Permits
ADEQ (602) 771-2300
ADWR (602) 771-8500

Water Quality Certification 401 Permits
ADEQ (602) 771-1440
www.azdeq.gov/swppermitting

Clean Water Act Section 404 Permits
Los Angeles District, Regulatory Branch
U.S. Army Corps of Engineers
(602) 230-6900
3636 N. Central Avenue, Suite 900
Phoenix, AZ 85012
www.usace.army.mil

Native Plant Law
Arizona Dept. of Agriculture Plant Services Division (602) 542-0994
www.agriculture.az.gov

Endangered Species Act
U.S. Fish & Wildlife Service
(602) 242-0210
https://www.fws.gov/office/arizona-ecological-services

Historic & Prehistoric Sites
State Historic Preservation Office
(602) 542-4009
www.azstateparks.com/shpo

Town of Prescott Valley
Public Works Department
7501 E. Skoog Blvd
Prescott Valley, AZ 86314
(928) 759-3073
https://www.prescottvalley-az.gov/170/Public-Works

4.3 National Flood Insurance Program (NFIP)

4.3.1 Introduction

The National Flood Insurance Act of 1968, as amended in 1973, provides for a federally subsidized NFIP conditioned on active management and regulation of floodplain development by states and local governments. 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 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 to reduce future flood risks to new construction in SFHA, 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 floods and resultant property damage is achieved through the delineation of property subject to flood events and the establishment of specific rules concerning development within these designated areas. FEMA publishes FIRMs for certain flood prone areas that delineate different SFHAs.

The Town participates in the NFIP and has adopted floodplain regulations and ordinances so that its citizens have access to the subsidized insurance.

4.3.2 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 CRS 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.

4.3.3 FEMA Special Flood Hazard Areas

Citizens within the Town 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 special flood hazard maps, FIRMs, are available for review at the Town, YCFCD, and the ADWR. FIRMS may also be viewed online at www.FEMA.gov. The FIRMs are used to determine if a property is located within a SFHA regulated by FEMA and used by the insurance industry to determine flood insurance rates. Areas considered within the SFHA includes the boundary line as shown on the FIRMs. If part of the structure is located within the SFHA, the whole structure is considered in the SFHA. Flood insurance through the NFIP is available for all properties, both those located outside and within the SFHA, unless a Notice of Violation (NOV) has been issued by the Floodplain Administrator.

4.3.4 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. The 100-year event is an event that has a one (1) percent chance of occurring in any given year. FEMA sets its jurisdictional limits to the water surface elevation from the 100-year event, which is cited as the BFE. Jurisdictional limits are defined by horizontal flooding limits using the BFE. The 100-year floodplain is divided by FEMA into the following hazard zones for flood insurance rating purposes:

    a. **Zone A**: No BFEs determined.

    b. **Zone AE**: BFEs determined.

    c. **Zone AH**: Flood depths of 1 to 3 feet (usually areas of ponding), BFEs 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.

Floodway: That portion of the 100-year floodplain that is required to convey the 100-year flood with a rise in water surface no greater than 1 foot. The allowable rise and the limits of the floodway are predetermined by the governing municipality.

4.3.5 Application Process

The following figures illustrate a generic representation of the permitting process for a single building lot and a larger community tract.

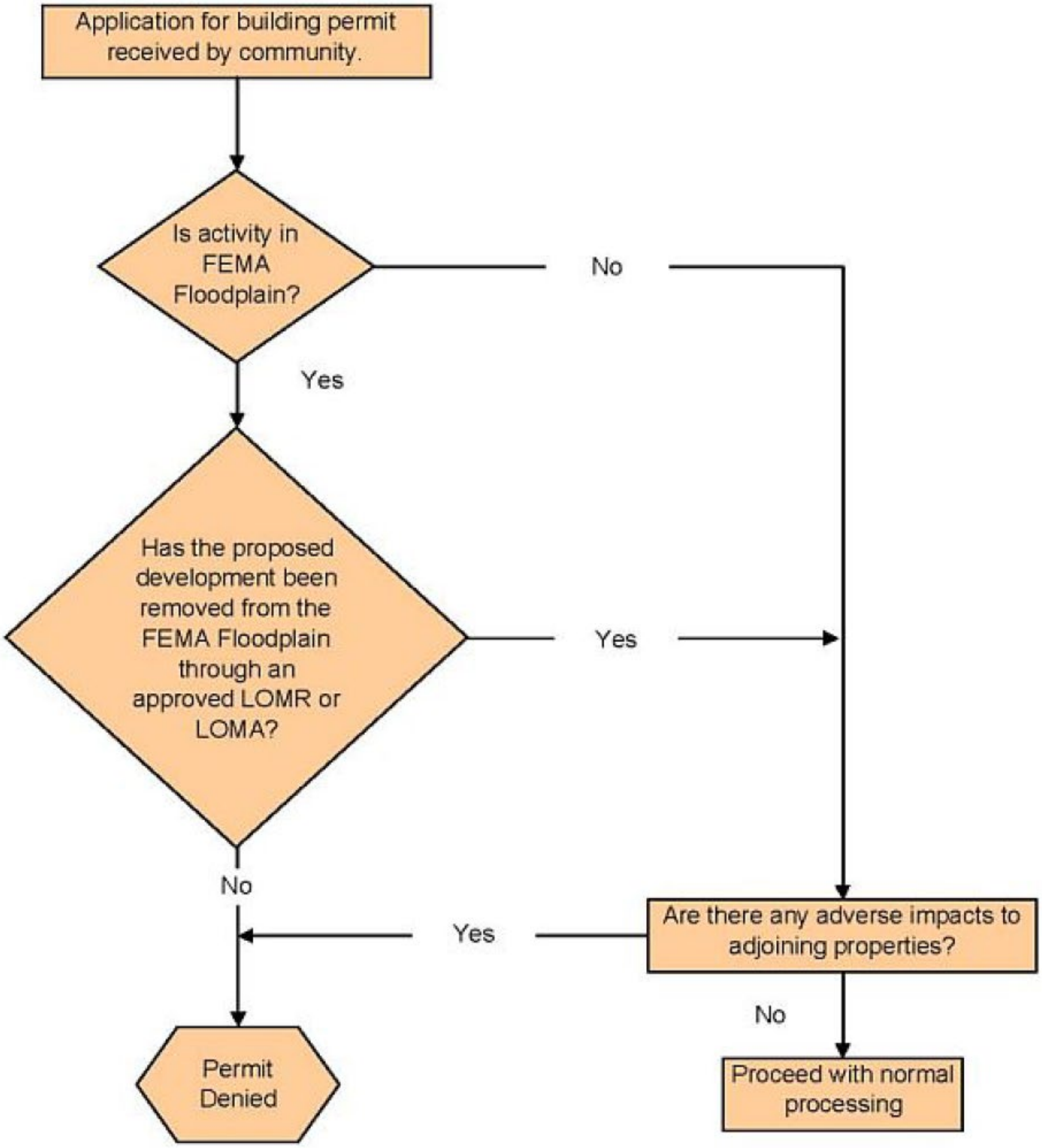

Figure 4-1. FEMA Floodplain Development Permit: Single Lot.

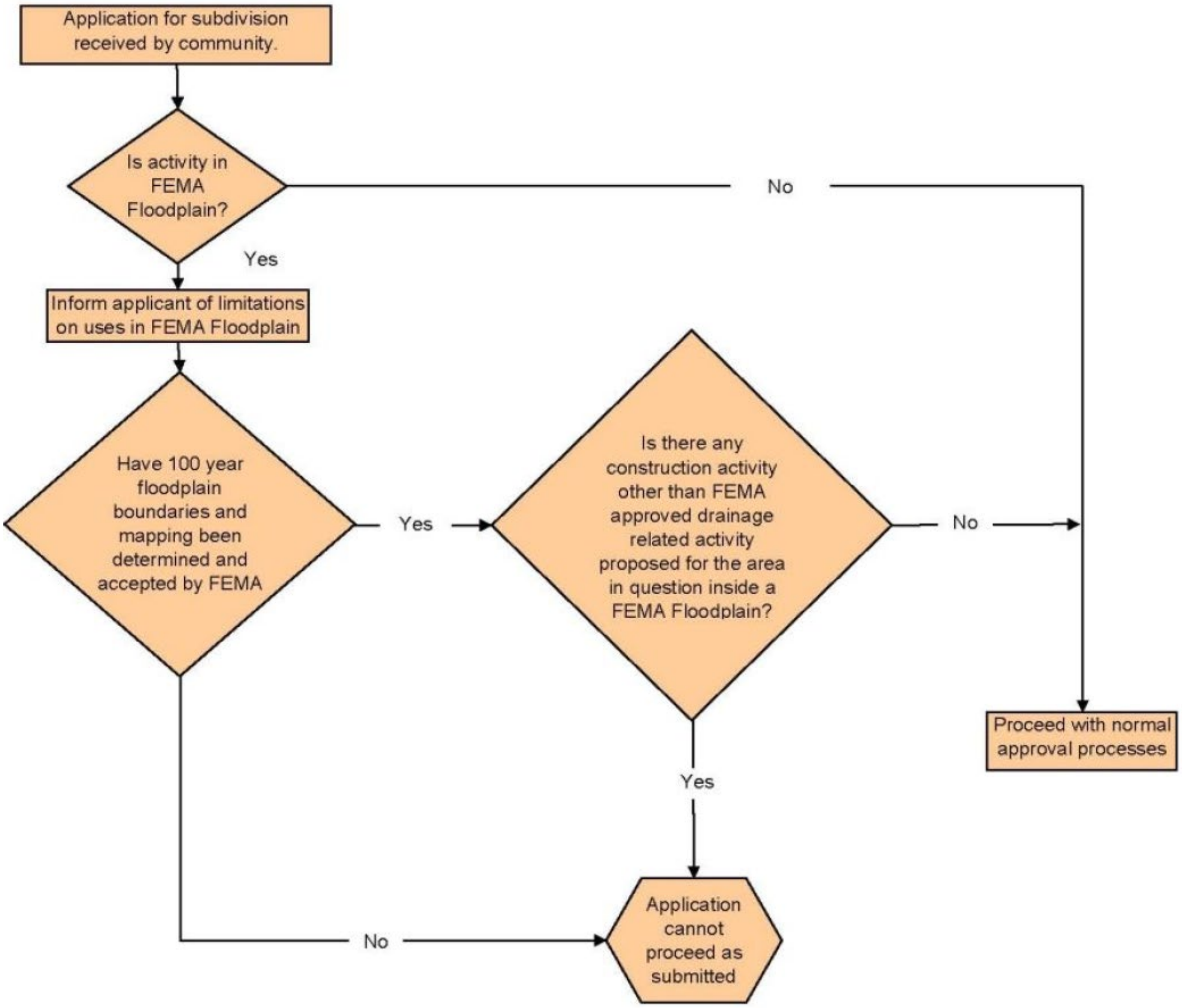

Figure 4-2. FEMA Floodplain Development Permit: Subdivision.

4.3.6 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 needs of the property and satisfies FEMA requirements. Each permit option requires completion of specific application forms and requires 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 to the MT-1 FEMA form. FEMA's contact address is provided in Section 4.2.

5. **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.

6. **Letter of Map Revision (LOMR)** - A letter from FEMA officially revising the current FIRM to show changes to floodplains, floodways, or flood elevations over a relatively small area. Physical changes include watershed development, flood control structures, etc.

7. **Physical Map Revision (PMR)** - A reprinted FIRM incorporating changes to floodplains, floodways, or flood elevations over a relatively large area. Because of the time and cost involved to change, reprint, and redistribute a FIRM, a PMR is usually processed when a revision reflects extensive changes in flood hazards or large-scope changes, and therefore typically takes a much longer time to process than a LOMR.

Application forms for the three items listed above can be obtained from FEMA by reference to the MT-2 FEMA form. FEMA's contact address is provided in Section 4.2.

Projects receiving a conditional letter must re-apply for a letter of map amendment or revision upon completion of construction. The conditional letter allows financing and local approvals 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 the

Town 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 includes revised data, 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 digital flood insurance rate map (DFIRM) panel.

4.3.7 Construction in Special Flood Hazard Areas

Per FEMA and NFIP minimum guidelines, the lowest floor of all residential structures constructed in the SFHA must be constructed to a minimum of the Regulatory Flood Elevation (RFE). The RFE is one (1) foot above the BFE which is defined as the 100-year water surface elevation. 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 RFE. See FEMA guidelines for further specifications. Basements of residential structures located in the SFHA must be elevated above the RFE.

The NFIP regulations allow non-residential 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 or above the RFE instead of being elevated. 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 RFE and elevating items subject to flood damage above the RFE. Flood-proofed structures must comply with appropriate sections of the NFIP regulation 44 CFR §60.3 and the Floodplain Regulations. A minimum of two (2) openings, on at least two (2) sides, having a total net area of not less than one (1) square inch for every square foot of enclosed area subject to flooding shall be provided. The bottom of all openings shall be no higher than one (1) foot above finished grade. Openings may be equipped with screens, louvers, valves, or other coverings or devices provided they allow the automatic entry and exit of floodwaters. See FEMA guidelines for further specifications.

Modular buildings must have the bottom of the structure (bottom of lowest beam and utilities) raised, as a minimum, to or above the RFE regardless of its use.

All new construction and substantial improvements shall be constructed with electrical, heating, ventilation, and air conditioning (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 RFE as a minimum. Below ground tanks such as propane and water tanks must be anchored against flotation. Above ground tanks are considered structures for floodplain management purposes.

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 of 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. Any development or changes in floodway elevation, width or location will require approval of FEMA by a CLOMR and a LOMR.

An "Elevation Certificate" (FEMA Form FF-206-FY-22-152) must be completed for each structure constructed in the SFHA prior to the electrical clearance and final acceptance for that structure. One copy of the "Elevation Certificate" is to be submitted to the Town as the Floodplain Administrator. See form instructions for a complete list of requirements.

4.3.8 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 44 CFR §60.3.c.(7). Non-residential or commercial structures may be flood-proofed in lieu of elevation. See *FEMA Publication, Best Practices, Building and Developing on Alluvial Fans* for more information (Federal Agency of Emergency Management, 2020). Adequate drainage paths must be provided in accordance with 44 CFR §60.3.c.(11).

4.3.9 Post Construction Review

After the proposed improvements have been constructed per a CLOMR, the owner/developer is required to submit as-built/documents of record to FEMA along with a request for a LOMR or amendment as appropriate. The Town Floodplain Administrator shall receive a copy. The Town must be in receipt of the final LOMA/revision from FEMA before the issuance of a building permit and occupancy will be approved.

4.3.10 Fees

Fees will be assessed by FEMA for its review of proposed and "as-built" projects, as outlined in NFIP regulations 44 CFR §72. See the Town Development Services Department for fee to help defray its cost for administering floodplain management in conformance with NFIP.

4.3.11 Additional Information

FEMA publishes numerous documents to aid those within or adjacent to a SFHA that can be located using FEMA's contact address in Section 4.2. Documents that are useful to consult if a property is determined to be within a SFHA are:

1. Floodplain Management and Protection of Wetlands, FEMA, 44 CFR §9.

2. See FEMA's site Guidelines and Standards for Flood Risk Analysis and Mapping Activities Under the Risk Mapping, Assessment, and Planning (MAP) Program at FEMA.gov.

3. "Technical Bulletin 2, Flood Damage-Resistant Materials Requirements for Buildings Located in Special Flood Hazard Areas in accordance with National Flood Insurance Program," FEMA, dated August 2008, revised 2010.

4. "Technical Bulletin 3, Requirements for the Design and Certification of Dry Floodproofed Non-Residential and Mixed-Use Buildings Located in Special Flood Hazard Areas in accordance with National Flood Insurance Program," FEMA, January 2021.

5. "Technical Bulletin 10, Reasonably Safe from Flooding Requirement for Building on Filled Land Removed from the Special Flood Hazard area in Accordance with the National Flood Insurance Program," FEMA, March 2023. Other publications about the NFIP can be found online at: www.fema.gov.

4.4 Non-FEMA Flood Hazard Areas

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 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 the ADWR website.

SS 1 - Requirement of Flood Study Technical Documentation

SS 2-96 - Requirement for Floodplain and Floodway Delineation in Riverine Environments

SS 3-94 - State Standard for Supercritical Flow

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

SS 10-07 - State Standard for Hydrologic Modeling Guidelines

4.4.1 Construction in Non-FEMA Flood Prone Areas

There are flood prone areas in the Town that do not have floodplains or floodways identified by FEMA. For these areas see Section 3.6.2. The lowest floor of any structure, residential or non-residential, constructed in any Town designated flood prone area must be above the RFE. The RFE is one (1) foot above the BFE which is defined as the 100-year water surface elevation (see Policy 3.6.2.1 in Section 3.6.2). Structures located within the flood prone areas may be protected from floods up to and including the 100-year flood by placement of fill to elevate the structure above the RFE.

All electrical, HVAC, plumbing and other service facilities shall be located as to prevent water from entering or accumulating within the components during conditions of flooding. Mechanical and electrical equipment must be installed one (1) foot above the BFE.

Stem walls for structures in flood prone areas shall be solid grouted concrete block or reinforced concrete. No openings shall be permitted along the sides most likely impacted by flowing storm runoff.

No adverse impacts to adjoining property are allowed (see Policy 3.6.2.3 in Section 3.6.2). No adverse impact to adjoining property is defined as raising stormwater runoff water surface elevations or as causing stormwater runoff to flow where it historically or currently does not flow by wholly or partially blocking or disturbing an existing water course with fill or a structure.

Engineering in the form of a drainage report and engineering plans sealed by a registered Arizona engineer must be completed prior to the Town allowing any permit for construction. Existing data such as the *Area Drainage Master Plan, Hydrology and Hydraulic Modeling, Technical Support Data Notebook* and other such studies may be used for this engineering, however, this information should be reviewed and verified by the engineer. The engineer takes sole responsibility for the information included in the final drainage report submitted to the Town.

A FEMA Elevation Certificate may be required prior to receiving a final approval by the Town.

4.5 Section 404 Permit for Waters of the United States

The USACE has been involved in regulating certain activities in the nation's waterways since the 1890s (River and Harbors Act of 1899). Until 1968, the primary thrust of the USACE regulatory program was the protection of navigation. As a result of several new laws and judicial decisions (CWA of 1968; Marine Protection, Research, and Sanctuaries Act of 1972), the program evolved to one that considers the full public interest by balancing the favorable impacts against the detrimental impacts.

Section 404 of the CWA, passed in 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. This act established federal jurisdiction over "navigable water," and is defined in the act as the "waters of the United States." Arizona recognized that there is continued debate on what is and what is not a WOTUS. The "Revised Definition of 'Waters of the United States'" was passed by the Arizona Legislature, and became effective March 20, 2023. This was later revised on August 29, 2023, to incorporate the US Supreme Court decision in the case of *Sackett v. Environmental Protection Agency*.

Also in response to federal changes in rules related to the WOTUS, the Arizona Legislature recognized a “local control approach” was necessary. ADEQ was given additional authority to create a state Surface Water Protection Program (SWPP) (see Arizona Revised Statue (ARS) §49-221). ADEQ rules regarding the SWPP became effective on February 20, 2023 (see AAC 18-11). As part of the SWPP a Protected Surface Water List was created, which lists all the water regulated either under the CWA or the SWPP. There are two waters and one lake in the Town that are included on the list, Agua Fria River, Lynx Creek, and Yavapai Lake, however there are numerous water courses in the Town that may qualify as jurisdictional under the CWA.

Any person, firm, or agency (including federal, state, and local government agencies) planning to work in or place dredged or fill material in WOTUS, must first obtain a permit from the USACE. The regulatory area is designated "Waters of the United States" or "jurisdictional waters." WOTUS 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 Prescott Valley, ephemeral streams (washes) may be jurisdictional if they exhibit certain characteristics. The width of the wash, presence of hydraulic sorting, and the presence of riparian habitat are factors, among other things, considered by the USACE. The regulations governing WOTUS (including wetlands) apply to both public and private property.

A jurisdictional delineation should be conducted in the early stages of planning to determine and mitigate (avoidance, replacement) potential impacts to resources from project alignment. A jurisdictional delineation determines the presence or absence and extent of jurisdictional waters on a site. Subsequently, it is highly recommended that the inexperienced seek guidance from the USACE and other environmental professionals.

4.5.1 Permits

Any ground disturbance in a watercourse or wetland may require a USACE permit. Common activities that require Section 404 and Section 401 of the CWA compliance, if conducted within WOTUS, 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 - environmental, social, and economic - in the Section 404 of the CWA permit decision-making process. As part of its responsibility to protect water quality, the USACE Section 404 permit program extends to many areas that were not regulated prior to the CWA.

Capital improvement projects undertaken on behalf of and paid for by the Town must coordinate their efforts with the Town prior to contacting the USACE. Joint ventures between the Town and private entities must coordinate with the Town prior to any inquiries or submittals to the USACE. Privately funded projects that are later to be conveyed to the Town will need to certify proper compliance with Section 404 of the CWA requirements. Should a permit be required, there are several options depending on the type of land disturbance activity including individual, nationwide, and regional.

4.5.2 Individual Permits

Individual permits are required for activities that have more than minimal adverse effects on the aquatic environment and other public interest review factors. Due to the more significant impacts, a full public interest review is required as part of the application for an individual permit. A public notice is distributed to all known interested persons including adjacent property owners and regulatory agencies. After evaluating all comments and information received, a final decision on the application is made.

The permit decision is generally based on the outcome of a public interest balancing process where the benefits of the project are balanced against the detriments. A permit will be granted unless the project is not found to be the least environmentally 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 requires a Section 401 Water Quality Certification from ADEQ. Application forms for individual permits are available from all USACE regulatory offices and ADEQ.

4.5.3 Nationwide Permits

A nationwide permit (NWP) is a form of general permit that authorizes a category of specific activities that exhibit minimal adverse effects on the aquatic 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 will be required. Please note that the NWP program was revised on February 25, 2022, and will expire March 14, 2026. This information is subject to change, so the user should check the USACE website periodically for changes since the date of this manual. NWPs listed below may be modified to accommodate regional conditions following review at a regional level. Contact the USACE office provided in Section 4.2 to obtain the most current information on the NWP program changes including a complete listing, permit details, and regional limitations placed upon NWPs. Some activities under NWPs require notification submittals to the USACE prior to the carrying out of those activities. For instance, notification requirements are described in General Condition 13, Bank Stabilization. All NWPs must comply with the requirements of the particular NWP, the general conditions of the NWP, the Section 401 conditions (for water quality), and, if adopted, the Los Angeles District regional conditions and definition of WOTUS. A list of the more pertinent, presently available, NWPs follows.

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

provided that the structure or fill is not to be put to uses differing from those uses specified or contemplated for it in the original permit or the most recently authorized modification. This NWP also authorizes the removal of previously authorized structures or fills. Any stream channel modification is limited to the minimum necessary for the repair, rehabilitation, or replacement of the structure or fill; such modifications, including the removal of material from the stream channel, must be immediately adjacent to the project. This NWP also authorizes the repair, rehabilitation, or replacement of those structures or fills destroyed or damaged by storms, floods, fire, or other discrete events, provided the repair, rehabilitation, or replacement is commenced, or is under contract to commence, within two years of the date of their destruction or damage.

NWP 6: Survey Activities. Survey activities including core sampling, seismic exploratory operations, plugging of seismic shot holes and other exploratory-type bore holes, exploratory trenching, soil survey, sampling, sample plots or transects for wetland delineations, and historic resources surveys.

NWP 7: Outfall Structures and Associated Intake Structures. Activities related to construction or modification 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 NPDES (Section 402 of the CWA). The construction of intake structures is not authorized by this NWP unless they are directly associated with an authorized outfall structure.

NWP 12: Oil or Natural Gas Pipeline Activities. Activities required for the construction, maintenance, repair, and removal of oil and natural gas pipelines and associated facilities in WOTUS, provided the activity does not result in the loss of greater than 1/2-acre of WOTUS for each single and complete project.

NWP 14: Linear Transportation Projects. Activities required crossings of WOTUS associated with the construction, expansion, modification, or improvement of linear transportation projects (e.g., highways, railways, trails, driveways, airport runways and taxiways) in WOTUS subject to acreage limitations.

NWP 18: Minor Discharges. Minor discharges of dredged or fill material into all WOTUS subject to volume or acreage limitations. Also, the discharge of dredged or fill material is not placed for the purpose of a stream diversion.

NWP 20: Response Operations for Oil or Hazardous Substances. Activities conducted in response to a discharge or release of oil and hazardous substances which are subject to the National Oil and Hazardous Substances Pollution Contingency Plan (40 CFR §300) including containment, cleanup, and mitigation efforts in accordance with certain state and federal requirements.

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 or for general navigation, such as mooring cells, including the excavation of bottom material from within the form prior to the discharge of concrete, sand, rock, etc. This NWP does not authorize filled structural members that would support buildings, building pads, homes, house pads, parking areas, storage areas and other such structures.

NWP 29: Residential Developments. Discharges of dredged or fill material into non-tidal WOTUS, including non-tidal wetlands for the construction or expansion of a single residence, a multiple unit residential development, or a residential subdivision and attendant features (such as a garage, driveway, storage shed, and/or septic field).

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, retention/detention basins, levees, and channels. The maintenance is limited to that approved in a maintenance baseline determination made by the District Engineer.

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 WOTUS for the construction or expansion of 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 WOTUS for agricultural activities, including 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 WOTUS; and similar activities.

NWP 41: Reshaping Existing Drainage and Irrigation Ditches. Discharges of dredged or fill material into non-tidal WOTUS excluding non-tidal wetlands adjacent to tidal waters, to modify the cross sectional configuration of currently serviceable drainage and irrigation ditches constructed in WOTUS, for the purpose of improving water quality by regrading the drainage or irrigation ditch with gentler slopes, which can reduce erosion, increase growth of vegetation, and increase uptake of nutrients and other substances by vegetation. The reshaping of the ditch cannot increase drainage capacity beyond the original design capacity nor expand the area drained by the ditch as originally constructed (i.e., the capacity of the ditch must be the same as originally constructed and it cannot drain additional wetlands or other WOTUS).

NWP 42: Recreational Facilities. Discharges of dredged or fill material into non-tidal WOTUS for the construction or expansion of recreational facilities.

NWP 43: Stormwater Management Facilities. Discharges of dredged or fill material into non-tidal WOTUS for the construction of stormwater management facilities, including stormwater detention basins and retention basins and other stormwater management facilities; the construction of water control structures, outfall structures and emergency spillways; the construction of low impact development integrated management features such as bioretention facilities (e.g., rain gardens), vegetated filter strips, grassed swales, and infiltration trenches; and the construction of pollutant reduction green infrastructure features designed to reduce inputs of sediments, nutrients, and other pollutants into waters, such as features needed to meet reduction targets established under Total Maximum Daily Loads set under the CWA.

NWP 44: Mining Activities. Discharges of dredged or fill material into non-tidal jurisdictional wetlands, non-tidal jurisdictional open waters, or non-tidal navigable WOTUS for mining activities other than coal mining activities subject to certain limitations.

NWP 45: Repair of Uplands Damaged by Discrete Events. Discharges of dredged or fill material, including dredging or excavation, into all WOTUS for activities associated with the restoration of the 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 WOTUS prior to the construction of the ditch, (3) divert water to an area determined to be a WOTUS prior to the construction of the ditch, and (4) are determined to be WOTUS. The discharge must not cause the loss of greater than one acre of WOTUS.

NWP 57: Electric Utility Line and Telecommunications Activities. Activities required for the construction, maintenance, repair, and removal of electric utility lines, telecommunication lines, and associated facilities in WOTUS, provided the activity does not result in the loss of greater than 1/2-acre of WOTUS for each single and complete project.

NWP 58: Utility Line Activities for Water and Other Substances. Activities required for the construction, maintenance, repair, and removal of utility lines for water and other substances, excluding oil, natural gas, products derived from oil or natural gas, and electricity. Oil or natural gas pipeline activities or electric utility line and telecommunications activities may be authorized by NWPs 12 or 57, respectively. This NWP also authorizes associated utility line facilities in WOTUS, provided the activity does not result in the loss of greater than 1/2-acre of WOTUS for each single and complete project.

NWP 59: Water Reclamation and Reuse Facilities. Discharges of dredged or fill material into non-tidal WOTUS for the construction, expansion, and maintenance of water reclamation and reuse facilities, including vegetated areas enhanced to improve water infiltration and constructed wetlands to improve water quality.

To apply for a NWP, an application form must be completed. USACE application forms for the permits are available from the local USACE regulatory offices (see contact information included in Section 4.2).

4.5.4 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 the regional permit reduces duplication of regulatory control by State and Federal agencies,

2. Can be issued to work with Special Area Management Plan, large-scale Habitat Conservation Plan. Can have different limits, conditions than NWPs.

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

4.6 Stormwater Quality NPDES

Stormwater systems are subject to the requirements and permitting process of the NPDES which is a U.S. Environmental Protection Agency (EPA) program and is the administrative mechanism chosen for stormwater permitting. The EPA issued regulations in 1990 authorizing the creation of an NPDES permitting system for stormwater discharges from a large group of industrial and construction activities. In Arizona, the NPDES program is called AZPDES. An AZPDES permit is required for any point source discharge of pollutants to a WOTUS. Because stormwater runoff can transport pollutants to either municipal storm sewer systems or to WOTUS, 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.

4.6.1 Permits

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.

4.6.2 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. In many areas, however, the effectiveness of ordinances in reducing pollutants is limited due to inadequate enforcement or incomplete compliance with local ordinances by construction site operators.

4.6.2.1 Construction Activity 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 NOI in accordance with the general permit.

Stormwater associated with large construction activity refers to the disturbance of 5 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 5 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 5 acres.

Permit Waivers. A Erosivity Waiver is available for small construction activities (between one and five acres) where there is low potential for soil erosion. Construction activities that disturb five acres or greater, or less than five acres but are part of a common plan of development or sale, are not eligible for the Erosivity Waiver. Low potential for erosion is defined as a rainfall erosivity (R) factor of less than five (5) and is calculated in myDEQ, which uses the EPA's methodology for determining if a site qualifies for the erosivity waiver, based on the *USDA Handbook 703-Predicting Soil Erosion by Water: A Guide to Conservation Planning with the Revised Universal Soil Loss Equation (RUSLE)*, dated January 1997. EPA has updated its Rainfall Erosivity Factor Calculator to correct known problems and to use updated data from the NRCS Revised Universal Soil Loss Equation, Version 2 (RUSLE2) database. myDEQ is using the Version 2 for erosivity calculations for the 2020 Construction Activity General Permit (CGP). The small construction site's rainfall erosivity calculation shall be less than five (5) acres during the entire period of construction activity. See the current general permit requirements for more information.

How to Obtain Coverage. The operator of a construction site is responsible for obtaining coverage under an AZPDES permit. 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.

Submit a NOI to ADEQ via myDEQ, ADEQ's permitting portal. This form must be complete and accurate and signed by the appropriate party in order to obtain coverage. The form also serves as a promise by the operator that there will be compliance with the permit conditions. The operator must also develop and implement a SWPPP that satisfies the conditions of the permit. If your site meets one of the following conditions the SWPPP must be submitted with the NOI:

- Any portion of the construction site is located within ¼ mile upstream of a receiving water listed as impaired under Section 303(d) of the CWA.

- Any portion of the construction site is located within ¼ mile upstream of a receiving water listed as an Outstanding Arizona Water in AAC R18-11-112(G).
- ADEQ specifically requests a copy of the site SWPPP be submitted for department review. This may occur as part of the NOI evaluation, at any time during permit coverage, in response to an inspection conducted by ADEQ, as part of the NOT, and for up to three years after the NOT is submitted.
- A Change of Operator form is submitted as a result of bankruptcy/foreclosure.

In all other cases, do not submit the SWPPP to ADEQ; however, the SWPPP must be available for ADEQ review. Coverage under Arizona's CGP is obtained by submission of a complete and accurate NOI. The NOI must be submitted electronically via myDEQ. The myDEQ permitting portal is located on the ADEQ website, at www.azdeq.gov. The permittee is granted coverage under the CGP when they have received authorization from ADEQ, via myDEQ. It is the operator's responsibility to verify the date the NOI was received by ADEQ before initiating construction activities. The SWPPP can be requested by any agency (including the Town) and should remain available for review at the project site. For a more detailed description of unique or impaired waters, please see ADEQ's website.

Notice of Termination. To terminate permit coverage, the operator shall submit a complete and accurate NOT in myDEQ after any of the following conditions have been met:

- The operator has established final stabilization on all portions of the site for which the operator is responsible.
- Another operator who has a valid authorization number under this general permit or an individual AZPDES permit has assumed control over all areas of the site that have not been finally stabilized.
- For residential construction activities, temporary stabilization has been completed, and the residence has been transferred to the homeowner.
- The planned construction activity identified on the original NOI was never initiated (i.e., grading was never started) and plans for construction have been permanently abandoned or indefinitely postponed.
- The operator has obtained coverage for the site under another authorizing AZPDES permit.
- The operator qualifies for one of the alternatives as listed in the General Permit for Stormwater Discharges Associated with Construction Activity to Protected Surface Waters and submits the required documentation demonstrating compliance with the NOT in myDEQ.

See the General Permit for Stormwater Discharges Associated with Construction Activity to Protected Surface Waters for more information.

4.6.2.2 ADEQ's Construction Activity General Permit

ADEQ's CGP (AZG2020-001) was issued on March 27, 2020, replacing ADEQ's 2013 CGP. The 2020 CGP was modified to include non-WOTUS protected surface waters on September 29, 2021. The current AZPDES CGP expires on June 30, 2025.

The construction general permit authorizes stormwater discharges from large and small construction-related activities that result in a total land disturbance of equal to or greater than 1 acre, where those discharges enter surface WOTUS or a storm drain. Permittees discharging to non-WOTUS protected surface waters are subject to state requirements only per ARS § 49-255.04(C), enforceable solely by ADEQ. Note the AZPDES authorizing statute uses the term "navigable waters" which are defined as equivalent to the WOTUS. 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, which may not be able to be traveled by conventional vessels), this permit generally references discharges to WOTUS. This permit expands coverage from the 1998 construction general permit that provided coverage for large construction sites (i.e., those disturbing greater than 5 acres) to include both small and large construction activities (i.e., any project disturbing greater than 1 acre).

Permit Area. This general permit covers stormwater discharges from large and small construction activity in Arizona, except for those construction discharges in Indian Community Lands.

4.6.3 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 available to facility operators.

4.6.3.1 Multi-Sector General Permit

ADEQ's MSGP (AZMSG2019-001) was issued on January 1, 2020, replacing ADEQ's 2010 MSGP. The 2020 MSGP was modified to include non-WOTUS protected surface waters on September 29, 2021. The current MSGP expires on December 1, 2024.

The MSGP is designed for those industrial activities that are of a non-construction nature. This is one large permit divided into numerous separate sectors. Each sector represents a different type of activity. A facility's sector within the MSGP 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 an SIC code or narrative description is determined, review the Sector List on ADEQ's website 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.

Use myDEQ to obtain, modify or terminate a 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, within the sectors, details the requirements the EPA considers necessary for each sector to produce an acceptable SWPPP. There is no requirement to submit the SWPPP to ADEQ, however, the Town requires that the plan be submitted to and approved by the Town. The SWPPP can be requested by any agency and should remain available for review on-site. ADEQ will confirm permit coverage within myDEQ. 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 NOT to end participation in the NPDES stormwater program. Failure to develop specific BMPs 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.

4.6.4 Individual Permit

The Individual Permit is a permitting strategy used by those who believe the general permit requirements do not accurately represent the activity at their facility and wish a permit customized to their site. Another major reason an Individual Permit may be necessary is that the Limitations on Coverage section of the general permits does not allow the facility's discharge to be covered within the general permit. For example, if the stormwater discharge from the facility adversely affects an endangered species, an Individual Permit would be required. It is the responsibility of every applicant to determine if any of the Limitations on Coverage apply to the facility seeking the permit. To apply for an Individual Permit, see ADEQ's site for Individual Permits.

Permit information and forms may be obtained from the agencies provided in Section 4.2.

4.7 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 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 up to 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.

A JURISDICTIONAL DAM is either more than 25 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.

4.7.1 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.

4.8 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. Because of the existing soils in the area, the Town does not permit new drywells for stormwater disposal. 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.

4.8.1 Permits

Drywells are regulated by ARS § 49-241, 49-242, 49-250, and 49-252, and Aquifer Protection Permit (APP) statutes and rules. Drywells that drain areas where hazardous substances are used, stored, loaded, or treated are subject to the General Permit or full APP (see Section 4.9). Specific rules regarding drywells are found in AACs R18-9-102 and R18-9-A301. Program guidance documents are available from ADEQ, and should be followed for drywell construction, maintenance, siting, investigation, decommissioning, and closure. Registration is generally not required for drywells used in conjunction with golf course maintenance, and they are exempted from regulation under the drywell program. However, vadose zone injection wells (including drywells) 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 use permit is issued by statute in lieu of an individual permit, provided that six criteria, including registration, are met (ARS § 49-245.02).

Drywell registration and permit information and forms may be obtained from ADEQ at the location provided in Section 4.2.

4.9 Aquifer Protection Permit

An individual will need to obtain an APP if they own or operate a drywell 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 drywells in areas where hazardous substances are used, stored, loaded, or treated. Drywells that are used solely for the disposal of stormwater runoff does not require an APP; however, drywell registration is still a requirement.

4.9.1 Permits

The following APP Permits are available:

4.9.2 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.

4.9.3 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.

4.9.4 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 APPs are available from ADEQ at the location provided in Section 4.2.

4.10 Town of Prescott Valley Stormwater Management

It is the goal of the Town to protect, maintain, and enhance the public health, safety, and general welfare by establishing requirements and procedures to control the adverse effects of stormwater runoff and pollution associated with land development. This Manual sets forth the policies and standards for management of urban drainage and floodplains.

4.10.1 Permits

The Town has permit requirements for stormwater facilities. Individual permits are available for the following.

1. Drainage and Grading Permit.
2. SWPPP.
3. Right-of-Way Permit

4.10.2 Drainage and Grading Permit

A Grading and Drainage Permit is required for development activities that include excavation, fill, drainage swales and channels, drainage structures and pipes, and detention areas.

A Drainage and Grading Permit is also required in order to connect and discharge stormwater into the Town's drainage infrastructure. New storm drain segments or inlets, low- flow bleed-off lines from detention basins, or stormwater discharge pumps are examples of drainage facilities requiring a permit. This permit provides a procedure for the Town to track additions to the Town's storm drain system.

4.10.3 Stormwater Pollution Prevention Permit

The Town has adopted the EPA NPDES / ADEQ AZPDES system as its stormwater management program. A SWMP has been adopted in compliance with ADEQ's MS4 Permit requirements.

Any construction site of one acre or greater requires erosion control conforming to the SWMP and ADEQ AZPDES system. A SWPPP approved by the Town and the ADEQ NOI / NOT process must be complied with. For construction sites less than one acre the developer / contractor is still responsible for the control of silt and mud leaving the site per all applicable Town Standards.

Inspection for compliance with this program is carried out by the Town. If program permit conditions cannot be met through discussion between the Town and the permittee ADEQ staff are to be contacted and brought into the process.

4.10.4 Right-of-Way Permit

A Right-of-Way Permit is available at no charge to a permittee that needs to perform work in the Town's roadway or easements. All public utilities are located within the Town's right-of-way or easement. Inspection for compliance is carried out by the Town to ensure safe and proper environment exists for the contractor and the public. Depending on the work the Town may require the applicant to also submit drawings of the work to be performed or a traffic control plan to more adequately evaluate the project. Contact the Public Works Department for more information.

4.10.5 Contact Information

Town of Prescott Valley Public Works Department
7501 E Skoog Boulevard
Prescott Valley, Arizona 86314
(928) 759-3070
www.prescottvalley-az.gov/170/Public-Works

5 Town Regulations

Other regulations related to drainage and flood control are included in the Prescott Valley Municipal Code. The primary portions of the Code that apply are noted in the following sections. The full Municipal Code is available on the Town's website at www.prescottvalley-az.gov/237/Town-Code.

5.1 Excavating, Grading & Drainage Regulations for the Town of Prescott Valley

Section 7-02-350.J of the Town Municipal Code modifies Appendix Chapter J of the International Building Code, renaming it Grading. Many of the modifications to the Chapter deal with site grading and drainage for individual buildings.

5.2 Floodplain Regulations for the Town of Prescott Valley

Chapter 12, Floodplain Management, of the Town Municipal Code, is commonly referred to as the Floodplain Regulations. These regulations deal with activities within flood hazard areas as defined within the Chapter.

6 Standards

6.1 Introduction

The ADOT Hydrology, Maricopa County Hydraulics, and Maricopa County Erosion Control Manuals provide technical guidance for definition and evaluation of flood and erosion hazards, and for design of drainage facilities. This chapter contains the standards for applying the technical concepts contained in the manuals for design of drainage facilities in the Town.

The Town of Prescott Valley maintains Design and Construction Standards and Specifications for public works infrastructure, including drainage. This document is a supplement to the YCFCD Drainage Design Manual which is a supplement to the Maricopa Association of Governments (MAG) Uniform Standard and Specifications and Details for Public Works Construction. Together these three documents and the standards in this chapter define the vast majority of design and construction standards for drainage facilities within the Town.

6.2 Safety and Protection of the Natural Environment

Designs for hydraulic structures must address the issue of safety. Since the Town has established the policy that disturbances to natural watercourses shall be minimized (Policy 3.3.3), the design of hydraulic structures must also address the protection of the natural environment. The following standards address these issues:

Standard 6.2.1: 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, and any other hazards identified by the town or county engineer. Alluvial fans and distributary flow areas can be identified using guidance provided FEMA Guidance for Flood Risk Analysis and Mapping Alluvial Fans 2016 or more current version (FEMA, 2016). Engineering analysis and design are required for development within special hazard areas.

Standard 6.2.2: Protection Related to Depth and Velocity. 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 the design of all drainage facilities, including stormwater storage facilities, channels, storm drains and street systems. A hazard rating system, developed by the United Kingdom Environment Agency, identifies four severity levels that consider pedestrian safety for different conditions of flow depth and velocity and includes a factor for the presence of debris during the flooding event (Wallingford, 2006). These categories are combined with a similar rating system used by the U.S. Bureau of Reclamation (Bureau of Reclamation, 1988) that incorporates risk for vehicles. Definitions of each level are provided in Table 6-1. The hazard rating is determined from the following equation obtained from the United Kingdom Environmental Agency:

$$QHR = D(V + 0.5) + DF \qquad \textbf{Equation 6-1}$$

where:

HR = Hazard Rating

D = 100-year, 24-hour maximum flow depth, in meters

V = 100-year, 24-hour maximum flow velocity, in meters/second

DF = Debris factor

for D < 0.25 meters, DF = 0.5

for D > 0.25 meters, DF = 1.0

Table 6-1. Pedestrian and Vehicle Risk Severity

Severity Level Score	Hazard Rating	Definitions
Low (1)	Less than 0.75	Shallow flowing water with low velocity. Most people and vehicles can safely traverse these areas.
Moderate (2)	0.75 – 1.25	Dangerous for some people (i.e. children) and lightweight vehicles: flooded area with deep or fast flowing water and the chance of debris.
High (3)	1.25 – 2.0	Dangerous for most people and vehicles: flooded area with deep and fast flowing waters, with the likely inclusion of debris. Most people will be unable to stand in these conditions. People and vehicles are at risk of being swept downstream with the potential for loss of life.
Extreme (4)	Greater than 2.0	Dangerous for all people and vehicles: flooded areas with very deep and fast flowing, debris laden waters. All people and vehicles, including emergency response vehicles, are at risk of being swept downstream with loss of life.

Engineering judgment shall be applied in assessing the risks and determining which areas require special attention. With the areas of concern defined, the designer shall 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.
3. Flood warning alarm or announcement systems.
4. Physical barriers, such as fencing or railings.
5. Higher minimum technical standards for design of drainage facilities.

Standard 6.2.3: Channel Drop Structure Height. For all channel drop structures, the maximum vertical drop height from invert crest to invert toe shall be 2.5 feet. For greater drop heights, a 6-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.

Standard 6.2.4: Emergency Escape Requirements for Lined Channels. All concrete, shotcrete, or smooth sided soil cement channels with a design flow depth greater than 3 feet shall have emergency escape stair-steps formed, alternating every 300 feet from one side of the channel to the other.

Standard 6.2.5: Stormwater Storage Ponds with Permanent Water Body. For stormwater 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 8 feet of the edge of the water feature, and gradually get deeper as needed.

Standard 6.2.6: Amenities within Stormwater Storage Facilities. Amenities placed within the inundation area of a stormwater facility shall be adequately secured to prevent them from becoming water-borne debris. Methods for securing items shall be shown on the design and construction plans.

Standard 6.2.7: Fall Protection/Fencing Requirement. Fall protection or fencing will be required for all water storage detention basins where the high water level is deeper than three feet and the side slopes are steeper than 3:1. Fall protection or fencing is required for channels deeper than 30 inches and less than 10 feet with side slopes steeper 2:1 and for channels deeper than 10 feet with side slopes steeper than 3:1.

Standard 6.2.8: Security Lighting. Security lighting shall be placed to provide illumination within multi-use, stormwater storage facilities, per Town Outdoor Lighting Standards.

Standard 6.2.9: Walkways. Walkways, including multi-use paths, must meet the Americans with Disabilities Act (ADA) requirements and be elevated above the top of any low flow channel.

Standard 6.2.10: Access Ramps and Access Roads. All drainage facilities must be readily accessible by emergency or maintenance vehicles.

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, such that maintenance activities can be undertaken without encumbering traffic flow or encroaching into Section 404 of the CWA jurisdictional or natural open spaces designated for protection.
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. A minimum of one (1) access ramp will be required for each reach of channel, defined by vertical drops or obstructions such as street culvert crossings.
4. For all other small engineered channels such as swales, roadside drainage ditches, etc., reasonable access for emergency and ordinary maintenance vehicles shall be provided.

5. For natural washes, a minimum 16-feet wide accessible clear-zone area for emergency and ordinary maintenance vehicle access shall be provided.
6. 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.
7. 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.
8. Portions of access ways or ramps may be combined with portions of multi-use paths, subject to approval by the Town.

Standard 6.2.11: Trash Racks and Access Barriers. When any of the following conditions are met, trash racks are required on the entrances and access barriers on outlets to all conduits or other hydraulic structures. Refer to Table 6-5 for additional guidelines.

Table 6-2. Conduit and Hydraulic Structure Trash Rack and Access Barriers

Facility Description	Diameter or Cross Sectional Area (per barrel)	Length	Inlet Trash Rack Required	Outlet Access Barrier Required
Culvert 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
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

Flap gates may be substituted for access barriers on conduit or hydraulic structure outlets when it can be shown that sedimentation will not prevent the flap gate from opening or that the design of the outlet structure will reduce downstream sedimentation that would prevent the flap gate from opening.

Trash rack and access barrier assemblies shall be secured from public access, but hinged or removable to allow access for maintenance. The screen shall be fabricated of a minimum of 1/2" x 2" flat steel bars or larger, designed to withstand the hydrostatic load resulting from the 100-year design event. Attachment points shall be cast in the headwall concrete and anchored by appropriately designed anchor bolts. Shear pins shall be 1/8", 3/16", or 1/4" rods depending on the size of the barrier involved. The largest size possible shall be utilized. The rack assembly shall be galvanized steel or steel with a protective coating suitable for exposure to sunlight, as well as submerged conditions. An anti-vortex device shall be included with the trash rack design if vortices are anticipated which could affect hydraulic efficiency and cause erosion of adjacent earth slopes. Refer to U.S. Department of Transportation Federal Highway Administration, Hydraulic Design of Highway Culverts (Federal Highway Administration, 2012) for more information.

6.3 Hydrology

The hydrology for the Town is based on the methods in the Drainage Design Manual for Yavapai County (Yavapai County, 2015). The manual is based on the following documents:

- Drainage Design Manual for Mohave County (Mohave County, 2009)
- ADOT Highway Drainage Design Manual, Hydrology (Arizona Department of Transportation, 2014)
- FCDMC Drainage Design Manual, Hydrology (Flood Control District of Maricopa County, 2023)

The user is referred to these documents for background information and more detailed technical documentation.

6.3.1 Design Storm Criteria

The design storm for synthetic unit hydrograph calculation methods shall be a 24-hour duration "frequency storm" distribution as defined by the HEC-HMS program. The precipitation depth/duration/frequency shall be obtained from the precipitation data prepared by the U.S. Weather Service available from the Hydrometeorological Design Studies Center Precipitation Frequency Data Server (PFDS) at https://hdsc.nws.noaa.gov/pfds/. The data available from the PFDS varies across the Prescott Valley area. Table 6-3 provides the rainfall data for the Town Civic Center and shown on Figure 6-1. These values will be acceptable for all developments near the Town Civic Center and Rational Method calculations within the incorporated Town. Precipitation frequency data for a specific study shall be obtained from the U.S. Weather Service PFDS for larger projects. The design frequency to be employed is identified in the sections below for each type of facility. The mean precipitation frequency estimates shall be used.

In addition to the depth and frequency, HEC-HMS also requires the selection of the storm probability, input type, intensity duration, and intensity position. The probability relates to the rainfall probability to be modeled. The input type should match the data type retrieved from the PFDS website or Table 6-3. It is recommended to use partial duration statistics. It is recommended to use partial duration statistics and to use this setting (the default value) in HEC-HMS. The intensity duration will normally be set to 5 minutes but is related to the time of concentration as described in Section 6.3.5.3.

Table 6-3. Rainfall Data

RAINFALL DEPTH-INTENSITY-DURATION-FREQUENCY DATA

PRESCOTT VALLEY, AZ

Time in minutes	Depth in inches						Intensity in inches/hour					
	2 YR	5 YR	10 YR	25 YR	50 YR	100 YR	2 YR	5 YR	10 YR	25 YR	50 YR	100 YR
5	0.30	0.41	0.50	0.63	0.73	0.84	3.60	4.92	6.00	7.56	8.76	10.08
10	0.47	0.63	0.76	0.95	1.11	1.28	2.82	3.78	4.56	5.70	6.66	7.68
15	0.57	0.78	0.94	1.18	1.38	1.58	2.28	3.12	3.76	4.72	5.52	6.32
30	0.77	1.05	1.27	1.59	1.85	2.13	1.54	2.10	2.54	3.18	3.70	4.26
60	0.96	1.30	1.57	1.97	2.29	2.64	0.96	1.30	1.57	1.97	2.29	2.64
120	1.10	1.45	1.75	2.18	2.53	2.91	0.55	0.73	0.88	1.09	1.27	1.46
180	1.18	1.52	1.81	2.22	2.58	2.95	0.39	0.51	0.60	0.74	0.86	0.98
360	1.38	1.72	2.02	2.45	2.79	3.17	0.23	0.29	0.34	0.41	0.47	0.53
720	1.66	2.04	2.36	2.78	3.12	3.46	0.14	0.17	0.20	0.23	0.26	0.29
1440	2.00	2.51	2.92	3.47	3.91	4.35	0.08	0.10	0.12	0.14	0.16	0.18

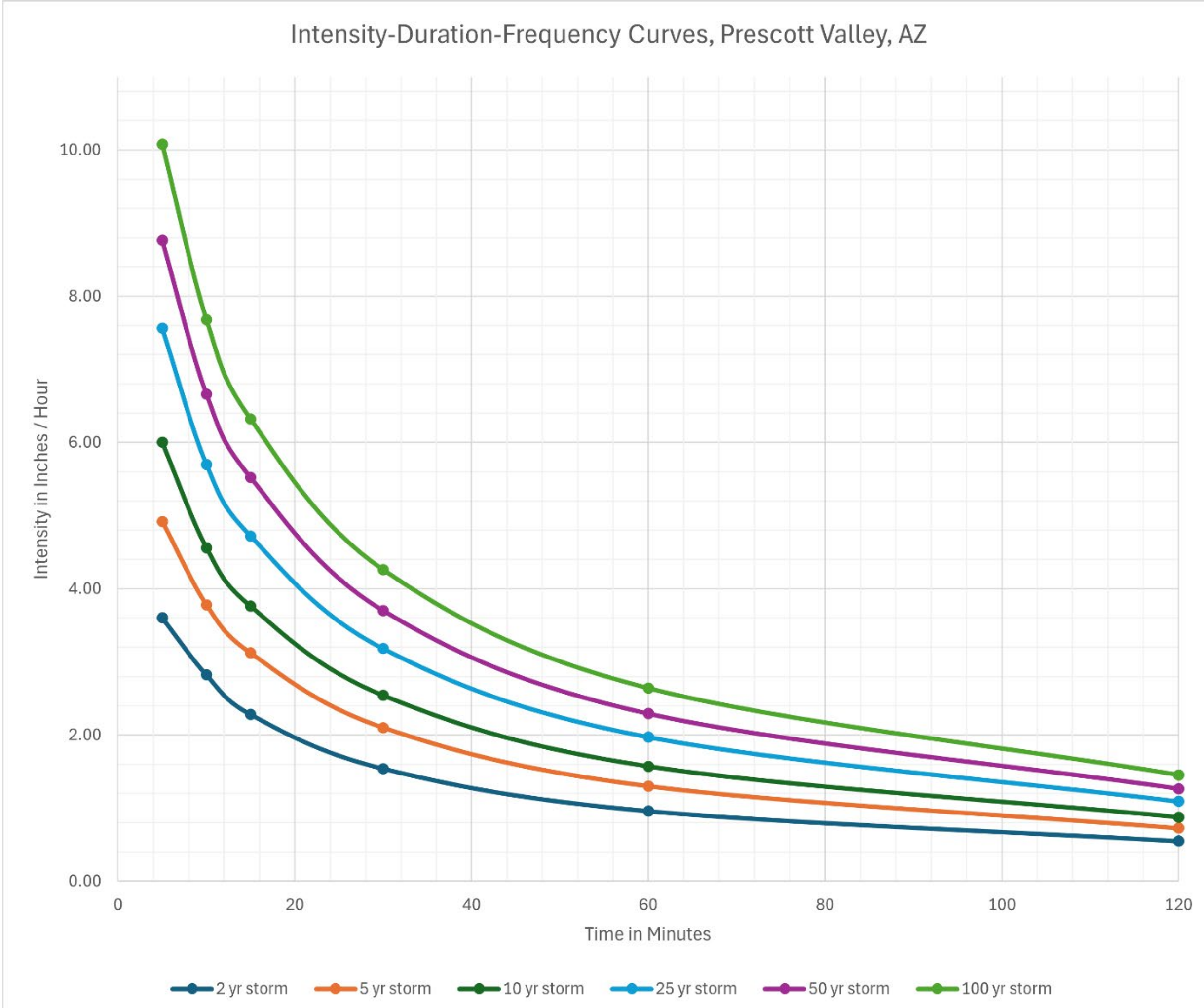

Figure 6-1. Rainfall Intensity-Duration-Frequency Curves

6.3.2 Depth-Area Relation

The rainfall depths provided by National Oceanic and Atmospheric Administration (NOAA) Atlas 14 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, compute the watershed centroid coordinate and then use PFDS to obtain a representative rainfall data set for the watershed.

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.

The default method for depth-area rainfall reduction in this Manual is the built-in method in HEC-HMS when using the Frequency Storm. That method is the same as in NOAA Atlas 2 which originated in TP-40. If the storm size is not specified in the Frequency Storm input, HEC-HMS reduces rainfall for each sub-basin individually based on its drainage area. Alternatively, the storm size can be input to a given value (normally the total watershed area) in which case HEC-HMS reduces the storm rainfall for all sub-basins based on the TP-40 reduction factor for the Storm Size area.

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.

6.3.3 Rational Method

6.3.3.1 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 Town implementation of the Rational Method are:

- 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).
- The peak runoff occurs when all parts of the drainage basin are contributing to the runoff.
- Rainfall is uniform over the watershed.
- 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.
- The calculated runoff is directly proportional to the rainfall intensity.
- 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:

$$Q = CiA \qquad \textbf{Equation 6-2}$$

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

A = the contributing drainage area, in acres

6.3.3.2 Procedure

Applicability

- The total drainage area must be less than or equal to 160 acres.
- T_c shall not exceed 60 minutes.
- The minimum T_c shall not be less than 10 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-HMS Program to estimate flood discharges.

General Considerations

- 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.
- 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).
- 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.
- Both C and the rainfall intensity (i) will vary if peak discharges for different flood return periods are desired.
- The T_c equation is a function of rainfall intensity (i); therefore, T_c will also vary for different flood return periods.

Estimation of Area

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.

Estimation of Rainfall Intensity (i)

The intensity (i) in Equation 6-2 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. Procedures for developing a site-specific I-D-F graph are also described in Section 6.3.1.

The intensity (i) in Equation 6-2 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 10-minutes is to be used if the calculated T_c is less than 10 minutes. The Rational Method should not be used if the calculated T_c is greater than 60 minutes.

Estimation of Time of Concentration (T_c)

Time of concentration (T_c) is to be calculated by the following equation:

$$T_c = 11.4\, L^{0.5}\, K_b^{0.52}\, S^{-0.31} i^{-0.38} \qquad \textbf{Equation 6-3}$$

Reference: (Papadakis & 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

i = the average rainfall intensity, in inches/hr, for a duration of rainfall equal to T_c (the same (i) as Equation 6-2) unless T_c is less than 10-minutes, in which case the (i) of Equation 6-2 is for a 10-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 the following equation:

$$S = \frac{H}{L}$$ **Equation 6-4**

where:

H = the change in elevation, in feet, along L

L = the length of the longest hydraulic flow path, in miles

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

$$S = 5280\left(\frac{d}{j}\right)^2$$ **Equation 6-5**

where:

d = the length of the longest flow path, in feet, 5280*L

$$j = \sum\left(\frac{d_i^3}{H_i}\right)^{\frac{1}{2}}$$ **Equation 6-6**

where:

d_i = an incremental change in length along the longest flow path, in feet

H_i = an incremental change in elevation for each segment d_i, in feet

Reference: (Pima County Department of Transportation and Flood Control District, 1979)

Values of resistance coefficient (K_b) may be selected from Table 6-4. Use of Table 6-4 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.

Table 6-4. Resistance Coefficient (K_b) for Use in the Rational Method T_c Equation

(Arizona Department of Transportation, 2014)

Description of Landform	K_b	
	Defined Drainage Network	Shallow Overland Flow Only
Mountain, with forest and dense ground cover (average slopes – 50% or greater)	0.015	0.30
Mountain, with rough rock and boulder cover (average slopes – 50% or greater)	0.12	0.25
Foothills (average slopes – 10% to 50%)	0.1	0.20
Alluvial Fans, Pediments and Rangelands (average slopes – 10% or less)	0.05	0.10
Irrigation Pastures [a]	--	0.20
Tilled Agricultural Fields [a]	--	0.08
Urban		
Residential, L is less than 1,000 ft [b]	0.04	
Residential, L is greater than 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 length in T_c equation. Streets serve as drainage network.

The solution of Equation 6-3 is an iterative process since the determination of (i) requires the knowledge of the value of T_c. Therefore, Equation 6-3 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 6-3 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).

Selection of Runoff Coefficient

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 Town's implementation of the Rational Method is focused on two key characteristics: land use and storm frequency. 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.

Runoff Coefficients for various land uses and storm frequency are included in Table 6-5. 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 the following equation:

$$C_{comp} = \frac{\sum_{i=1}^{n} C_i A_i}{A_t}$$ **Equation 6-7**

where:

C_{comp} = the area-averaged value of the runoff coefficient, 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

A_T = the total area of the sub-basin, in acres

Table 6-5. Rational Method General Runoff Coefficients

(Yavapai County, 2015)

Land Use Category [1]	Runoff Coefficients by Storm Frequency							
	2- to 10-year		25-year		50-year		100-year	
	min	max	min	max	min	max	min	max
Rural Residential	0.33	0.45	0.36	0.50	0.40	0.60	0.45	0.65
Estate Residential	0.42	0.48	0.46	0.55	0.50	0.64	0.53	0.70
Low Density Residential	0.48	0.65	0.53	0.72	0.58	0.78	0.60	0.80
Medium Density Residential	0.65	0.75	0.72	0.83	0.78	0.90	0.82	0.94
Medium-High Density Residential	0.65	0.75	0.72	0.83	0.78	0.90	0.82	0.94
Community and Regional Commercial	0.75	0.85	0.83	0.94	0.90	0.95	0.94	0.95
Industrial – Business	0.60	0.70	0.66	0.77	0.72	0.84	0.75	0.88
Industrial - Industrial	0.70	0.80	0.77	0.88	0.84	0.95	0.88	0.95
Sidewalks, Pavement and Roof Tops	0.75	0.85	0.83	0.94	0.90	0.95	0.94	0.95
Parks	0.20	0.35	0.25	0.40	0.30	0.45	0.35	0.50
Lawn and turf	0.10	0.25	0.11	0.28	0.12	0.30	0.15	0.35
Non-irrigated landscape	0.30	0.45	0.35	0.48	0.36	0.50	0.38	0.55
Rangeland, flat slope, vegetated	0.3	0.40	0.35	0.45	0.36	0.48	0.38	0.50
Rangeland, hill slope, vegetated	0.40	0.55	0.45	0.60	0.48	0.66	0.50	0.70
Mountain, steep slope, vegetated	0.50	0.70	0.65	0.80	0.70	0.90	0.75	0.90
[1] Land Use Category based on land use designations in the General Plan 2035.								

Volume Calculations

Rational method runoff volume estimations should be computed using Equation 6-8. In the case of volume calculations for stormwater storage facility design, P in Equation 6-8 equals the 100-year, 2-hour depth, in inches.

$$V = C\left(\frac{P}{12}\right)A \qquad \textbf{Equation 6-8}$$

where:

> V = runoff volume, in acre-feet
>
> C = runoff coefficient (or C_{comp})
>
> P = rainfall depth, in inches
>
> A = drainage area, in acres

If a runoff hydrograph is needed for design purposes, the Rational Method should not be used. The Unit Hydrograph Method (Section 6.3.5) should be used for that case.

6.3.3.3 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.

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.

C: Select the Runoff Coefficient (C) from Table 6-5. If the drainage area contains subareas of different runoff characteristics, and thus different C coefficients, arithmetically area-weight the values of C using Equation 6-7.

2. 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 6-5. If the profile is uniform, use Equation 6-4.
    c. K_b: Determine the K_b parameter from Table 6-4. If the drainage area contains subareas of different K_b values, arithmetically area-weight the values of K_b using Equation 6-7 and substituting K_b for C.
    d. I-D-F: Obtain the I-D-F table from either Table 6-3 or the NOAA Atlas 14 website.
3. 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 Tc using Equation 6-3. 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 10-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.

4. Q: Determine the peak discharge Q by using the above value of i in Equation 6-2.

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 6-2. 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 the Town 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 6-3. Referring to Figure 6-2 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.

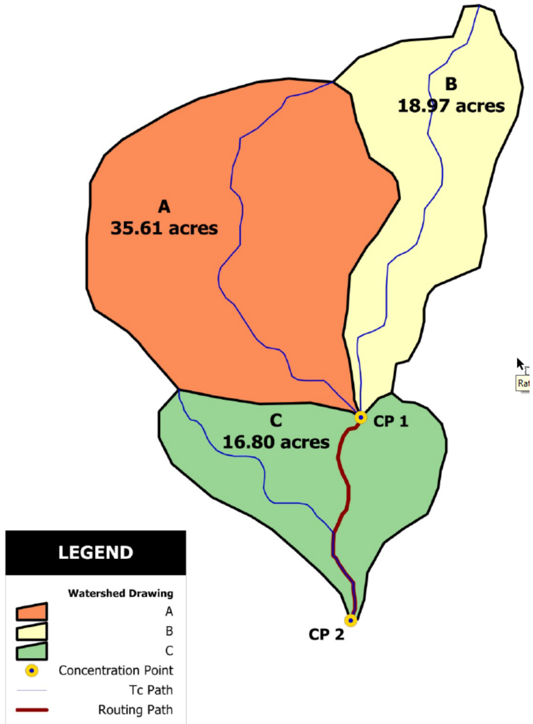

Figure 6-2. Example Multiple Basin Watershed

(Yavapai County, 2015)

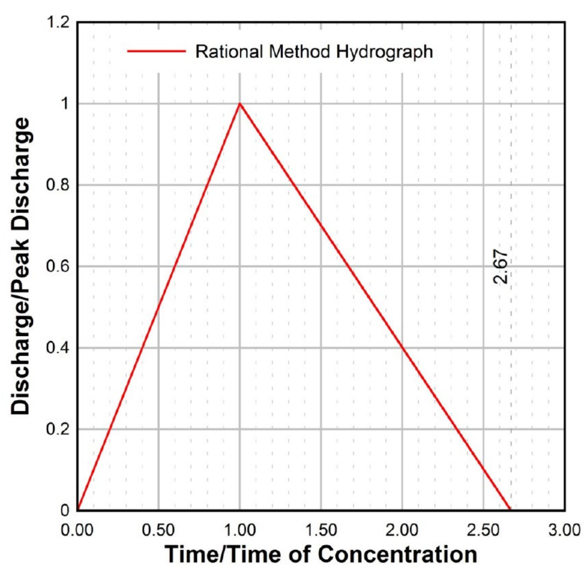

Figure 6-3. Triangular Hydrograph for use with the Rational Method.

(Yavapai County, 2015)

Note: Modified base on analysis of urban flow gage hydrograph data for short duration high intensity storms on watersheds less than 160 acres.

Instructions for Combined Watershed Approach

1. Compute the peak discharge for each individual sub-basin using steps 1 through 5 from the previous section.
2. Compute the arithmetically area-weighted value of C for combined sub-basins A and B.
3. Follow step 4 from the previous section 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 one the following two options (T_c cannot be greater than 60 minutes):
   a. Option 1 - Follow step 4 from the previous section to calculate the T_c for the single basin composed of all three sub-basins.
   b. Option 2 - Compute the travel time from CP 1 to CP 2 using the Manning's 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 the previous section. 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 the previous section.
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 6-3 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's 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.

6.3.4 Rainfall Losses

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.

The design rainfall is determined from the procedures in Section 6.3.1, 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 method shall be used to estimate rainfall losses: the Green and Ampt Method. This 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 1970s 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 (Bedient & Huber, 1988) and Chow, Maidment and Mays (Chow, Maidment, & Mays, Applied Hydrology, 1988).

6.3.4.1 Green and Ampt Method

Use of the Green and Ampt equation involves the simulation of rainfall loss as a two-phase process, as illustrated in Figure 6-4. The first phase is the simulation of the surface retention loss, and the second phase is infiltration. The first phase is modeled in two parts in HEC-HMS – the Canopy Method and Surface Method. The second phase of the rainfall loss process is the infiltration of rainfall into the soil matrix. In

FLO-2D the first phase is modeled using initial abstraction, which includes interception and depression storage, which must be filled prior to simulating the second phase, infiltration.

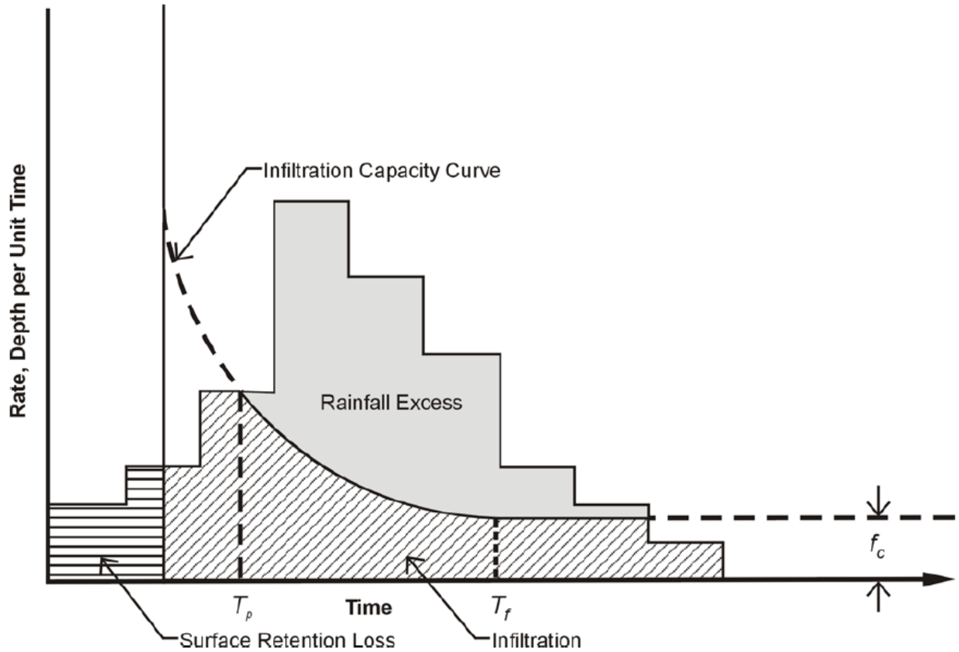

Figure 6-4. Simplified Representation of Rainfall Losses.

(Flood Control District of Maricopa County, 2023)

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 the Town with confidence, particularly those areas with more coarse grained soils, such as loam and sandy loam. The finer soil textures (those with 'clay" in the classification name) have relatively low infiltration rates. Use of the Green and Ampt 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. 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 should be calibrated if possible.

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 surface retention loss and volumetric storage of infiltrated rainfall may not be significant for large rainfall amounts associated with storms less frequent than the one percent storm.

Method Description

The first phase of the rainfall loss process is simulated with the surface retention loss parameter only. 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 the Town is discussed in the following sections.

Surface Retention Loss (Surface Method & Surface Tab)

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.

In HEC-HMS, the surface retention loss has two parameters specified on the Surface Tab– 'initial storage (%)' and 'max storage (in)'. For the purposes of this Manual, the 'max storage (in)' is to be taken as the sum of all initial losses including surface depression storage and interception losses. The initial storage (%) will generally be taken as 0 percent for drainage design applications. For saturated soil conditions such as agricultural fields or special forensic investigations, initial storage would be 100 percent. In FLO-2D, the surface retention loss is known as initial abstraction, which includes interception and depression storage.

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 (Tholin & Keefer, 1960), 0.11 inches for 1 percent slopes to 0.06 inches for 2.5 percent slopes (Viessman, Jr., 1967), and 0.04 inches based on rainfall-runoff data for an urban watershed in Albuquerque (Sabol G. V., 1983). In some estimates of surface retention losses during intense storms were measured as 0.20 inches for sand, 0.15 inches for loam, and 0.10 inches for clay (Hicks, 1944). Other estimates measured 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, Ward, & Seiger, 1982). Rainfall simulator studies in New Mexico result in estimates of 0.39 inches for eastern plains rangelands and 0.09 inches for pinon-juniper hill slopes (Sabol G. , et al., 1982). Chow (Chow, 1964) quotes Horton (Horton, 1935) as stating that initial detention "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 surface retention loss for various land uses and land surfaces is needed. Reference sources for values of surface retention loss for use with the Green and Ampt equation listed above are summarized in Table 6-6.

Surface retention loss is primarily a function of land use and surface cover, and recommended values for use with the Green and Ampt equation are presented in Table 6-7. The values have been selected to fit many typical settings in the Town; however, the engineer/hydrologist should always evaluate the specific circumstances in any particular area for hydrologic variations from these typical values.

Table 6-6. Estimated Surface Retention Losses

(Flood Control District of Maricopa County, 2023) & (Yavapai County, 2015)

Study	Physiography/Land use	Surface Retention Loss, inches
(Tholin & Keefer, 1960)	Turf	0.25 – 0.5
(Tholin & Keefer, 1960)	Impervious surfaces	0.0625 – 0.125
(Viessman, Jr., 1967)	1 percent slopes	0.11
(Viessman, Jr., 1967)	2.5 percent slopes	0.06
(Sabol G. V., 1983)	Urban area	0.04
(Hicks, 1944)	Sand	0.20
	Loam	0.15
	Clay	0.10
(Sabol, Ward, & Seiger, 1982)	Undeveloped alluvial plains	0.1 – 0.2
	Eastern New Mexico plain rangelands	0.39
	Pinion-juniper hillslopes	0.09
(Chow, 1964)	Flat areas	1/8 to ¾
	Cultivated fields, natural grass lands or forests	½ to 1.5

Table 6-7. Surface Feature Characterization Data

Type [1]	Description	IA inches	RTIMP %	VCD %	Initial Saturation	n-value
Rural Residential [2]	5-ac minimum with one dwelling unit (du) per lot	0.35	5	0	Dry	0.04
Estate Residential [2]	1 du/ac minimum to 1 du/4.99 ac maximum	0.35	10	0	Dry	0.04
Low Density Residential [2]	1.1 – 4 du/ac detached homes	0.25	30	0	Normal	0.04
Medium Density Residential [2]	4.1 – 8 du/ac, detached homes	0.25	60	0	Normal	0.04
Medium-High Density Residential [2]	8.1 – 15 du/ac, includes multi-family	0.15	80	0	Normal	0.04
Community Commercial [2]	Smaller scale business including retail, professional office, and service-oriented business	0.1	90	0	Dry	0.02
Regional Commercial [2]	Large retail shopping areas	0.1	90	0	Dry	0.02
Industrial – Business [2]	Office complexes, warehouses, storage	0.1	80	0	Dry	0.02
Industrial – Industrial [2]	Manufacturing, scrap lots, distribution	0.1	90	0	Dry	0.02
Sidewalks, Pavement and Rooftops	Sidewalks, Streets, parking lots	0.05	95	0	Dry	0.015
Mountain, Steep Slope, Vegetated	> 10% slope	0.25	0	0	Dry	0.05
Rangeland, Hill Slope, Vegetated	2 to 10% slope	0.15	0	0	Dry	0.045
Rangeland, Flat Slope, Vegetated	< 2% slope	0.35	0	0	Dry	0.04
Lawn and Turf	Golf courses, irrigated landscape	0.20	5	0	Normal	0.035
Parks	Recreational facilities	0.20	5	0	Dry	0.03
Non-Irrigated Landscape	Empty lots, road medians	0.20	0	0	Dry	0.03
Wash Bottom	Natural wash and river bottoms	0.25	0	0	Dry	0.04
Water	Lakes, canals, ponds	0.05	100	0	Saturated	0.015

[1] Type is based on the land use designations in the General Plan 2035.
[2] The Type does not include streets and sidewalks.

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. The Green and Ampt equation infiltration parameters are:

1. hydraulic conductivity at natural saturation (Conductivity or XKSAT),
2. wetting front capillary suction (Suction or PSIF),
3. volumetric soil moisture content or deficit at the start of rainfall (Initial Content or DTHETA), and
4. volumetric soil moisture content at saturation (Saturated Content).

The 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.

Soil survey data for the study area is prepared by the NRCS and ADOT complied soil mapping and associated data in shapefile format from the Soil Survey Geographic (SSURGO) and Digital General Soil Map of the United States (STATSGO) databases. For each soil map unit, Green and Ampt parameter values are calculated by ADOT and provided in shapefile format (Arizona Department of Transportation, 2014). These values are based on values of percent sand, clay, gravel organic matter, and porosity and use the equations based on the Saxton and Rawls method (Saxton, K.E. & Rawls, W.J., 2006). There are 45 unique soil map units within the study area. Descriptions of each soil map unit and the corresponding hydraulic conductivity and rock outcrop percentages are listed in Table 6-8. The spatial distribution of the soil map units with the watershed is illustrated in Figure 6-5. The computed Green and Ampt infiltration parameters are available in geographic information system (GIS) shapefile and tabular format (Appendix B) on the ADOT Drainage Design web page: (https://azdot.gov/business/engineering-and-construction/roadway-engineering/drainage-design/manuals-drainage-designhttp://www.azdot.gov/business/engineering-and-construction/roadway-engineering/drainage-design/manuals). (At the time of this manual update, the data provided in the shapefile format has fields for “DTHETA normal” and “DETHETA dry” switched.)

Soil map unit hydraulic conductivity values are evaluated based on the controlling soil horizon of the upper 6 inches of the soil. Hydraulic conductivity values for individual soil types are computed based on data in the NRCS soil surveys. Map unit values have been capped at 2 inches/hour to limit excessive rainfall losses.

The Saxton and Rawls (Saxton, K.E. & Rawls, W.J., 2006) equations produce a true saturated hydraulic conductivity, K_s. The Green and Ampt equation uses hydraulic conductivity at natural saturation. K_s must be adjusted to compute an estimate of effective saturated hydraulic conductivity using Equation 6-9. The adjustment factor is necessary to adjust for entrapped air and other physical effects of a natural setting that limit infiltration effectiveness.

$$XKSAT = AF * K_s$$ **Equation 6-8**

where:

XKSAT = effective saturated hydraulic conductivity, in in/hr

AF = adjustment factor

K_s = true saturated hydraulic conductivity, in in/hr

A single AF of 0.5 has been used to obtain XKSAT estimates given K_s (Rawls, W.J., Brakensiek, D.L., & Miller, N., 1983). The actual conversion factor is known to range from less than 0.1 to over 1, depending on many variables. Field measurements completed by the Desert Research Institute (DRI) on behalf of the FCDMC on test watersheds in Maricopa County have provided local data that could be used to develop an improved method for estimating XKSAT given K_s (Desert Research Institute, 2010), (Desert Research Institute, 2012), (Desert Research Institute, 2014), and (Desert Research Institute, 2016). The conversion of K_s to XKSAT is complex and potentially affected by many factors, including:

- sand, clay, gravel, and organic matter content,
- surface condition,
- vegetation,
- soil structure,
- air displacement,
- bulk density, and
- rainfall intensity.

There is research in this regard, but a definitive relationship between K_s and XKSAT that can be applied with existing data for a wide range of soil types has not been identified. Published papers include:

- (Bouwer, H., 1966)
- (Morel-Seytoux, H.J. & Kanji, J., 1974)
- (Ojha, R., Corradini, C., Morbidelli, R., & Govindaraju, R., 2017)

Given the large range in possible conversion factor values, establishing a more robust method for estimating adjustment factor (AF) is important for hydrologic model accuracy. This was attempted using the DRI field-measured data. Unfortunately, a reliable relationship could not be developed due to insufficient information. Therefore, a single AF of 0.5 is recommended for continued use in the Town.

The standard procedure is to the use the Green and Ampt parameters calculated by ADOT. However, the engineer/hydrologist is strongly encouraged to field verify the soil's 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 ADOT, a site-specific geotechnical investigation is encouraged. Such an investigation should include determination of the sand, clay, and gravel 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 Soil Water Characteristics computer program may then be used to compute an estimate of hydraulic conductivity for each soil type identified (Saxton, K.E. & Rawls, W.J., 2006). It should then be corrected using Equation 6-9 to obtain hydraulic conductivity.

Table 6-8. Soil Map Units

Map Unit	Description	XKSAT (in/hr)	Rock %
AbB	Abra-Lonti loams, 0 to 5 percent slopes	0.29	0
AnC	Abra-Wineg association, rolling	0.18	0
AuC	Arp gravelly clay loam, 0 to 20 percent slopes	0.02	0
AvD	Arp cobbly clay loam, 10 to 25 percent slopes	0.02	0
AwE	Arp very rocky clay loam, 20 to 40 percent slopes	0.02	20
AxD	Arp-Moano complex, 0 to 30 percent slopes	0.05	0
BgD	Balon gravelly sandy clay loam, 5 to 30 percent slopes	0.11	0
BmF	Barkerville cobbly sandy loam, 20 to 60 percent slopes	0.63	0
BnD	Barkerville very stony sandy loam, 5 to 25 percent slopes	0.28	0
BoF	Barkerville extremely rocky sandy loam, 20 to 60 percent slopes	0.63	20
CaD	Cabezon-Springerville complex, 5 to 25 percent slopes	0.02	0
CbC	Cabezon-Springerville cobbly complex, 5 to 15 percent slopes	0.02	0
DaF	Dandrea gravelly loam, 20 to 60 percent slopes	0.21	0
GdD	Gaddes gravelly sandy loam, 3 to 25 percent slopes	0.09	0
LkD	Lonti gravelly sandy loam, 15 to 30 percent slopes	0.61	0
LmB	Lonti gravelly loam, 0 to 8 percent slopes	0.20	0
LnC	Lonti cobbly loam, 0 to 15 percent slopes	0.16	0
LnF	Lonti cobbly loam, 30 to 60 percent slopes	0.16	0
LoD	Lonti complex, 2 to 30 percent slopes	0.44	0
LpB	Lonti-Abra gravelly sandy loams, 0 to 8 percent slopes	0.50	0
LrD	Lonti-Abra complex, 8 to 30 percent slopes	0.26	0
LsC	Lonti-Pastura complex, 0 to 20 percent slopes	0.17	0
LuC	Lonti-Wineg complex, 3 to 15 percent slopes	0.33	0
LwD	Luzena cobbly loam, 0 to 30 percent slopes	0.13	0

Map Unit	Description	XKSAT (in/hr)	Rock %
Ly	Lynx soils	0.15	0
Ly2	Lynx soils, eroded	0.09	0
MbF	Mirabal gravelly sandy loam, 20 to 60 percent slopes	0.50	0
MdF	Mirabal-Dandrea complex, 20 to 60 percent slopes	0.30	0
MgD	Moano gravelly loam, 0 to 30 percent slopes	0.18	0
MkF	Moano very rocky loam, 15 to 60 percent slopes	0.18	20
PmB	Pastura-Lynx association, undulating	0.16	0
Ro	Rock land	0.28	0
Rs	Rough broken land	0.27	0
S382	Lynx-Lonti-Balon	0.16	0
S419	Mollic Eutroboralfs	0.22	0
S482	Spudrock-Rombo-Rock outcrop	0.18	30
Sa	Sandy and gravelly alluvial land	0.90	0
ShB	Showlow gravelly sandy loam, 0 to 8 percent slopes	0.47	0
SlB	Springerville cobbly clay, 0 to 8 percent slopes	0.02	0
SnD	Springerville-Cabezon complex, 3 to 30 percent slopes	0.02	0
SuB	Springerville-Lonti association, undulating	0.06	0
TdC	Thunderbird cobbly clay loam, 0 to 15 percent slopes	0.02	0
TdE	Thunderbird cobbly clay loam, 15 to 40 percent slopes	0.02	0
W	Water	0.01	100
Wn	Wineg-Abra complex	0.16	0

There are 19 unique land use categories in the watershed. The category type and brief description are provided in Table 6-7. The spatial distribution of the land use categories is illustrated in Figure 6-6. Data used to compute the parameters of the Green and Ampt infiltration equation are Initial Abstraction (IA), Percent Impervious (RTIMP), Vegetation Cover Density (VCD) and the initial saturation of the watershed at the start of a rainfall event. Values assigned to each land use category for each component are based on general conditions within the Town and are listed in Table 6-7.

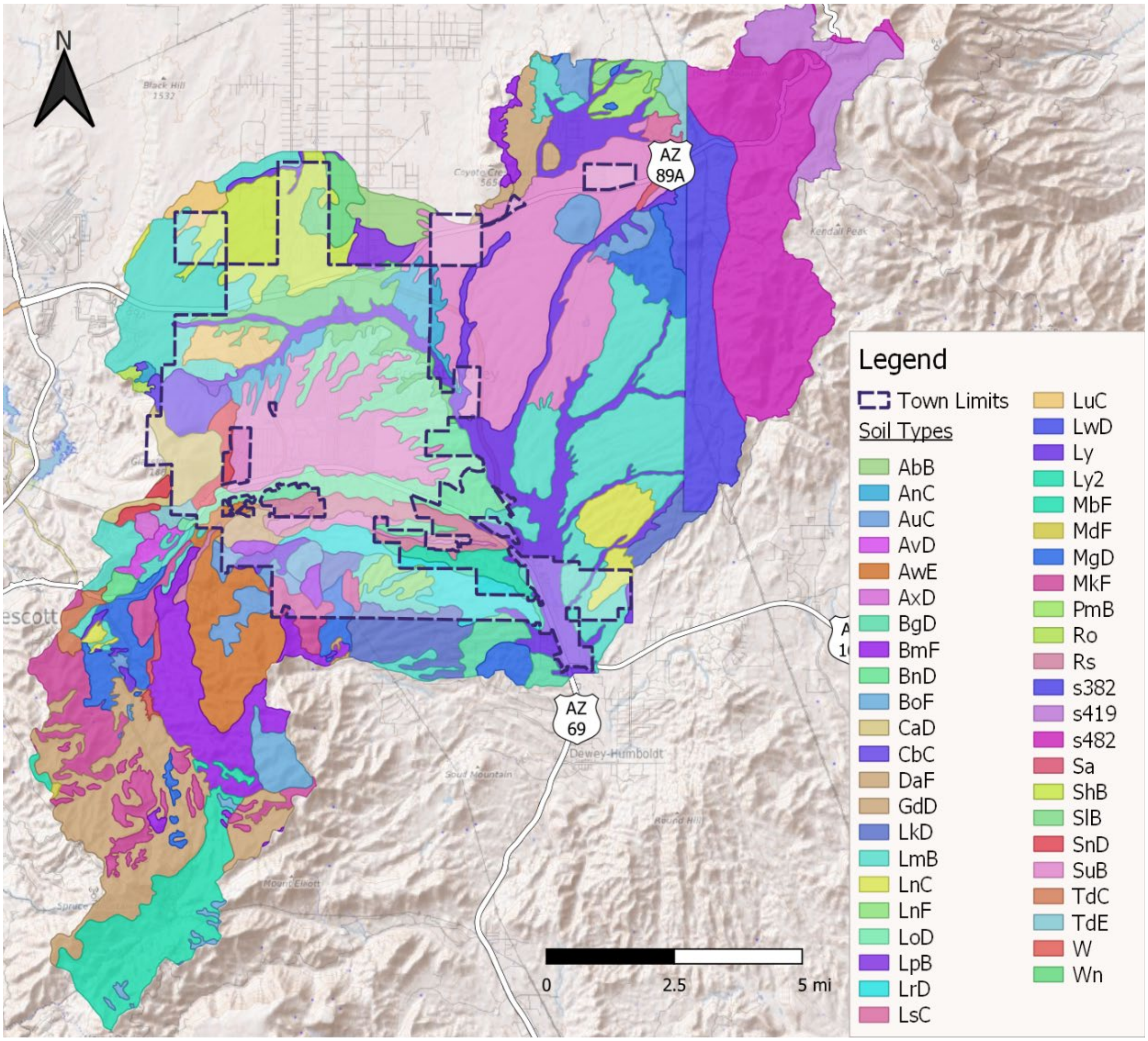

Figure 6-5. Soil Map Units.

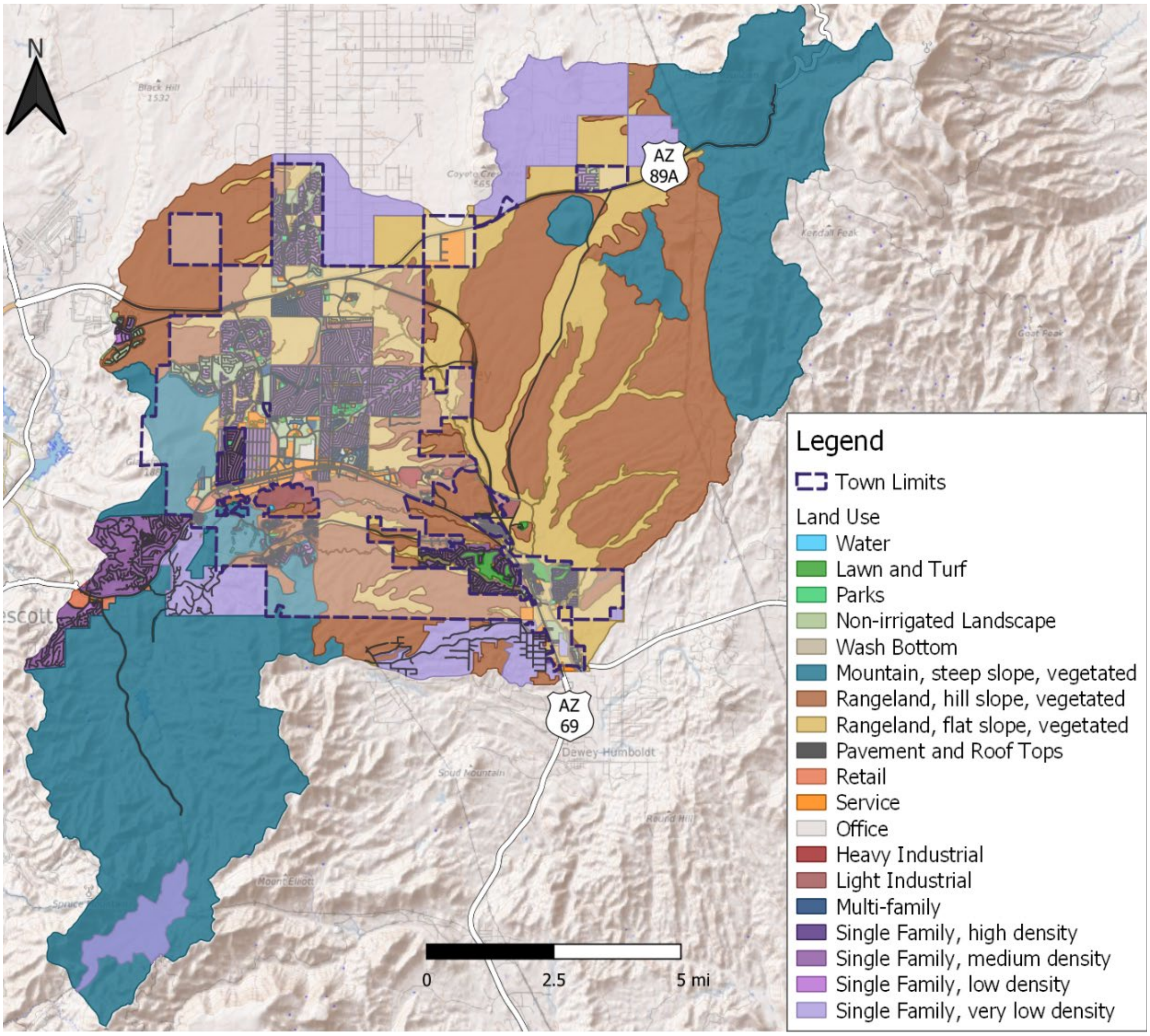

Figure 6-6. Land Use Designation.

Adjusting Bare Ground Conductivity for Vegetation Cover

The bare ground value of conductivity 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 VCD for each land use category provides a means to adjust the assigned infiltration rate for the effects of vegetation. While some portions of the watershed have a high VCD, particularly the areas in the upper Lynx Creek and Clipper Wash watersheds, the VCD is set to zero for all land use types. For flood estimation purposes it cannot be assumed that the vegetation will be in any particular state at the time of storm occurrence and, therefore, it is set to zero. All other parameter values are left at the default assignment from the ADOT tables.

Effective Impervious Area

Impervious area (or nearly impervious area) is composed of rock outcrop, paved roads, parking lots, roof tops, driveways, sidewalks, compacted gravel, artificial turf, and other areas where either asphalt, concrete, brick, stone, or other paving materials are used. Final determination of impervious area will be decided by the Town. When performing watershed modeling with the HEC-HMS program, the impervious area is to be the effective (directly connected) impervious area. For HEC-RAS and FLO-2D the impervious area can be spatially varying and does not need to be directly connected. For urbanized areas, the impervious area should be estimated from aerial photographs with guidance as provided in Table 6-7. The engineer or hydrologist may use the standard default values from Table 6-7. If values other than the standard defaults are more appropriate, the engineer/hydrologist should estimate new values based on estimates of actual impervious area 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.

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. Impervious area from rock outcrop is provided for each soil map unit in Table 6-8. In regard to HEC-HMS, the percent rock outcrop for soil units is not necessarily effective impervious area. 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 percent effective impervious area if the intended result includes a conservative estimate of rainfall runoff volume. Often, the impervious area value for such areas is reduced by a factor determined using engineering judgment. For HEC-RAS and FLO-2D, impervious area due to rock outcrop or areas of water should be included in the model.

For watersheds that have significant, contiguous rock outcrop, it may be necessary to establish those areas as separate sub-basins in HEC-HMS 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 an effective impervious area unless the road serves as a conveyance to the watershed outlet. However, for HEC-RAS and FLO-2D, paved roads and other impervious areas should be included in the model. See the manuals for the software for guidance on how to develop impervious areas.

Procedures for HEC-HMS are provided below. Procedures for using a 2D model are provided in Section 6.12.

6.3.4.2 *Procedure for Green and Ampt Method Using HEC-HMS*

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 or model. 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 Computer-aided Design and Drafting (CADD) or other software applications. 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. Sub-basin Delineation. 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. Subarea Delineation. Delineate subareas for each sub-basin for the purpose of assigning surface retention (Max Storage) and effective impervious estimates. The polygons from NRCS soil surveys delineating soil map units also are subareas, and often are used as subareas for estimation of surface retention and effective impervious, based on soil characteristics. Create GIS polygon coverages for each subarea and calculate the area of each subarea within each sub-basin.
3. Subarea Parameters. Assign estimates of surface retention and effective impervious for each subarea.
4. Estimate Composite surface retention loss (Max Storage) for each Sub-basin.
5. Estimate Composite Impervious (%) for each Sub-basin.
6. Estimate Bare Ground Conductivity for each Soil Map Unit (subarea).
7. Estimate Composite Bare Ground Conductivity for each Sub-basin.
8. Estimate Suction and Initial Content and Saturated Content for each Sub-basin.
9. Estimate Adjusted Composite Conductivity for each Sub-basin.
10. HEC-HMS Surface Tab. Enter the composite values of Max Storage and Initial Storage for the drainage area or each sub-basin.
11. HEC-HMS Loss Tab. Enter the composite values of Initial Content, Saturated Content, Suction, adjusted Conductivity, and Impervious (%) for the drainage area or each sub-basin.

Instructions for Sub-basin Composite Max Storage

1. Assign a Max Storage 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 Max Storage 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 section.

2. Compute a Composite Value of Max Storage: If there are multiple subareas within a sub-basin, calculate an area-weighted value of Max Storage using the following equation:

$$\overline{MaxStorage} = \left(\frac{\sum A_i MaxStorage_i}{A_T}\right)$$ **Equation 6-9**

where:

$\overline{MaxStorage}$ = Composite value of Max Storage in inches

$MaxStorage_i$ = IA of each subarea, in inches

A_i = size of IA subarea

A_T = size of the watershed or sub-basin

Instructions for Sub-basin Composite Conductivity

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 Conductivity value and then color code the Conductivity polygons.
3. Compute a Composite Bare Ground Conductivity Value for each sub-basin: Standard values of Conductivity for each soil map unit are listed in Table 6-8 or can be obtained from the Town as a shapefile. Use ArcMap/ArcGIS Pro to compute the area of each soil map unit within each sub-basin. Then either use ArcMap or ArcGIS Pro to apply Equation 6-10, or export the soil map unit number, Conductivity value, and area information for each sub-basin into a Microsoft Excel spreadsheet and apply Equation 6-10 within the spreadsheet. When performing these computations by hand, planimeter each color-shaded polygon to obtain the total area of each Conductivity value within the sub-basin. Then apply Equation 6-10 by hand or within a spreadsheet.

$$\overline{Conductivity} = antilog\left(\frac{\sum A_i logConductivity_i}{A_T}\right)$$ **Equation 6-10**

where:

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

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

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

A_T = total area of the watershed or sub-basin

Instructions for Sub-basin Composite Suction

Composite soil suction is computed using a log-averaging method in the same manner as hydraulic conductivity, simply replace Conductivity with Suction.

Instructions for Initial and Saturated Content

1. Assign the Appropriate Initial Soil Moisture Condition: Assign land uses for the watershed area to either the dry (wilting point) or normal (field capacity) initial soil moisture condition category per Table 6-7. Undeveloped areas should be assigned the dry (wilting point) Initial Content. Developed areas with irrigated landscaping should be assigned to the normal (field capacity) Initial Content.
2. Intersect the GIS Coverages: Use the ArcMap/ArcGIS Pro 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 and land use areas. 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. Then repeat with the land use coverage. Alternatively, the order may be reversed (i.e., intersect land use and soils then watersheds). The results are soil map unit polygons completely contained within each sub-basin polygon with the initial content category also assigned.
3. Simplify the GIS Soils Coverage: Use the ArcMap/ArcGIS Pro 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 Initial and Saturated Content values and then color code the polygons.
4. Compute a Composite Initial and Saturated Content Value for each sub-basin: Standard values of Initial Content for each soil map unit are listed in Table 6-7. Use ArcMap/ArcGIS Pro to compute the area of each soil map unit within each sub-basin. Then either use ArcMap/ArcGIS Pro to apply Equation 6-11, or export the soil map unit number, Initial and Saturated Content value and area information for each sub-basin into a Microsoft Excel spreadsheet and apply Equation 6-11 within the spreadsheet. When performing these computations by hand, planimeter each color-shaded polygon to obtain the total area of each Initial and Saturated Content value within the sub-basin. Composite soil moisture contents for both initial and saturated condition are computed using a simple area-weighted procedure as shown in the following equation:

$$\overline{SMC} = \frac{\sum A_i SMC_i}{A_T}$$ **Equation 6-11**

where:

$\overline{SMC}$ = composite soil moisture content

SMC_i = soil moisture content of the soil in a subarea

A_i = size of a subarea

A_T = size of the drainage area or modeling sub-basin

Instructions for Sub-basin Composite Impervious (%)

1. Assign an Impervious (%) Estimate to Sub-basin Subareas: Sub-basins may have to be divided into subareas based on land use and/or surface characteristics. Impervious (%) 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 impervious area (% $Impervious_N$) is defined separately from developed condition impervious area (% $Impervious_D$). A composite % Impervious is computed for each sub-basin for natural and developed conditions. Then the two are added to obtain a total composite % Impervious 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 Impervious (%) for natural areas. Standard values of natural Impervious (%) are provided for each soil map unit in Table 6-8 and as a shapefile but should be field verified if significant.

2. Compute a Composite Value of Impervious (%): If there are multiple subareas within a sub-basin, calculate an area-weighted value of Impervious (%) using the following equation. Use this equation for computing both % $Impervious_N$ and % $Impervious_D$.

$$\overline{Impervious\%} = \frac{\sum A_i Impervious\%_i}{A_T}$$ **Equation 6-12**

where:

$\overline{Impervious\%}$ = composite Impervious %

Impervious $\%_i$ = Impervious % on the soil in a subarea

A_i = size of the subarea

A_T = size of the drainage area or modeling sub-basin

6.3.5 Unit Hydrographs in HEC-HMS

6.3.5.1 Introduction

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 6.3.5.2.

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 the Town is the Clark unit hydrograph. The Clark unit hydrograph method uses two numeric parameters; Time of Concentration (T_c) and Storage Coefficient (R), and a graphical parameter, the time-area relation. Procedures are provided herein to estimate the Clark unit hydrograph parameters for HEC-HMS. Unit hydrograph procedures other than the Clark procedure can be used for specific applications; however, this will require justification and approval by the Town for such use.

6.3.5.2 Applicability

The Clark unit hydrograph, as described herein, can be used for virtually any watershed that will be encountered in the Town. 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-HMS by use of a user-specified unit hydrograph. 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 the Town and approval received for deviations from the recommended procedures before incorporating such deviations into the project hydrology analysis.

Equations are 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 Equation 6-13 through Equation 6-15 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 6-16) is a function of T_c, the R value should be recalculated if T_c is adjusted for a given return period.

The Unit Hydrograph Method is normally applied for watersheds greater than 160 acres in area and when a runoff hydrograph is needed.

6.3.5.3 Procedure

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

$$T_c = 2.4A^{0.1}L^{0.25}L_{ca}^{0.25}S^{-0.2}$$ **Equation 6-13**

Agricultural Fields

$$T_c = 7.2A^{0.1}L^{0.25}L_{ca}^{0.25}S^{-0.2}$$ **Equation 6-14**

Urban

$$T_c = 3.2A^{0.1}L^{0.25}L_{ca}^{0.25}S^{-0.14}RTIMP^{-0.36}$$ **Equation 6-15**

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

RTIMP = effective impervious area, in percent

In using Equation 6-13 through Equation 6-15, 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 0.5L.

5. RTIMP is the effective impervious area. This is the same value that was determined for the watershed by the procedures in Section 6.3.4.1. RTIMP is used to estimate T_c for urban watersheds only (Equation 6-15).

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 (Equation 6-13 through Equation 6-15) 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 (Equation 6-13 through Equation 6-15) is selected based on the watershed type that contains the greatest portion of L.

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 the following:

$$R = 0.37T_c^{1.11}L^{0.80}A^{-0.57}$$ **Equation 6-16**

where:

R = storage coefficient, in hours

T_c = time of concentration, in hours

L = length of watercourse to the hydraulically most distant point, in miles

A = area, in square miles

Time-Area Relation

The time-area relation defines the accumulated area of the watershed that is contributing runoff to the outlet of the watershed at any point in time. Either a synthetic time-area relation must be adopted or the time-area relation for the watershed must be developed. In general, use of a synthetic time-area relation is acceptable for watersheds within the corporate limits as well as watersheds contributing runoff to areas within the corporate limits. Three synthetic time-area relations are recommended for use.

Elliptical Watershed Approximation: this relation, in general, assumes that at 50% of the overall travel time (T_c), 50% of the area is contributing runoff at the watershed outlet. This time-area relation is defined by the following equation and shown graphically in Figure 6-7.

$$\frac{A_t}{A} = \begin{cases} 1.414\left(\frac{t}{T_c}\right)^{1.5} & For\ t \le \frac{T_c}{2} \\ 1 - 1.414\left(1 - \frac{t}{T_c}\right)^{1.5} & For\ t \ge \frac{T_c}{2} \end{cases}$$

Equation 6-17

where:

A_t = cumulative watershed area contributing at time t, in square miles

A = total watershed area, in square miles

t = travel time, in hours

T_c = watershed time of concentration, in hours

Undeveloped Watershed Approximation: derived during the reconstitution of recorded rainfall-runoff events for undeveloped watersheds in the southwest United States (data compiled as part of the development of the Flood Control District of Maricopa County Hydrology Manual). The undeveloped watershed time-area relation is shown in Figure 6-7 and the percentage curve is defined in Table 6-9.

Developed Watershed Approximation: derived during the reconstitution of recorded rainfall-runoff events for developed watersheds in Arizona (data compiled as part of the development of the Flood Control District of Maricopa County Hydrology Manual). The developed watershed time-area relation is shown in Figure 6-7 and the percentage curve is defined in Table 6-9.

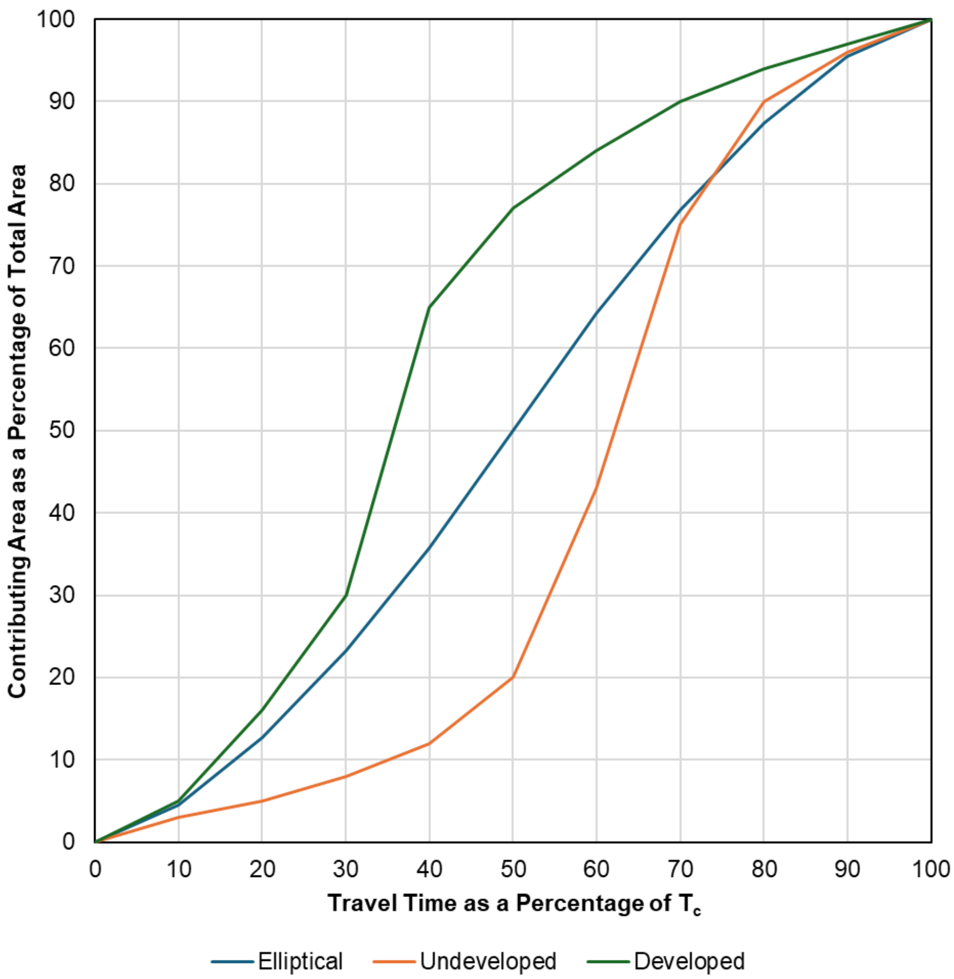

Figure 6-7. Time-Area Relations

Table 6-9. Time-Area Relations

Time as a Percentage of T_c	Contributing Area, as a Percentage of Total Area		
	Elliptical	Undeveloped	Developed
0	0	0	0
10	4	3	5
20	13	5	16
30	23	8	30
40	36	12	65
50	50	20	77
60	64	43	84
70	77	75	90

Time as a Percentage of T_c	Contributing Area, as a Percentage of Total Area		
	Elliptical	Undeveloped	Developed
80	87	90	94
90	96	96	97
100	100	100	100

If a synthetic time-area relation is not used, the time-area relation is developed by dividing the watershed into incremental runoff producing areas that have equal incremental travel times to the outlet location. This is a difficult task to implement manually. For these cases, the ModClark transform method in HEC-HMS is to be used. The ModClark method, as described in the HEC-HMS User Manual, is a quasi-distributed approach based on the Clark unit hydrograph that represents the watershed as a collection of grids. Instead of a time-area relation, the ModClark method uses a travel time index for each cell that is scaled by the overall T_c. The travel time index is calculated internally by HEC-HMS using a flow accumulation grid and the selected discretization method for the watershed.

Model Time Interval Requirements

The duration of the unit hydrograph (or all unit hydrographs in a multiple sub-basin model) is related to the model time interval. In HEC-HMS, the model time interval is specified under "Control Specifications" in the Time Interval pull down. In general, the model time interval will be selected according to the following criteria:

1. Time interval = 5 minutes for a 24-hour storm duration.
2. Time interval should not exceed 0.25 T_c for the sub-basin with the shortest T_c.

However, there may be special situations where the following additional rules should be considered in the selection of the model time interval:

1. Time interval = 0.15 T_c provides adequate definition of the hydrograph peak with an optimum number of hydrograph coordinate calculations.
2. Time interval = 0.25 T_c is the maximum value for the time steps.
3. Time interval 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.

6.3.5.4 Instruction 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 6.3.4.1),
   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. A = area, in square miles
   b. L = length of watercourse to the hydraulically most distant point, in miles
   c. L_{ca} = length measured from the concentration point along L to a point on L that is perpendicular to the watershed centroid, in miles
   d. S = watercourse slope, in feet/mile
   e. RTIMP = effective impervious area, in percent
7. Calculate T_c depending on the type of watershed, desert/mountain, agricultural fields, and urban using Equation 6-13 through Equation 6-15.
8. Calculate R using Equation 6-16.
9. Enter the values of T_c and R on the Transform tab for the watershed or each sub-basin.

10. Select the appropriate synthetic time-area relation or for cases where the ModClark transform method is used, consult the HEC-HMS User Manual for the input requirements.
    a. The Elliptical Watershed Approximation is the default option for HEC-HMS. No other input is required other than specifying the time-area relation as the default option.
    b. Undeveloped or Developed Watershed Approximation
        i. Create a Percentage Curve table from the Paired Data Manager
        ii. Select the newly created table and input the corresponding data from Table 6-9.
        iii. For the Time-Area Method, select the Percentage Curve table

6.3.6 Channel Routing in HEC-HMS

6.3.6.1 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-HMS, 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. Three methods are recommended for channel routing applications in the Town—the Muskingum-Cunge method, the Kinematic Wave method, and the Modified Puls method.

6.3.6.2 Applicability

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. When channel routing travel time is shorter than the model computation interval, the routing may be excluded from the model structure.

The Muskingum-Cunge routing method will be used in most instances. If significant backwater effects exist within a reach, the Modified Puls method should be used. It can also be used if channel-discharge relationships are readily available. The Kinematic Wave method may also be used for routing through uniform constructed channels. HEC-HMS Technical Reference Manual (U.S. Army Corps of Engineers, 2024) provides additional guidance on selection of appropriate routing methods.

6.3.6.3 Procedure

The following sections present the implementation of the Muskingum-Cunge, Kinematic Wave, and Modified Puls routing methods in HEC-HMS. The routing method is selected in the Routing Method on the Reach Tab in HEC-HMS. The input for the Reach Tab are:

Reach Description

The reach description is a user-defined/input description of reach.

Downstream

The downstream model element to which the routing reach connects is specified in the Downstream pulldown. For a routing reach the downstream element is typically a basin concentration point.

Routing Method

The selected routing method is selected in the Routing Method pulldown, e.g., Muskingum-Cunge, Kinematic Wave, or Modified Puls.

Loss/Gain Method

Typically, the Loss/Gain Method is set to "none." The Loss option can be used when transmission losses need to be included (see Section 6.3.8).

Muskingum-Cunge

In HEC-HMS, channel routing data are input on the Routing tab. The Routing tab contains the required and optional input to be specified for each routing reach. For the Muskingum-Cunge routing method, the following data input are required:

Time Step Method

The Time Step Method is a new parameter available in HEC-HMS. It is recommended that the Automatic Fixed Interval method be selected for use with the methods presented in this Manual.

Routing Reach Length (Length)

This is the length of the channel or major flow path. The length will be measured on the best available map. The units of reach length are feet.

Energy Grade Line Slope (Slope)

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 Slope are ft/ft.

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. Values of Manning's n-value can be assessed using information presented in Maricopa County Hydraulics Chapter 7.

When using the 8-point cross section input, Manning's n-values need to be specified for the channel and the left and right overbanks separately. If a simple shape is used (e.g., trapezoidal), a single composite n-value is specified.

Invert Elevation

The invert elevation may be specified in this field. The flow depth will be added to the invert elevation in the HEC-HMS output. Reach stage-time output plots are also available when the invert elevation is specified. When using transmission losses, the invert elevation is also used as the elevation to compute the area available for percolation losses.

Shape/Cross Section

The routing reach shape is specified in the shape pulldown. Normally, an eight-point cross section will be specified. The cross section must be input in the Paired Data Manager and then selected in the Cross Section pulldown on the Routing tab. Simpler cross section shapes - Circle, Triangle, Rectangle, and Trapezoid - may also be selected when appropriate.

Kinematic Wave

Again in HEC-HMS, the routing method is selected from the Routing Method pull down on the Reach tab. The Routing tab contains the required and optional input to be specified for each routing reach. In addition to reach length, slope, Manning's n-value, and channel invert elevation, the following additional data input are required for the Kinematic Wave routing method:

Subreaches

The subreaches parameter default value is 2 but may be optionally increased to assist the program in computation of the correct distance step used in the internal routing calculations.

Shape/Cross Section

Available shapes are circle, deep, rectangle, trapezoid, and triangle. The 'deep' option is for flow conditions where the flow depth is approximately equal to the flow width.

Modified Puls

Again in HEC-HMS, the routing method is selected from the Routing Method pull down on the Reach tab. For the Modified Puls method, the Routing tab includes:

Storage-Discharge (Stor-Dis) Function

The storage-discharge function is a table relating routing reach storage (in acre-feet) to outflow discharge (in cfs). This table is added under the Paired Data Manager. If a HEC-RAS model of the reach is available, a storage-discharge relation can be computed with a multiple profile run.

Subreaches

The number of subreaches in the Modified Puls routing method can be estimated using:

Number of subreaches = routing reach length/(average velocity x Model time interval).

Initial

The initial condition represents the flow in the channel at the start of the routing computation. The Manual recommends the use of a dry channel (Discharge = 0 cfs) for channels in Arizona unless a regular base flow is appropriate. This can be modeled in HEC-HMS by specifying the Initial condition as “Inflow = Outflow” on the Routing tab.

Elevation-Discharge (Elev-Dis) Function

The Elevation-Discharge function is a table relating routing reach elevation, or stage, (in feet) to outflow discharge (cfs). This table is added under the Paired Data Manager. If a HEC-RAS model of the reach is available, an elevation-discharge relation can be computed with a multiple profile run.

Invert

The invert is an optional input that specifies the lowest elevation in the routing reach. The flow depth is computed from the invert elevation to the routed flow elevation. The invert elevation also is used to compute the area for percolation when transmission losses are included with the Loss/Gain Method.

6.3.6.4 Instructions for Channel Routing

The following steps should be used with the Muskingum-Cunge routing method. Many of these steps are applicable to the Kinematic Wave and Modified Puls method as well.

1. From the watershed base map, identify the routing reaches. (See Section 6.3.9.2, 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.

5. Estimate the main channel roughness coefficient using the information and procedures in Maricopa County Hydraulics Chapter 7.

6. If an 8-point cross section is used that contains overbank floodplains, select the n for each of the overbanks, also using the information and procedures in Maricopa County Hydraulics Chapter 7.
7. Measure the routing reach length from the base map.
8. Estimate the energy gradient by calculating the channel bed slope from the base map.
9. Input the routing information into the Routing Tab for each routing reach.

6.3.7 Storage Routing

6.3.7.1 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-HMS input data to describe the storage capacity and discharge relations of the structure and its outlet works.

6.3.7.2 Applicability

Routing of runoff hydrographs through volumetric storage structures.

6.3.7.3 Procedure

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 or elevation and corresponding storage volume, or 2) water stage and its corresponding surface area for the water stored to that elevation. Either method is acceptable and to some extent the selection depends upon the information that is available. If surface area data are provided, the storage volume is calculated during the execution of the HEC-HMS program.

Stage-Discharge Relation

relation describing the discharge through the structure as a function of stage of water behind the structure must be provided. Discharges are entered into the Paired Data Manager that corresponds to water stages in the Elevation-Discharge Function. Stage-discharge relations are established by hydraulic analysis of the structure or from design reports. Computer software, such as HY-8 (Federal Highway Administration), can be used to facilitate computation of stage-discharge relations. Stage-discharge relations can also be computed by the HEC-HMS program by describing the hydraulic features of the outflow structures. The outflow structures can include culverts, spillways, overtopping, or pumps. Elevations, geometry, and discharge coefficients are entered for each outflow structure type selected.

Structure Overtopping

Structure overtopping can be modeled in HEC-HMS as an outflow structure using the Dam Tops option on the Reservoir Tab when the Outflow Structures method is selected. Level and non-level overtopping weirs can be described for each Dam Top specified. For the non-level option, a cross section must be defined in the Paired Data Manager.

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 also be included in HEC-HMS as an outflow structure associated with a reservoir. Like Dam Tops, pumps are selected on the Reservoir Tab and parameterized on the Pump Tab. Pump station operation where multiple pumps and/or variable pump capacity is required to be modeled cannot be adequately modeled with HEC-HMS. In such cases, more sophisticated pump station models should be used. The HEC-HMS 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.

Instructions for Storage Routing

1. Define the stage-storage relation from the most appropriate maps and input the relation in Elevation-Storage or Elevation-Area Functions in the Paired Data Manager.

2. Define the stage-discharge relation for the outflow through the structure by use of an Elevation-Discharge Function in the Paired Data Manager. The use of the Outflow Structures Method on the Reservoir tab can alternatively be used to define hydraulic outflows from a reservoir. Hydraulic structure types include outlets (such as culverts), spillways, overtopping (aka Dam Tops), and pumps.
   The recommended approach is to use Elevation-Discharge Function to define the complete discharge rating curve for all types of discharge through (or over) the structure. These input calculations should be performed 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-HMS program is adequate for the analysis, provide pump station information on the Pump Tab using the Outflow Structure method.

6.3.8 Transmission Losses

6.3.8.1 Introduction

Storm runoff and floods in the Town 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. 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, 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 Town and the procedure and assumptions clearly documented.

Transmission losses can be modeled within HEC-HMS via the Percolation Loss/Gain option available for use with the Muskingum-Cunge and Modified Puls routing options. These two options are specified under the Reach element by selection of the “Loss/Gain Method” as “Percolation” or “Constant” respectively. The Town recommends use of the “Percolation” Loss/Gain Method to simulate transmission losses when applicable. Percolation rates in cfs/acre are approximately equal to inches/hour. Therefore, hydraulic conductivity values for transmissive channel bottoms can be used to estimate percolation rates for use in the Loss/Gain Method when applicable. Range of values provided in Table 6-10 for various bed material types.

The second method, ‘Constant” loss, can also be used with the Muskingum-Cunge or Modified Puls 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 the constant loss rates.

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.

6.3.8.2 Applicability

This approach should not normally be applied to design studies.

This approach may be applied for flood insurance studies, but only after careful consideration of all the points listed in Section 6.3.8.3. Use of this approach requires prior approval by the Town.

6.3.8.3 Procedure

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-HMS 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 channel routing reach information if the Muskingum-Cunge method is used or input of the necessary normal depth storage routing information if the Modified Puls method is used.

The transmission loss will be calculated using the percolation rate on the Loss/Gain tab for each routing reach. Very little guidance is available for estimating the percolation rates, which can vary from more than 100 inches per hour to less than an inch per hour. Table 6-10 provides some guidance for the percolation rate that can be expected in channels of various bed materials. If using the eight-point cross section option or a stage-discharge function as part of the routing input, the elevation of the channel invert should correspond to the lowest elevation used in the 8-point cross section or stage-discharge function for that routing reach.

The hydraulic conductivity values for the NRCS soil map units covering the wash in question can also be used as an estimate for Percolation Rate. However, the engineer/hydrologist should be aware that the values presented in Section 6.3.4.1 and ADOT, 2014, are based on a maximum value for of 2.0 inches per hour. The true value of hydraulic conductivity for sands and gravels, estimated using the equations in Saxton and Rawls (2006), should be used for estimating transmission losses. These values should correspond well with the general guidelines presented in Table 6-10.

Table 6-10. Percolation Rates for Various Channel Bed Materials

(Natural Resources Conservation Service, 2007)

Bed Material	Transmission Loss Class	Percolation Rate Inches/hour ≈ cfs/acres
Very clean gravel and large sand	Very High	>5
Clean sand and gravel, field conditions	High	2.0 – 5.0
Sand and gravel mixture with low silt-clay content	Moderately High	1.0 – 3.0
Sand and gravel mixture with high silt-clay content	Moderate	0.23 – 1.0
Consolidated bed material; high silt-clay content	Insignificant to Low	0.001 – 0.10

6.3.9 Modeling Techniques Using HEC-HMS

6.3.9.1 Introduction

General Discussion

Practical application of the rainfall-runoff modeling procedures in this manual can be accomplished through use of the HEC-HMS Hydrologic Modeling System. This computer program is available from the USACE Hydraulic Engineering Center website. This section contains an overview of the major theoretical assumptions upon which the HEC-HMS computer program is based, and the resultant limitations. A modeler's/reviewer's HEC-HMS checklist can be provided by the Town, for use in developing and reviewing HEC-HMS 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-HMS program.
3. Procedures presented in this manual.

Applicable HEC-HMS Versions

There are many versions of the HEC-HMS 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-HMS program was originally developed by the USACE, Hydrologic Engineering Center (HEC) in 2000. Since that time, there have been several significant updates and numerous error corrections.

The current version of HEC-HMS at the time of this Manual was Version 4.12. The information presented herein pertains specifically to this version.

Assumptions and Limitations of HEC-HMS

Proficiency in use of the HEC-HMS 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-HMS 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-HMS 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. Hydrologic Routing: All routing (channel and storage) is by hydrologic methods. Hydraulic routing (the use of the St. Venant equations) is not performed.
2. 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).

6.3.9.2 Watershed Modeling

Instructions for Watershed Modeling Process

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

1. Collect all pertinent information for the watershed:
    a. maps,
    b. aerial photographs,
    c. soil surveys/data,
    d. land use maps/data/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 map(s)/data and most practical map scale.
3. Perform a preliminary sub-basin delineation using best available topographic data and aerial photographs.
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-HMS input file.
12. Execute the HEC-HMS model.
13. Debug and calibrate the model, where possible.
14. Evaluate the model and results based on available information.
15. 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.
16. Review the input data and make adjustments as necessary.
17. Execute the final HEC-HMS model.
18. Make final model verifications and evaluations.
19. Prepare a report.

HEC-HMS Model Logic

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).

The model diagram is also depicted within the HEC-HMS input. Use of GIS data can facilitate more accurate and easily understood visual presentation of the model logic structure within the HEC-HMS input data itself. This data can be added as background layers in the Basin Model view in HEC-HMS.

Model Simulation Time and Computation Time Interval

The model simulation time period and computation time interval are specified in the Control Specifications in HEC-HMS.

The model simulation time period should span at least 24 hours to cover the entire rainfall event duration. Additional time may be required for larger watersheds where routing of the peak discharge through the entire model area extends beyond a 24-hour period. Model hydrographs should be plotted and examined at the downstream model limits to determine if a sufficient model time period has been simulated.

Generally, a model time interval of 5 minutes will be used. However, the time interval should also be checked against the recommendations based on the shortest time of concentration (T_c) as discussed in Section 6.3.5.3.

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:

Concentration Points

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

Estimate the time of concentration (T_c) for the smallest sub-basin. Using this value, determine the integer number of minutes for the computation interval. A computation interval of 0.15 * T_c will provide adequate definition of the hydrograph peak. The computation interval should not exceed 0.25 * T_c for the sub-basin with the shortest T_c. A computation interval of 5 minutes will usually be adequate to meet both criteria.

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-HMS model is based.

Routing Lengths

The length of the channel reaches defined as a result of the sub-basin delineation should be considered while breaking down the watershed. If short reaches are required in the watershed subdivision, combine hydrographs directly rather than route through a reach that is too short.

The Muskingum-Cunge method recommended in this manual uses the automatic fixed time interval method which does not require the number of subreaches to be input.

Precipitation and Rainfall Distributions

For a multiple sub-basin model, the storm area must be specified in the Frequency Storm so that the correct rainfall depth-area reduction factor will be applied. Normally this is the total drainage area to the point of design at the model outlet. If design discharges are needed at internal concentration points within the basin model, then either several different models will need to be developed (one for each concentration point of interest) or the Depth-Area Analysis option can be used. The internal points of interest can be added as analysis points to obtain the correct areal reduction for rainfall to each point of interest. The HEC-HMS output must be carefully examined to obtain the correct results for each point of interest when using the Depth-Area Analysis option. Instructions in the HEC-HMS User's Manual for use of the Depth-Area Analysis option in conjunction with the Frequency Storm should be consulted.

Rainfall Losses

Keep in mind that the rainfall loss parameters are averages, assumed to be evenly distributed, for the sub-basin. The percent effective impervious value 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, the effective impervious area 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 effective impervious area for rock outcrop.

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 feet per second (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 four-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

Compute the overland flow travel time with the following equation:

$$T_{OF} = \frac{0.007(nL)^{0.8}}{P_2^{0.5} s^{0.4}} \quad \textbf{Equation 6-18}$$

where:

T_{OF} = overland flow travel time, in hours

n = overland flow roughness

L = overland flow length, in feet

P_2 = 2-year, 24-hour rainfall, in inches

s = overland flow slope, in feet/foot

Equation 6-18 is taken from NRCS Urban Hydrology for Small Watersheds TR-55 (Natural Resources Conservation Service, 1986). Guidelines for selecting the overland flow roughness (n) are provided in the NRCS reference, as well as in the HEC-HMS Technical Reference Manual. Overland flow lengths are generally less than 300 feet.

Junction and Diversion Operations

The primary hydrograph operations available with the HEC-HMS 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:

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:

The split is done using a discharge rating table for the diversion with a maximum volume cutoff option.

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.

It is very important to check the shape of diversion hydrographs for oscillations and to verify that the expected results are obtained.

When a diverted hydrograph is reconnected downstream, the drainage area associated with the hydrograph is zero.

Hydrograph Channel Routing Operations

The preferred channel routing method is the Muskingum-Cunge routing method. It will be the routing method of choice for most applications. In cases of uniform constructed channels, the Kinematic Wave method may also be used. Finally, in instances of significant backwater effects, the Modified Puls method should be used. Some general considerations when implementing these channel routing methods are discussed in the following sections.

Number of Subreaches

The Muskingum-Cunge method recommended in this manual uses the automatic fixed time interval method which does not require the number of subreaches to be input.

The Kinematic Wave method requires an initial estimate of the number of subreaches to determine the correct distance step used during the routing calculations. The default value in HEC-HMS is 2 but may be optionally increased if needed.

When using the Modified Puls routing method for channel routing, the user must input the number of subreaches. The number of subreaches affects attenuation where one subreach produces the greatest attenuation and a large number produces little or no attenuation. The number of subreaches should be determined as a function of the reach length, travel time, and computation interval. A good estimate of the number subreaches is to divide the total reach length by the flow velocity and the computation interval. Remember to account for proper units. Reach length is given in feet, flow velocity in feet/second. Therefore, computation interval must also be converted to seconds. The actual travel time computed by HEC-HMS should be compared to the assumed flow velocity and the number of subreaches adjusted if needed.

Channel Geometry

When using the Muskingum-Cunge method, an eight-point cross section may be specified to describe the routing reach. Considerations for selection of the appropriate cross section, which should be checked by field reconnaissance when possible, 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.

Reservoir Routing

Modeling of reservoirs and detention basins can be accomplished using a Reservoir element in HEC-HMS. It is recommended that low level outlets, spillways, and structure overtopping be modeled using an elevation-discharge rating curve input in the Paired Data Manager. The rating curve should be developed using appropriate calculation methods including using an appropriate computer program such as HEC-RAS, HY-8, manual methods.

6.3.10 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 generally sparse in Arizona, but in or near the Town there are a few watercourses with relatively long periods of record. The engineer/hydrologist with a need to perform a flood frequency analysis for watersheds in or near the Town is referred to (U.S. Geological Survey, 2019), *Applied Hydrology*, Chapter 12, “Frequency Analysis” (Chow, Maidment, & Mays, Applied Hydrology, 1988), and ADOT Hydrology Manual, Chapter 10, “Flood Frequency Analysis” for detailed procedures. The US Geologic Survey’s (USGS) National Streamflow Statistics (NSS) Application and USACE’s HEC-SSP software can assist with the calculation. Also, the USGS StreamStats program can be used.

6.3.11 Indirect Methods for Discharge Verification

6.3.11.1 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-HMS 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 or around the Town.

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.

6.3.11.2 Applications and Limitations

The three indirect methods can be applied to any watershed in or around the Town, 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 database 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 overgrazing,

f. transmission losses that may occur in the watercourses,

g. the existence of distributary flow areas, and

h. upstream water regulation or diversion.

6.3.11.3 Procedure

Three procedures are provided for obtaining indirect estimates of peak discharges for watersheds in the Town:

1. a graph of numerous unit peak discharge versus drainage area curves,
2. a graph 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 and around the Town.

Indirect Method No. 1 - Unit Peak Discharge Curves

Figure 6-8 presents eight 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) (Creager, Justin, & Hinds, 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).

H - An envelope curve of maximum peak discharges from USGS gages in Arizona from Methods for Estimating Magnitude and Frequency of Floods in Arizona, Developed with Unregulated and Rural Peak-Flow Data through Water Year 2010 (Paretti, N.V., Kennedy, J.R., Turney, L.A., & Veilleux, A.G., 2014). A cloud of common values this curve is based on is shown on Figure 6-9.

When using Figure 6-8, 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 the Town are C, D, and G.

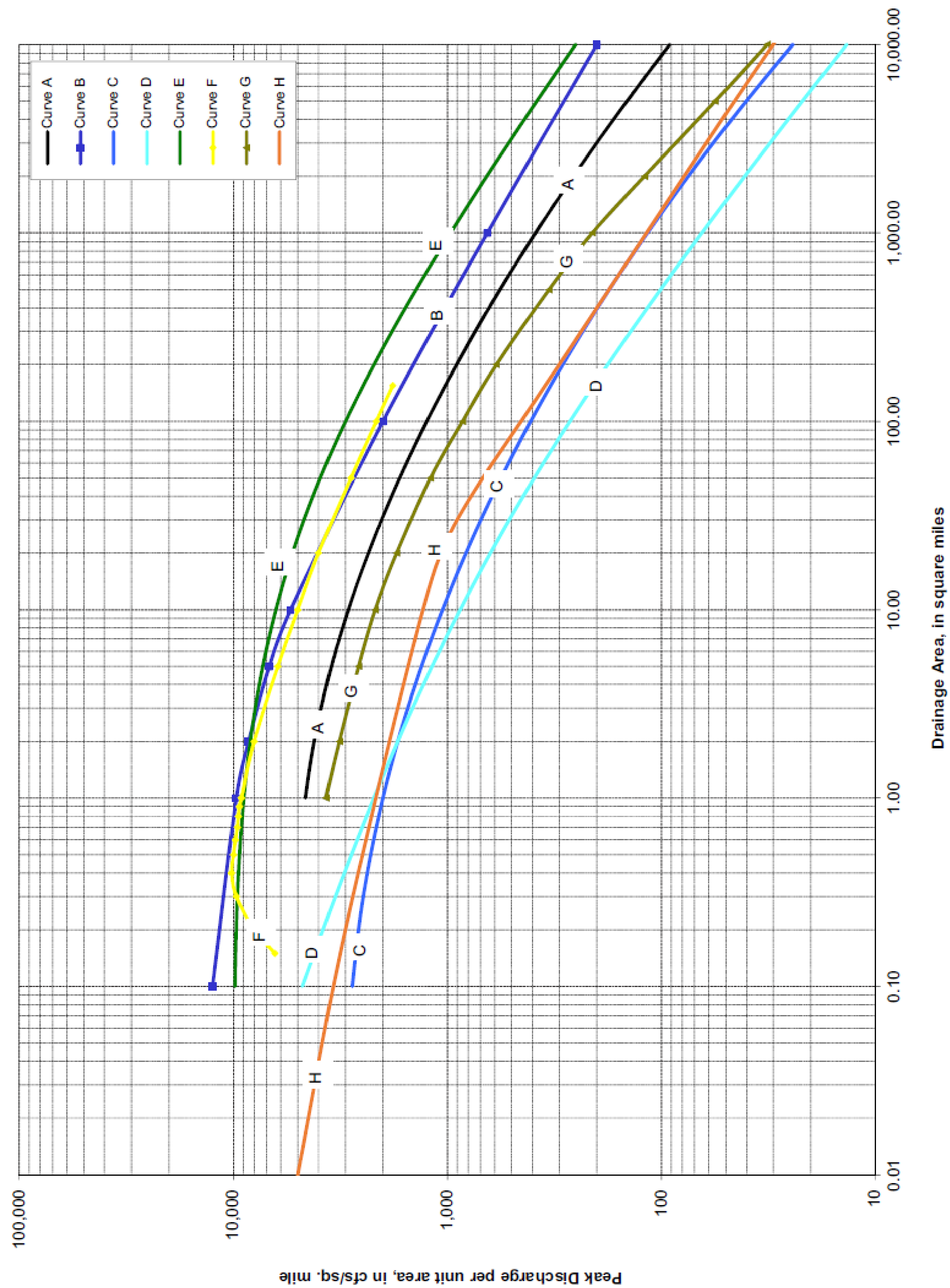

Figure 6-8. Extreme Event Peak Discharge Relations and Envelope Curves

(Flood Control District of Maricopa County, 2023)

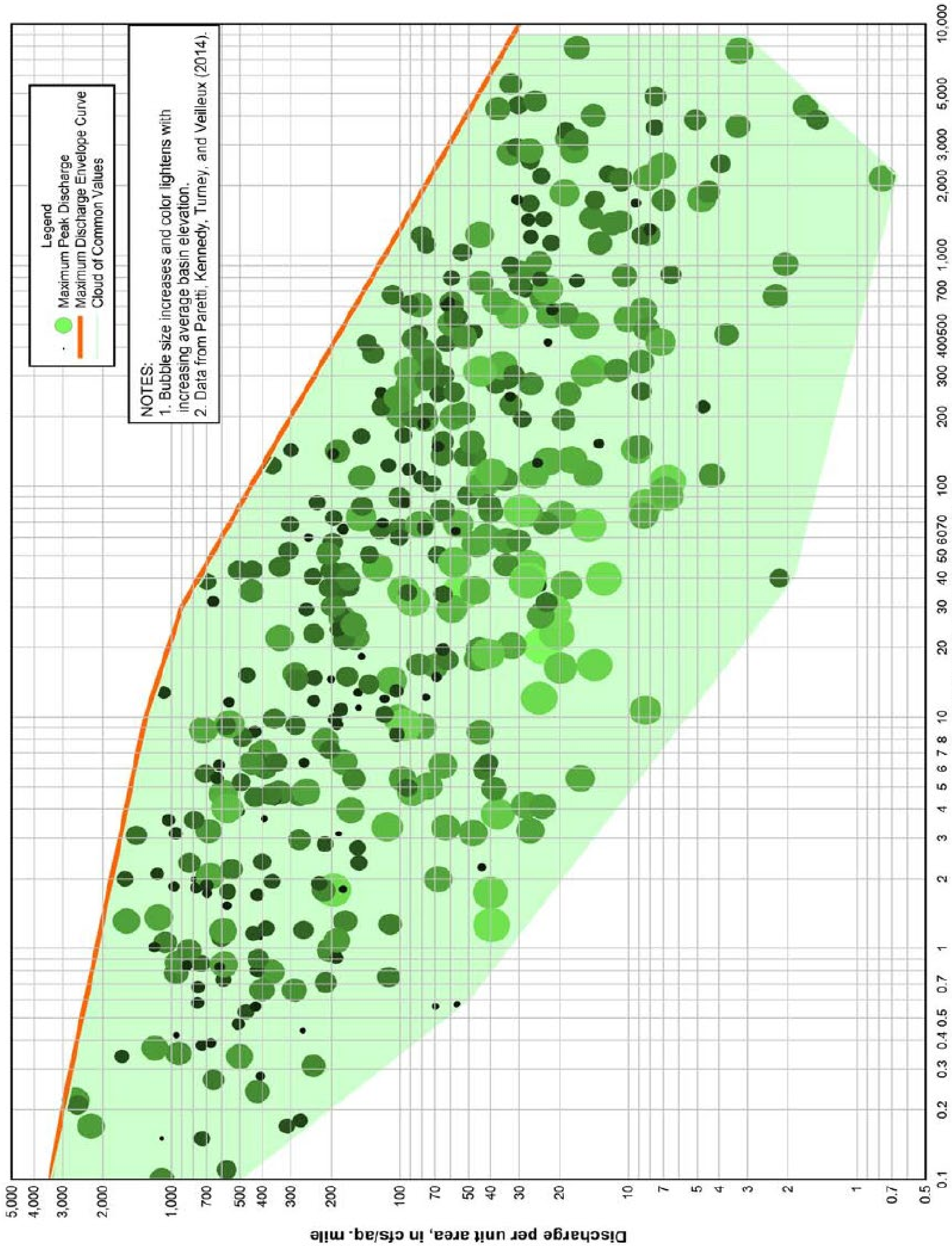

Figure 6-9. Cloud of Common Values for AZ Max Peak Discharge

(Paretti, N.V., Kennedy, J.R., Turney, L.A., & Veilleux, A.G., 2014) *and* (Flood Control District of Maricopa County, 2023)

Indirect Method No. 2 – USGS Data for Arizona

Indirect Methods Numbers 2 and 3 are based on Paretti, Kennedy, Turney, and Veilleux (Paretti, N.V., Kennedy, J.R., Turney, L.A., & Veilleux, A.G., 2014). Data can be obtained from the USGS StreamStats website for Arizona. The USGS collected annual peak-flow data through water year 2010 compiled from 448 unregulated streamflow-gaging stations in Arizona, hereafter referred to as streamgages, having a minimum of 10 years of record.

Per (Paretti, N.V., Kennedy, J.R., Turney, L.A., & Veilleux, A.G., 2014), the USGS first computed flood frequency estimates with station (or at-site) skew using the Expected Moments Algorithm with a multiple Grubbs-Beck test to identify multiple potentially influential low flows to fit a Pearson Type III distribution. Next, a multiple step Bayesian least-squares-regression approach was used to determine a new statewide regional skew of −0.09. No basin characteristics analyzed were statistically significant in explaining the variation in skew and as a result, the constant model was chosen as the best regional skew model for the Arizona study area. The mean square error used in Bulletin 17B (B17B) of the Interagency Advisory Committee on Water Data was used to describe the precision of the regional skew (Interagency Advisory Committee on Water Data, 1982). The constant model had a mean square error equal to 0.08, which corresponds to an effective record length of 85 years. This is a marked improvement over a previous Arizona regional skew analysis from USGS Water Supply Paper 2433, Thomas, Hjalmarson, and Waltermeyer (Thomas, B.E., Hjalmarson, H.W., & Waltermeyer, S.D., 1997), which reported a mean square error of 0.31, for a corresponding effective record length of approximately 17 years. Thus, the new regional model had almost five times the information content (as measured by effective record length) of that calculated in Thomas, Hjalmarson, and Waltermeyer (1997), or the value of 0.302 reported in the B17B generalized skew map.

The flood frequency estimates were recalculated using a weighted skew of the station and regional skew. Station flood frequency estimates for each stream gage were published for the 50-, 20-, 10-, 4-, 2-, 1-, 0.5-, and 0.2-percent annual exceedance probabilities in Paretti, Kennedy, Turney, and Veilleux (2014) along with the maximum recorded discharge for each station. Plots of peak discharge versus drainage area for stations with drainage areas smaller than 10,000 square miles are shown on Figure 6-10 through Figure 6-15 for the 2-, 10-, 25-, 50-, 100-, and 500-year storm frequencies. A nonlinear best-fit line was created by FCDMC for this data for each storm frequency. The Upper and Lower 10% Points Line were created parallel to the Nonlinear Regression Best Fit Line, such that 10% of the points are located outside the lines. The equations for the best-fit line for each storm frequency are listed in Equation 6-19 through Equation 6-23.

$$Q_2 = (37.297)A^{0.617}$$ **Equation 6-19**

$$Q_{10} = (179.148)A^{0.600}$$ **Equation 6-20**

$$Q_{25} = (305.531)A^{0.582}$$ **Equation 6-21**

$$Q_{50} = (477.836)A^{0.549}$$ **Equation 6-22**

$$Q_{100} = (670.977)A^{0.526}$$ **Equation 6-23**

$$Q_{500} = (1090.053)A^{0.560}$$ **Equation 6-24**

where:

Q = peak discharge, in cubic feet per second.

A = drainage area, in square miles

Also shown on Figure 6-10 through Figure 6-15 are the 90 percent upper and lower confidence bounds about the Log-Pearson Type 3 (LP3) frequency discharge line from Equation 6-19 through Equation 6-24. A listing of the data that was used to produce Figure 6-10 through Figure 6-15 is included in the FCDMC Hydrology Manual, Appendix E (Flood Control District of Maricopa County, 2023). The listing includes USGS streamflow-gaging station numbers, the associated drainage areas and the 2-, 10-, 25-, 50, 100-, and 500-year flood peak discharge estimates using a LP3 probability distribution.

Watershed characteristics for each of these gaging stations are provided in Paretti, Kennedy, Turney, and Veilleux (2014). A map of Arizona showing the locations of the gaging stations for this data compilation are shown in Figure 6-16.

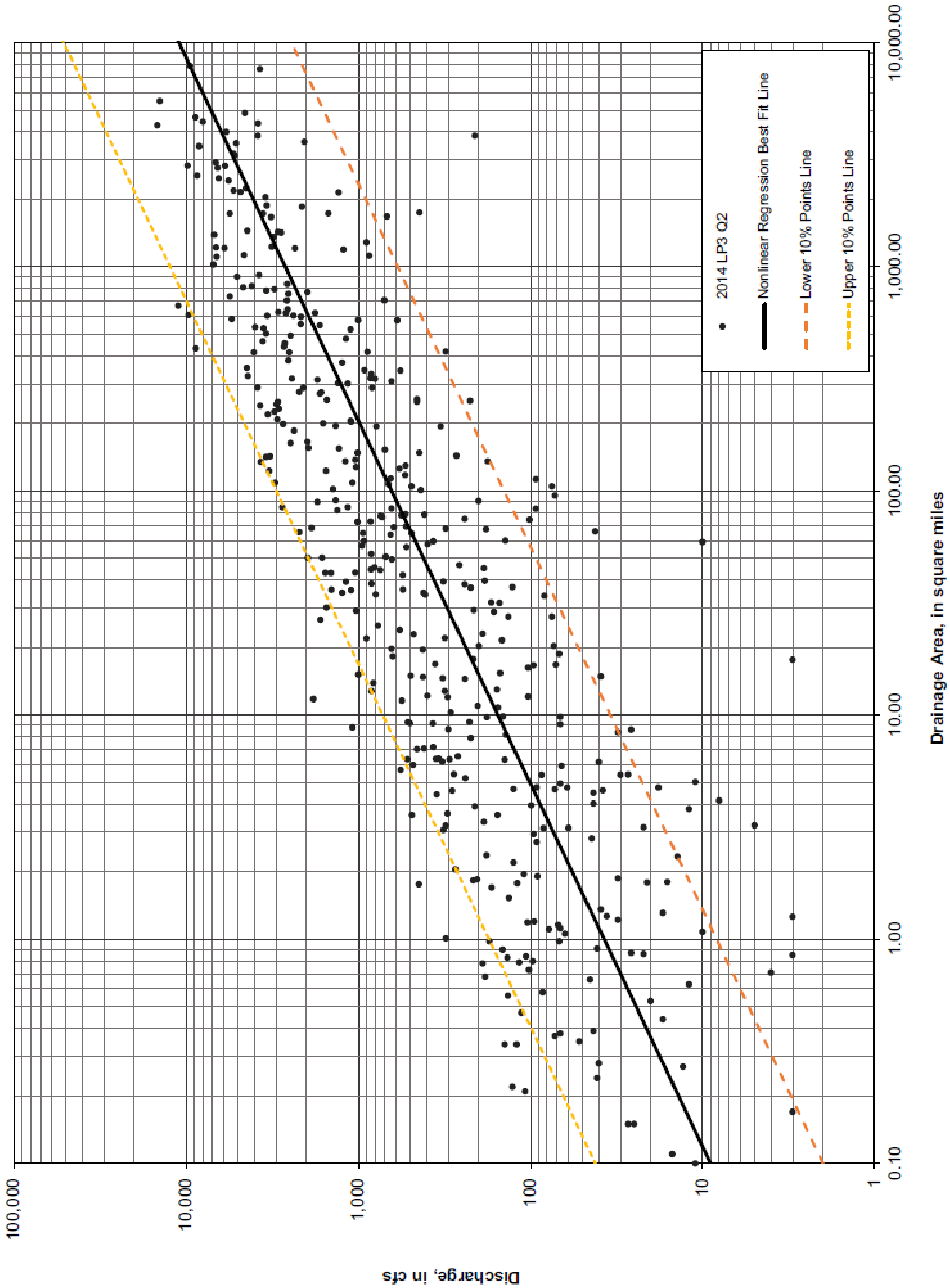

Figure 6-10. 2-Year Peak Discharge by LP3 Analysis

(Flood Control District of Maricopa County, 2023)

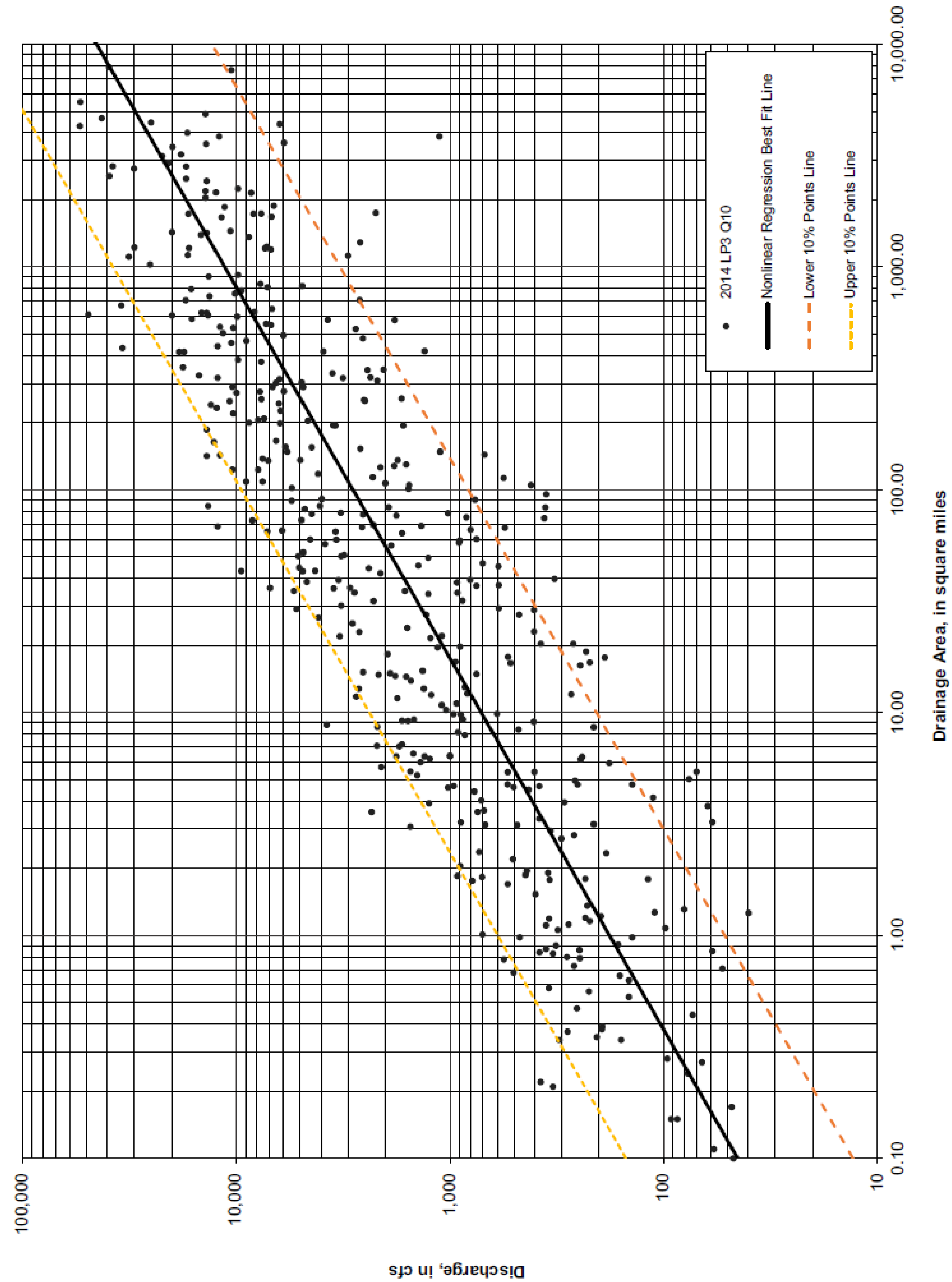

Figure 6-11. 10-Year Peak Discharge by LP3 Analysis

(Flood Control District of Maricopa County, 2023)

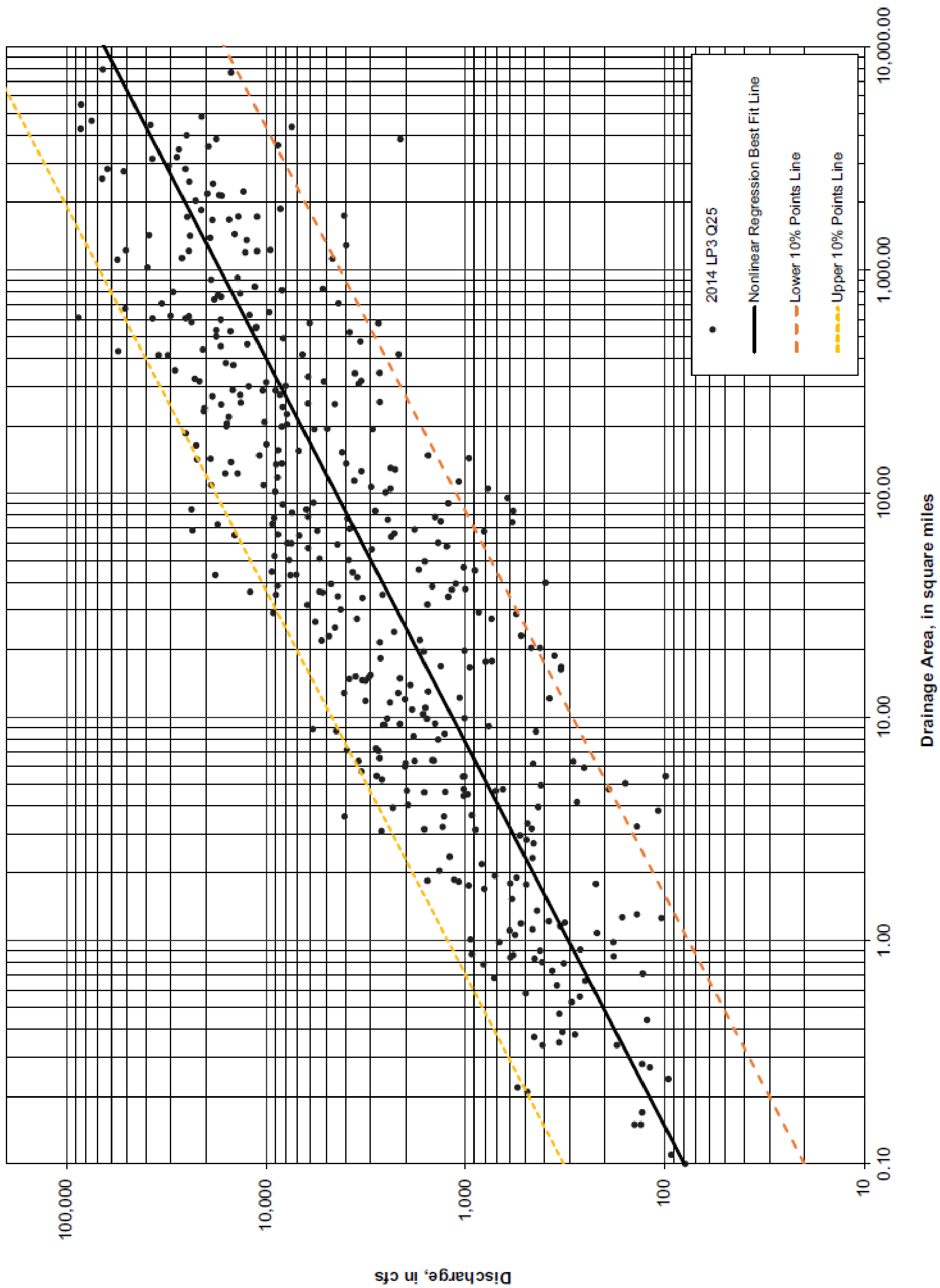

Figure 6-12. 25-Year Peak Discharge by LP3 Analysis

(Flood Control District of Maricopa County, 2023)

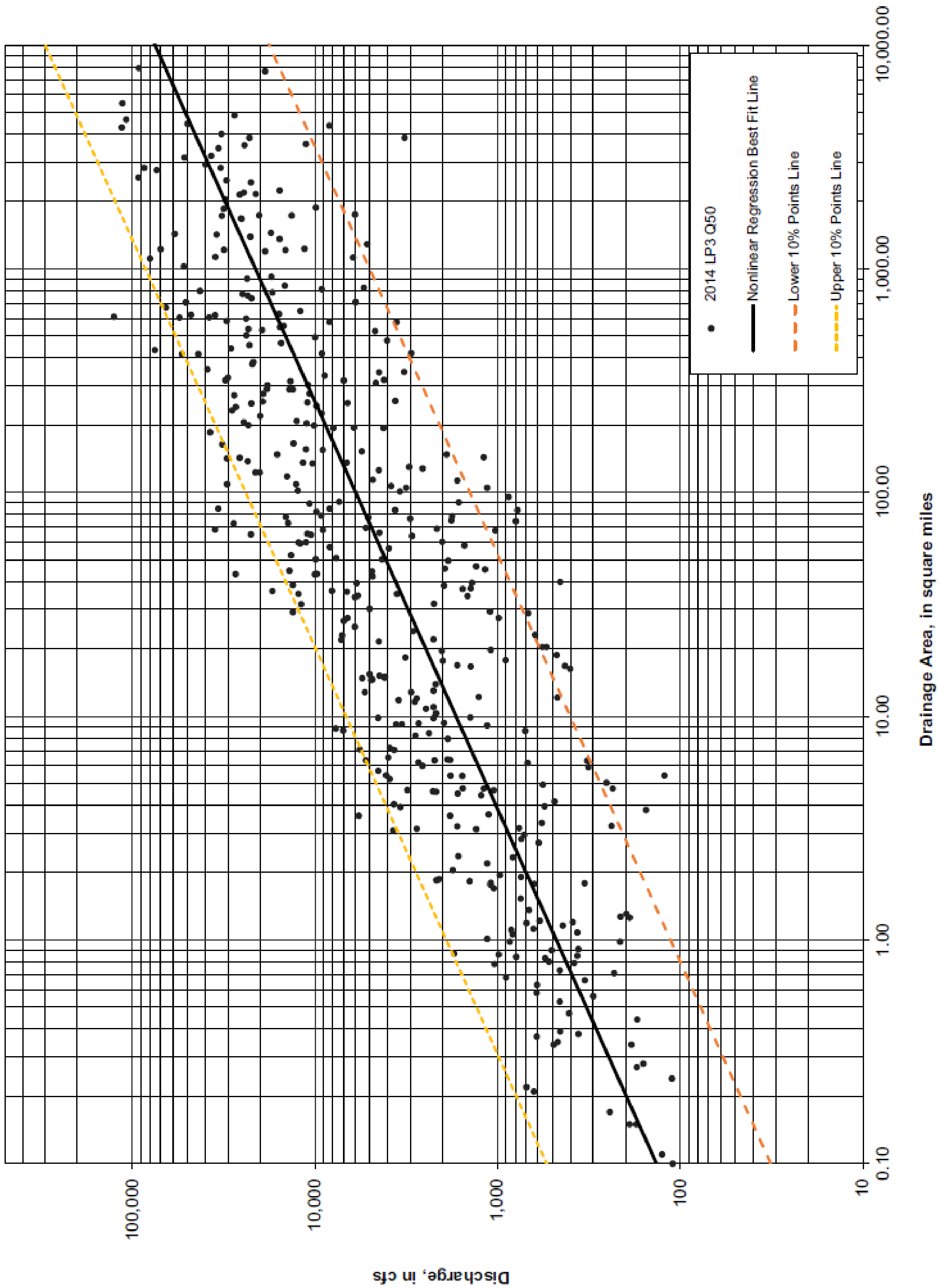

Figure 6-13. 50-Year Peak Discharge by LP3 Analysis

(Flood Control District of Maricopa County, 2023)

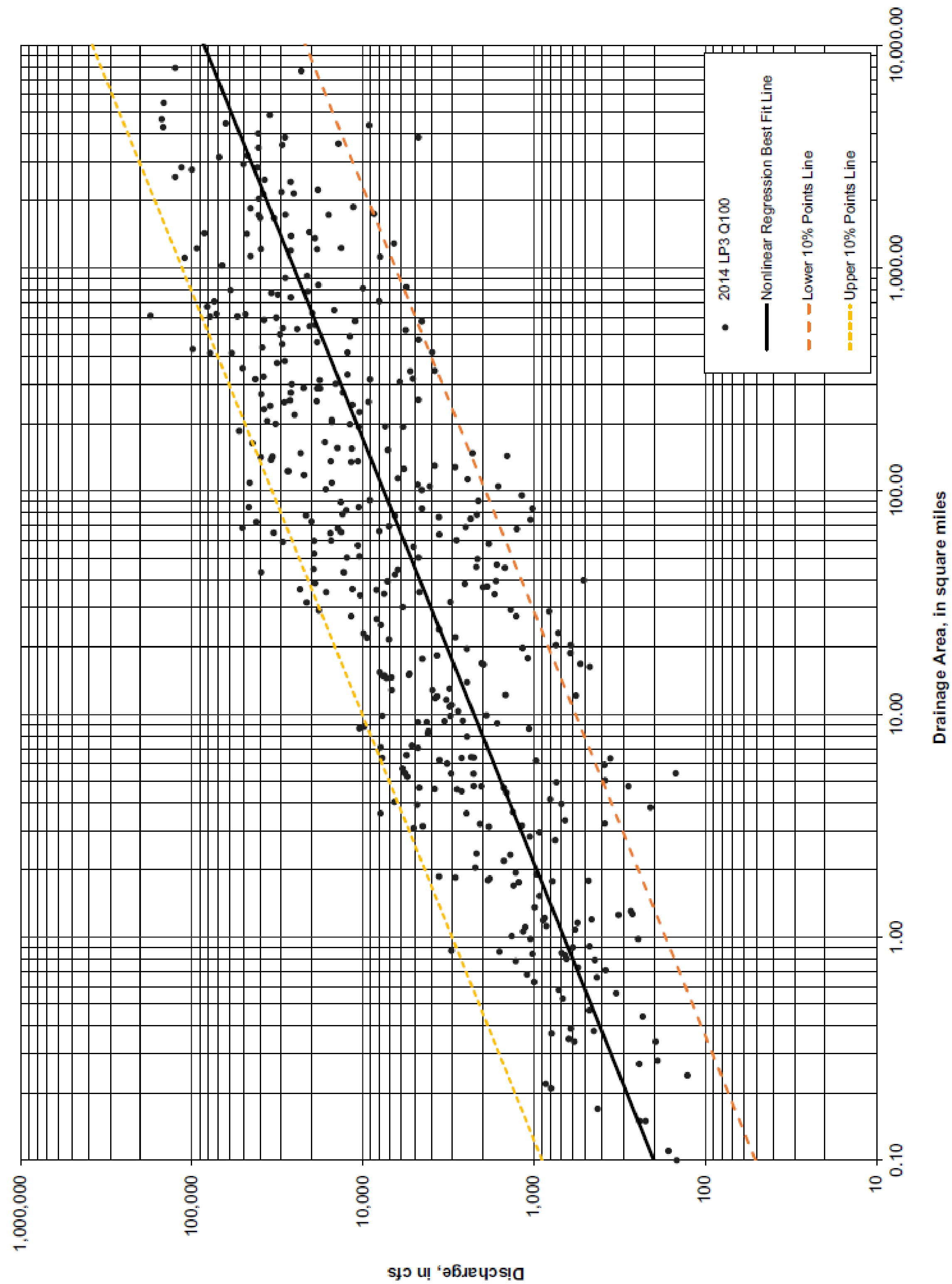

Figure 6-14. 100-Year Peak Discharge by LP3 Analysis

(Flood Control District of Maricopa County, 2023)

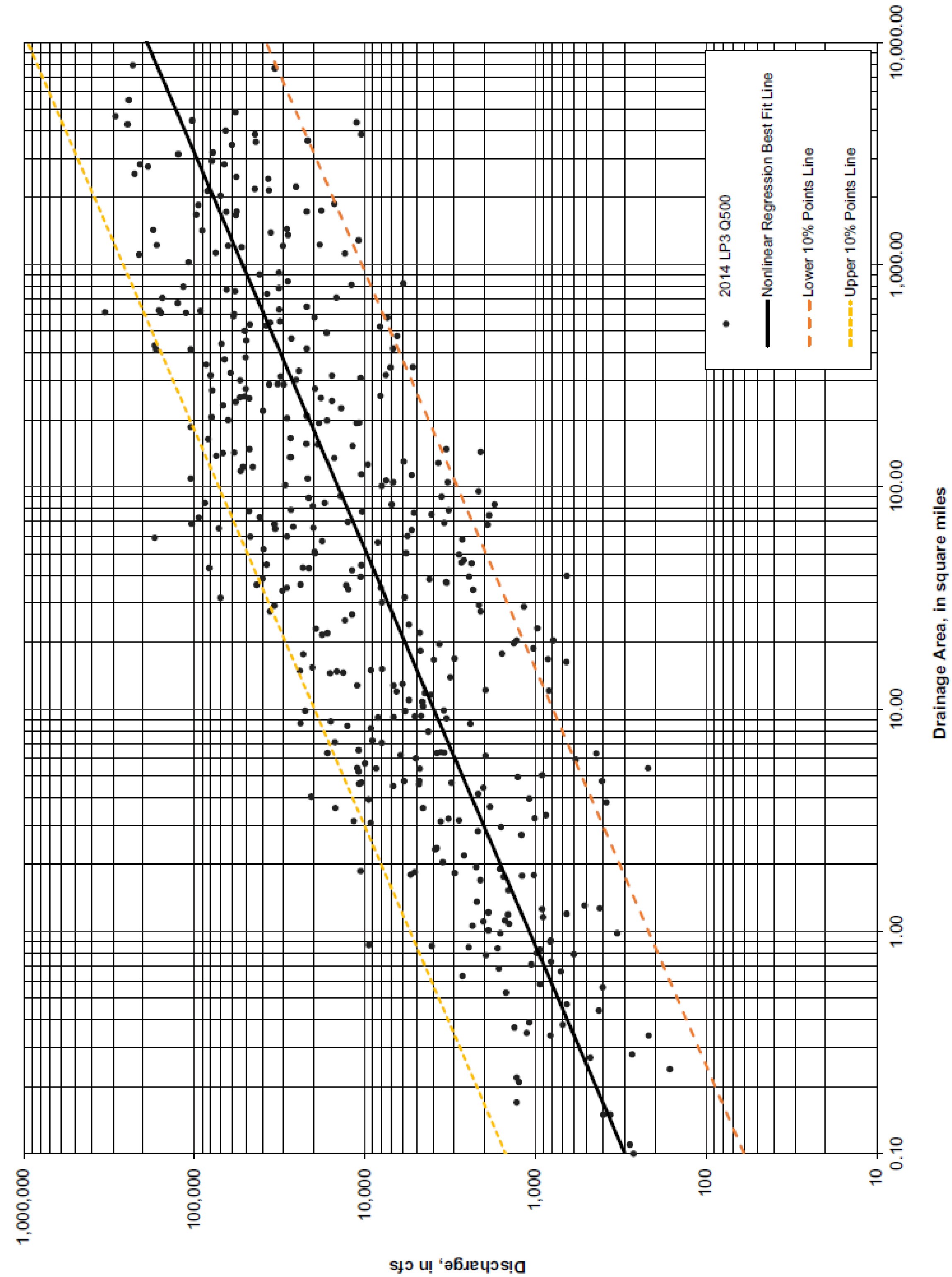

Figure 6-15. 500-Year Peak Discharge by LP3 Analysis

(Flood Control District of Maricopa County, 2023)

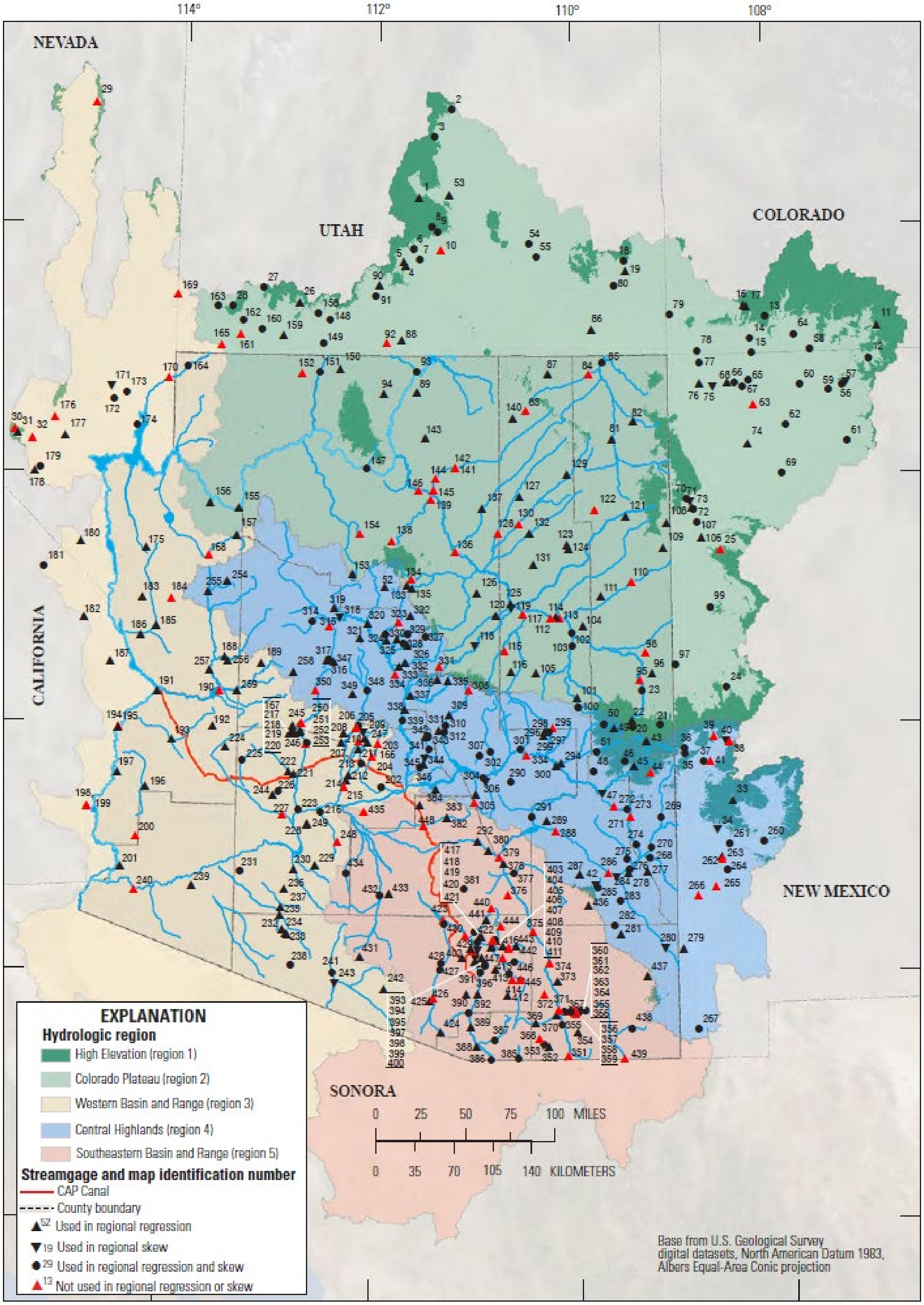

Figure 6-16. Locations of USGS Gaging Stations

(Paretti, N.V., Kennedy, J.R., Turney, L.A., & Veilleux, A.G., 2014)

Indirect Method No. 3 – Regression Equations for the Town

An analysis of streamflow data was performed by the USGS for a study area comprised of Arizona, and parts of California, Nevada, Colorado, Utah, New Mexico, and Sonora, Mexico (Paretti, N.V., Kennedy, J.R., Turney, L.A., & Veilleux, A.G., 2014). That analysis resulted in five sets of regional regression equations for the study area. One region, R4 – Central Highlands, covers the Town limits and shown on Figure 6-17. The regional regression equations can be used to estimate flood magnitude-frequencies for the watersheds that affect the Town. Regression equations are provided to estimate peak discharge for frequencies of 2-, 5-, 10-, 25-, 50-, 100-, 200-, and 500-years. They are functions of drainage area, and some are also functions of the independent variable average annual precipitation. Average annual precipitation for the Town is 12.78 inches (WEST Consultants, Inc., 2024). This value is based on the record from the Prescott Valley – PD Yard Gage for the period of 2002 through 2023. The regression equations are included in Table 6-11 and are recommended only if the independent variable values for the watershed of interest are within the range of data used to derive the regression equation. In general, the equations are applicable to unregulated watersheds with drainage areas between 0.1 and 1,000 square miles although data is plotted for up to 10,000 square miles.

Table 6-11. Flood Magnitude-Frequency Relations for the Central Highlands Region (R4)

(Paretti, N.V., Kennedy, J.R., Turney, L.A., & Veilleux, A.G., 2014)

Annual exceedance probability, in years and P-percent	Equation	Average standard error of model, in percent
2 (50%)	$Q = 54.7(DRNAREA)^{0.664}$	99.2
5 (20%)	$Q = 51.2\,(DRNAREA)^{0.658}(PRECIP)^{0.903}10^{\frac{-0.135*ELEV}{1000}}$	54.7
10 (10%)	$Q = 43.2\,(DRNAREA)^{0.643}(PRECIP)^{1.204}10^{\frac{-0.150*ELEV}{1000}}$	38.2
25 (4%)	$Q = 33.6\,(DRNAREA)^{0.624}(PRECIP)^{1.528}10^{\frac{-0.160*ELEV}{1000}}$	26.7
50 (2%)	$Q = 30.8\,(DRNAREA)^{0.614}(PRECIP)^{1.687}10^{\frac{-0.161*ELEV}{1000}}$	24.6
100 (1%)	$Q = 30.0\,(DRNAREA)^{0.605}(PRECIP)^{1.805}10^{\frac{-0.161*ELEV}{1000}}$	24.4
200 (0.5%)	$Q = 30.6\,(DRNAREA)^{0.598}(PRECIP)^{1.893}10^{\frac{-0.161*ELEV}{1000}}$	25.9
500 (0.2%)	$Q = 33.3\,(DRNAREA)^{0.591}(PRECIP)^{1.976}10^{\frac{-0.160*ELEV}{1000}}$	31.9

where:
Q = the peak discharge, in cfs
DRNAREA = drainage area, in square miles
Precip = man annual precipitation, in inches
Elev = mean basin elevation, in feet divided by 1,000

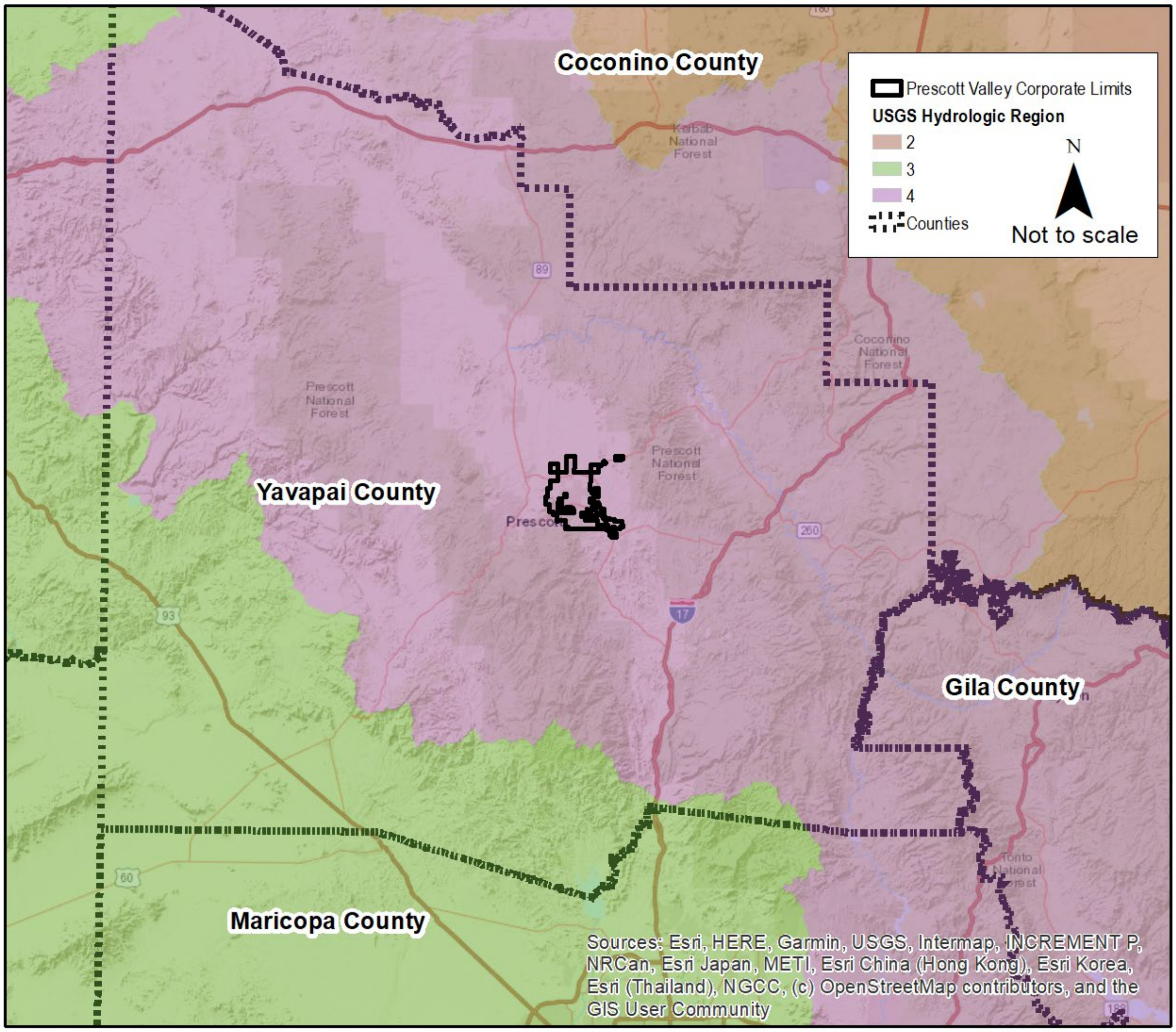

Figure 6-17. USGS Hydrologic Regions

6.3.12 HEC-HMS Criteria

Coefficients for the soil and land use types encountered in the detailed study area of the Prescott Valley Master Drainage Plan are tabulated in the Technical Addendum, Part I, to the Plan.

6.4 Street Drainage

The conveyance of stormwater in a roadway is influenced by the typical roadway cross section, cross-slope, longitudinal slope, and roadway material. Technical guidance for design of roadways and street drainage shall be in accordance with the street drainage chapter of the Maricopa County Hydraulics Manual. Specific design standards for the Town are noted below.

Standard 6.4.1: Building Finished Floor Elevations.

- The finished grading of lots shall be such that all runoff from the lot is directed to the street adjacent to the lot, unless topographic conditions preclude reasonable grading to achieve this objective. Figure 6-18 illustrates the preferable method of grading and draining a lot. Figure 6-19 illustrates an acceptable method of grading and draining a lot when only a portion of the lot can drain to the adjacent street. The grading and drainage illustrated on Figure 6-20 shall only be used when the slope of the lot is such that it is impractical to set the finished floor above the adjacent street. Extreme care must be taken in designing and implementing the grading on a lot using the method shown in Figure 6-20 to direct flow around the structure. The Figure 6-20 method may not be acceptable if the street would carry flow above the top-of-curb or roadside ditch. Driveway cuts must not decrease the capacity of the street to carry water with Figure 6-20 grading.
- For streets with curbs, pad grade shall be 6 inches above the road top-of-curb elevation at the midpoint of the lot, or 12 inches above the Q100, whichever is higher.
- For streets without curbs, the stem wall shall be 13 inches above the natural grade high elevation corner of the structure, or 12 inches above the street centerline at the midpoint of the lot or 12 inches above the Q100, whichever is higher.
- For finished pad grade elevations on lots where runoff is routed between, or direction is changed, or where vertical curb for flow control is used to control the potential for flow across the lot by 'street water', pad grade shall be at least 6 inches above the road top-of-curb elevation at the midpoint of the Lot, or 12 inches above the Q100, whichever is higher.
- The lowest floor of any structure, residential or non-residential constructed in the Town's flood prone area must be a minimum of one (1) foot above the RFE. The RFE is one (1) foot above the BFE which is defined as the 100-year water surface elevation. Structures located within the flood prone areas may be protected from floods up to and including the 100-year flood by placement of fill to elevate the structure above the RFE, provided no adverse impact to adjoining property is caused.
- In all other cases, finished floor elevations shall be a minimum of 1.0 feet above the RFE (Q100).

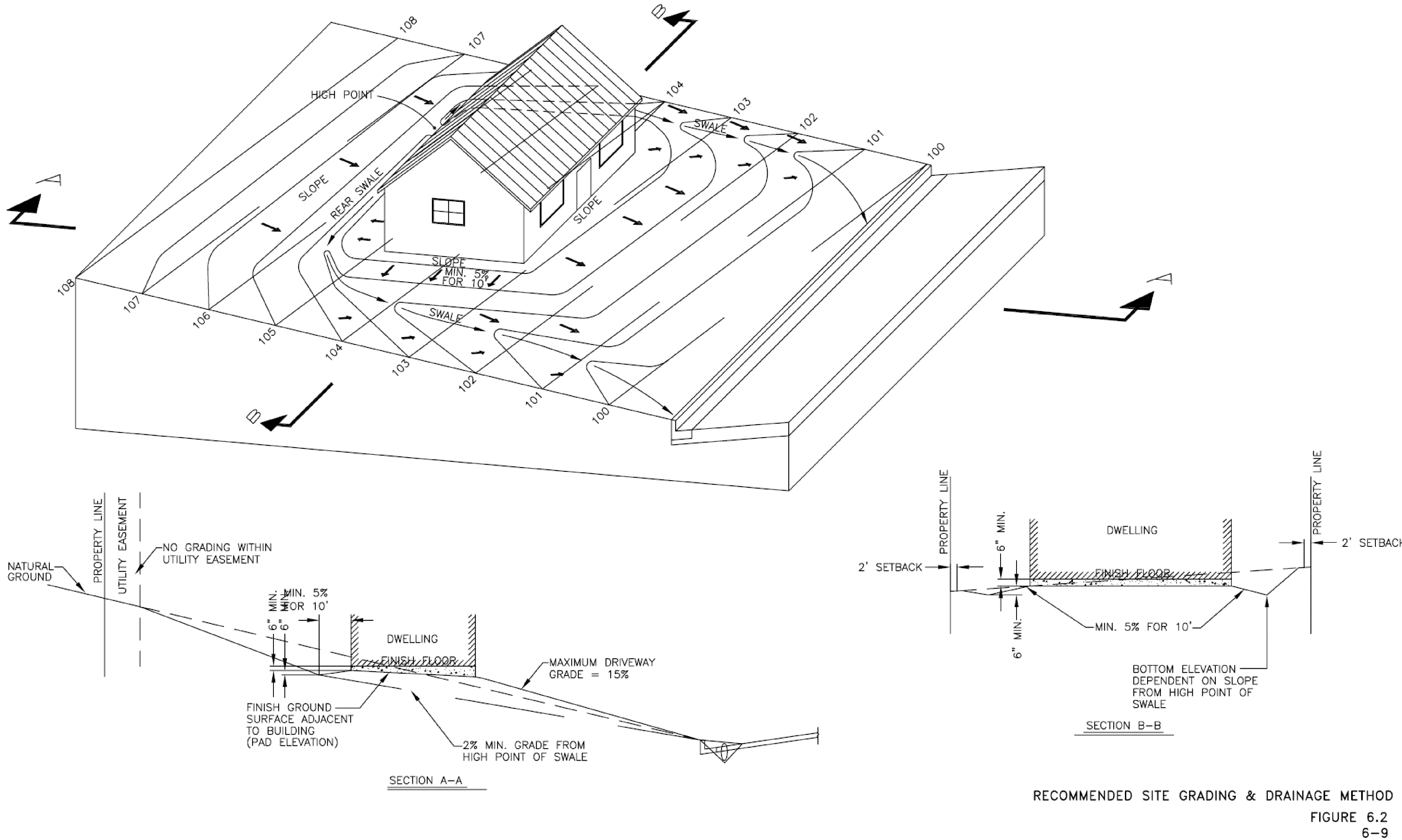

Figure 6-18. Recommended Site Grading and Drainage Method

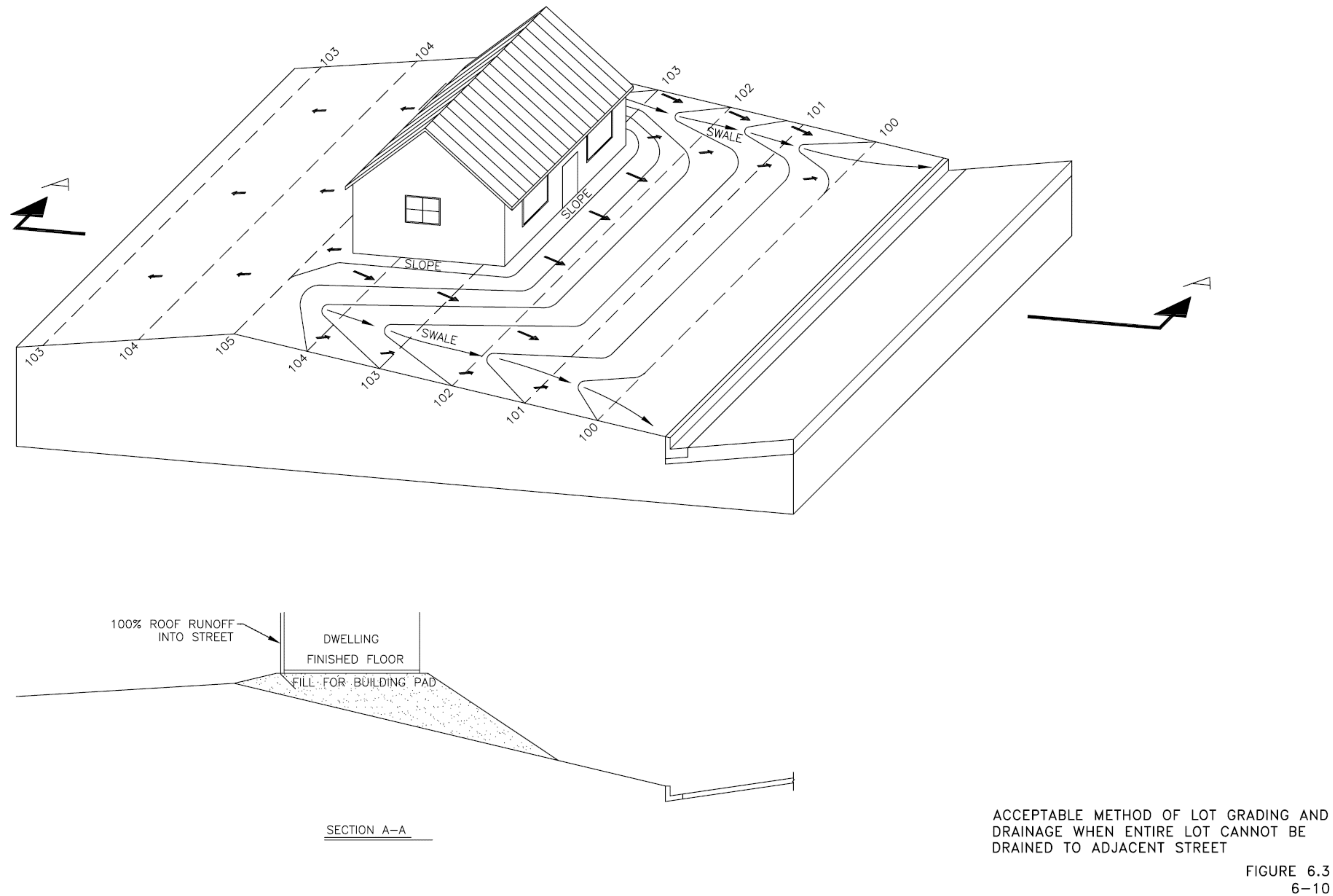

Figure 6-19. Acceptable method of lot grading and drainage when entire lot cannot be drained to adjacent street

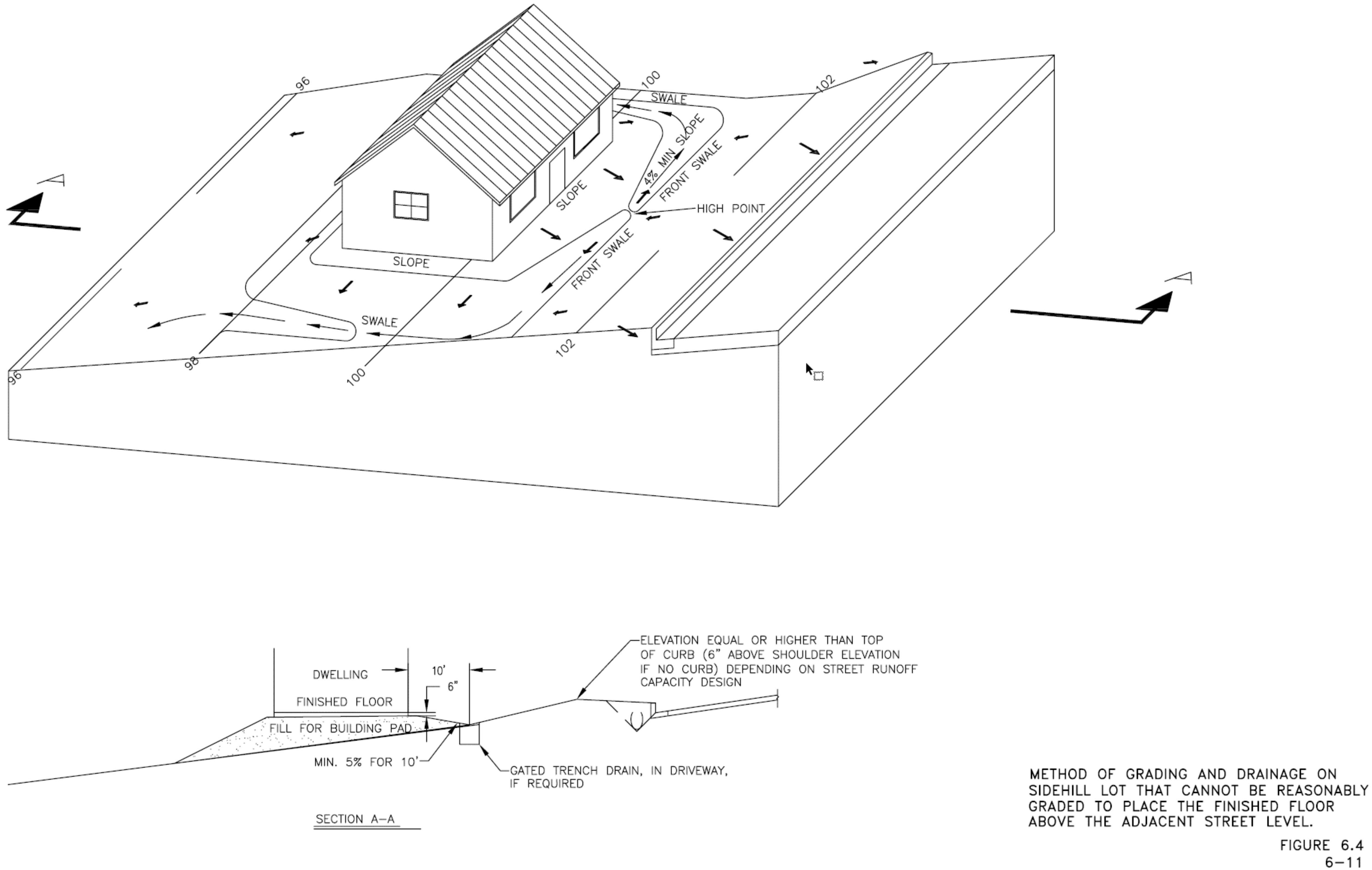

Figure 6-20. Method of grading and drainage on sidehill lot that cannot be reasonably graded to place the finished floor above the adjacent street level

Standard 6.4.2: Sizing Inlets and Laterals. Runoff calculations for the sizing of inlets and lateral connection pipes shall be based on acceptable hydrologic criteria. The Rational Method may be used for sizing inlets and connection pipes even if another method is used for the main drainage system.

Standard 6.4.3: Street Capacity. A Manning's n-value of 0.015 shall be used to calculate the theoretical street flow capacity unless special conditions exist. The allowable width of spread and depth of flow shall be:

1. County Highway/Arterial/Emergency Access:
    a. 10-year: One 12-foot dry lane each direction, and flow depth shall not exceed curb height.
    b. 100-year: Maximum flow depth between curbs $\leq$ 8 inches. Curbs are optional.
    c. Maximum velocity = 8 fps
2. Collector/Distributor/Local Roadways:
    a. 10-year: Flow depth shall not exceed curb height.
    b. 100-year: Maximum flow depth between curbs $\leq$ 8 inches.
    c. Maximum velocity = 8 fps

For all roadways without curb and gutter the 100-year peak discharge shall be contained in a channel with the maximum design storm flow depth not to exceed the adjacent roadway bottom of subgrade elevation. The calculated capacity of a street that has cross-fall, as at the approach to an intersection, shall be recognized as shown on Figure 6-21.

The term Emergency Access refers to roadways designated by the Town 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.

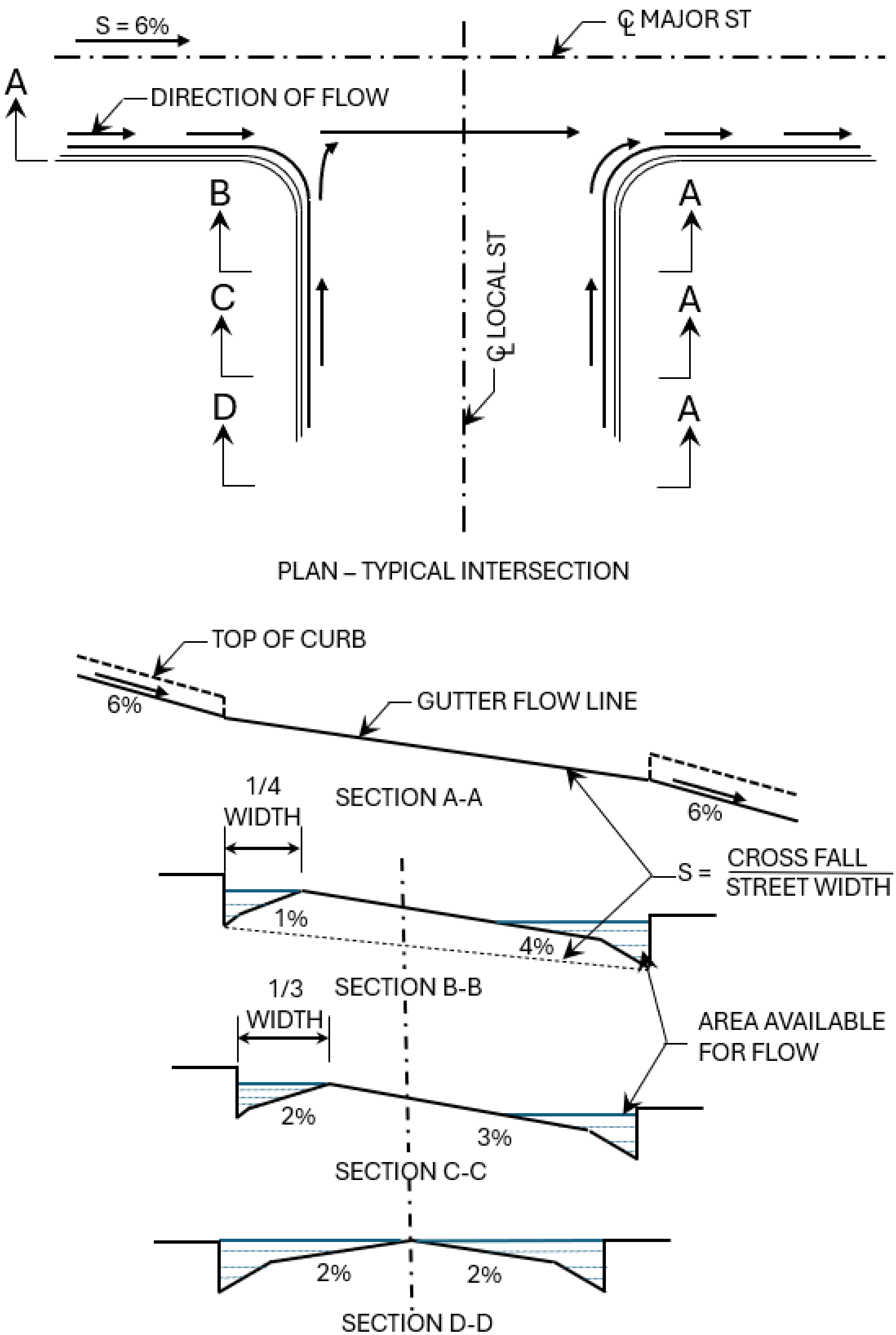

Figure 6-21. Typical Intersection Construction at Junction of Local and Major Street

Standard 6.4.4: Valley Gutters. Valley gutters will only be allowed on local roadways, unless approved by the Town (see Quad City Standard Details 240Q-1 and 240Q-2).

Standard 6.4.5: Curb Return Slope. Curb returns shall have a minimum slope of 0.01 feet of fall for every 1.0 feet 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.

Standard 6.4.6: Maximum Sump Depth. The ponding depth at sumps shall not exceed the limits given in Standard 6.4.3: Street Capacity.

Standard 6.4.7: Maximum Catch Basin Spacing. For arterial and collector streets, the maximum distance that drainage may be carried in the street is based the maximum depths as listed in Standard 6.4.3: Street Capacity.

Standard 6.4.8: On-Grade Catch Basins. Catch basins constructed on a continuous grade are not required to intercept 100% of the design flow. An interception of 100% of the design flow may be required at roadway intersections.

Standard 6.4.9: Catch Basin Design Criteria. The curb opening for a catch basin shall not be greater than 6-inches nor less than 4-inches in height. Grated catch basins are not allowed on collector or arterial streets where bike lanes may occur, unless approved by the Town. If a rated 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, as identified in Table 6-12, Catch Basins Clogging Factors, shall be applied to the specified variable to obtain the interception capacity used for design or to the theoretical capacity of a standard grate.

Table 6-12. Catch Basins Clogging Factors

Condition	Inlet Type	Clogging Factor	Reduction Factor
Sump	Curb Opening	1.25L[1]	0.80
Sump	Grate	2.0P [2]	0.50
Sump	Combination	1.25L and 2.0P [3]	0.65
Continuous Grade	Curb Opening	1.25L_t [4]	0.80
Continuous Grade	Longitudinal Bar Grate with recessed transverse bars with transverse bars Reticuline Grate	0.75R_f and 1.25L [5] 0.60R_f and 1.5L 0.40R_f and 2.0L 0.35 R_f and 2.25L	0.75 0.60 0.40 0.35
	Longitudinal Bar Grate with recessed transverse bars	0.60R_f and 1.5L [5]	0.60
Continuous Grade	Combination	Apply factor 1.25L_t to curb opening only [6]	Apply factors separately to grate &curb opening
Shallow Sheet Flow 1	Slotted Drains	1.25L [7]	0.80

Condition	Inlet Type	Clogging Factor	Reduction Factor
[1] Applied to total length, L, per Example 5 in Chapter 3 of Maricopa County Hydraulics Manual. [2] Applied to total grate perimeter, P, per Example 6 in Chapter 3 of Maricopa County Hydraulics Manual. [3] Apply clogging factors to both curb opening and grate. [4] Applied to L_t per Example 2 in Chapter 3 of Maricopa County Hydraulics Manual. [5] Applied to R_f and L per Example 3 in Chapter 3 of Maricopa County Hydraulics Manual. [6] Applied to L_t per Example 4 in Chapter 3 of Maricopa County Hydraulics Manual. [7] Applied to total length of slotted drain. Slotted drains are most effective for shallow sheet low conditions. With greater deaths and flows, a different type of inlet should be used.			

6.5 Storm Drains

Technical guidance for design of storm drains shall be in accordance with the storm drains chapter of the Maricopa County Hydraulics Manual. Specific design standards for the Town needed for application of the technical information contained within the Maricopa County Hydraulics Manual are noted below and in Table 6-13.

Table 6-13. Storm Drain Hydraulic Design Standards

Design Variable	Design Standard
Minimum Velocity	5 fps for Qdesign The lesser of 3 fps for 0.5 x Q design Or Flow depth = 1-foot
Maximum Velocity	15 fps
Minimum Pipe Size Main Line Lateral Connector (short C.B. to lateral)	 18-inches 18-inches 15-inches
Pipe Diameter Changes	The elevation of pipe crowns, not inverts, are to be matched at manholes and structures.
Maximum distance to first catch basin	The first catch basin should be placed where the roadway drainage hydraulic criteria from Section 6.4, Standard 6.4.3 is first exceeded.
Manhole Spacing (SD = Storm Drain Diameter)	≤ 30 inches SD (straight) = 330 feet max < 30 inches SD (numerous bends) = 200 feet max 33-45 inches SD = 440 feet max > 48 inches SD = 660 feet max
Maximum Hydraulic Grade Line Elevation, Qdesign	Shall not be higher than 12 inches below inlet gutter flowline elevation
Maximum Energy Grade Line Elevation, Qdesign	Shall not exceed gutter flowline elevation
Manning's n-values [1] Reinforced Concrete Pipe (RCP) Corrugated Metal Pipe (CMP)-Concrete Lined CMP, annular corrugations	 0.013 0.013 0.024

Design Variable	Design Standard
CMP, helical corrugations	0.018
CMP, spiral rib	0.013
High Density Polyethylene Pipe (HDPE), smooth interior	0.013
Cast In-Place-Pipe (CIPP). Increase minimum size required for hydraulics by 6-inches	
[1] With appropriate documentation, other n-values may be used.	

Standard 6.5.1: Flow Velocity. Storm drains with flow velocities less than 3 fps for 0.5 x Q design or in excess of 15 fps shall require Town approval. The flow to be carried in a storm drain is that quantity necessary to maintain flows on the street surface within the prescribed depth and velocity limits as described in Standard 6.4.3.

Standard 6.5.2: Storm Drain Profiles. Storm drain pipes and manholes will be shown in profile along with existing and proposed grades. Catch basin and connector pipe profiles shall be provided in the design drawings. The pipe size and slope to four significant figures shall be shown. All existing and proposed utilities crossing over and under the proposed storm drain shall be shown at their proper elevation. Clearance with water and sewer facilities requires a minimum of six feet horizontally and one foot vertically. Clearance with other utilities shall be a minimum of one foot (horizontal & vertical) unless approved by the Town. The information provided in profile format shall include: pipe stationing, pipe size, pipe discharge (Q), pipe velocity, pipe material, hydraulic grade line, energy grade line, and finish grade over pipe. See Section 6.15 for construction plan requirements.

Standard 6.5.3: Hydraulic and Energy Grade Lines. Storm drain systems shall be designed so that the hydraulic grade line is at least 12 inches below the inlet 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 lateral storm drain pipes shall be submitted by the design engineer.

Standard 6.5.4: Tabular Information Requirement. The information provided in tabular format shall include: pipe stationing, pipe size, pipe discharge (Q), pipe velocity, pipe material, hydraulic grade line, energy grade line, finish grade over pipe, gutter flowline, and inlet elevations.

Standard 6.5.5: Pipe Structural & Minimum Cover Requirements. The selection of pipe wall thickness, reinforcing and other structural design shall be based on the manufacturer's recommendations recognizing the depth of cover, soil conditions, bedding conditions and any other relevant factors. Additional consideration for Town plans to lower the elevations of many existing roads, typically 1 foot and possibly up to 2 feet during future road improvement projects must be factored into the final design.

Minimum cover requirements shall be per the manufacturer's specifications.

Standard 6.5.6: Soil Boring Requirements. Soil boring logs shall be provided with the design documentation for all storm drains within a proposed right-of-way. Storm drains in excess of 660 feet in length shall have multiple borings at intervals not to exceed 660 feet. The boring depth shall be a minimum of 5 feet below pipe invert. If cemented or rock material is encountered during drilling which results in refusal, then a rock core shall be taken to identify the type and extent of refusal to 2 feet below proposed pipe invert. Borings will be located in plan and tied to the same datum as the proposed project. Resistivity and pH testing of the soils shall be required to support pipe design in terms of! 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 area of low resistance soil is fully defined. Boring log data shall include the following information:

- The name of the company that produced the soil report.
- The date the test boring was made.
- The type of equipment used to drill the hole and take the samples.
- The size of the auger used.
- A description of caving that occurred during the excavation, if any.
- Horizons of each type of soil encountered.
- Description of the soil.
- Classifications by the Unified Soil Classification System.
- Plasticity index.
- Percent passing No. 200 sieve.
- Water encountered.
- Pavement structure (asphalt concrete thickness, sub-base thickness, if applicable).
- Relative moisture content (specify depth taken).
- Representative unit weight of native material (specify depth taken).
- Laboratory calculated optimum moisture content.
- Resistivity and pH readings.

Standard 6.5.7: Storm Drain Junctions.

- Junctions for new construction storm drains shall be prefabricated 'T's or manholes. Connection to an existing storm drain shall be per an approved detail.
- Manholes are required at all storm drain pipe size changes and junctions.
- Opposing laterals greater than 24 inches in diameter shall be joined by a junction structure designed to HS-20 loading by a professional engineer. The junction structure will be designed to be hydraulically efficient.

- All Tee of Fitting junctions must be directly (straight) accessible from a manhole or catch basin in at least 2 directions of no more than 200 feet.

Standard 6.5.8: Storm Drain Connections.

- Opposing connector pipes shall be offset a minimum of 5 feet horizontally as measured from the centerline of each pipe to minimize headless. When this offset is not practical, the headless created by opposed connectors shall be recognized.
- Prefabricated pipe fittings are to be used on all connections to the main storm drains where a new main is being installed and the connection is not at a manhole location.
- On projects where the storm drain main is existing, connections are to be made with MAG Detail 524, if the outside diameter of the connecting pipe is less than half the inside diameter of the main.

Standard 6.5.9: Horizontal and Vertical Deflections.

- Pipe deflection is allowed only if within manufacturer's tolerances. Specifications for horizontal deflection are to be noted on the construction plans, citing manufacturer's requirements.
- Manholes are required at all horizontal angle points where the deflection angle exceeds the manufacturer's tolerances. Manholes are required at all vertical grade breaks of a storm drain greater than 10%.
- Manholes at vertical or horizontal deflection points shall be at or as close as practical to the point of deflection, with allowance for manufactured bends. If the manhole is not at the point of deflection, it shall be located immediately upstream from the deflection.

Standard 6.5.10: Right-of-Way Width Requirement. A town-owned property, dedicated right-of-way, drainage easement or privately owned drainage tract shall be provided for underground storm drains if not under a designated road right-of-way. Minimum width of the property, right-of-way or easement shall be as follows:

Pipe Diameter (inches)	Width (feet)
≤ 48	15
> 48	Pipe diameter plus 16’

Standard 6.5.11: Pipe Material Requirements.

- The minimum gauge for CMP storm drain pipe is 14 gauge. Soil borings and resistivity tests shall be supplied to support the design of the wall thickness and lining material to obtain design life requirements of 75 years to first perforation.
- Minimum thickness, in inches, for CIPP is equal to the inside diameter of the pipe (in feet) plus 1-inch with the minimum thickness being 2.5 inches.
- Alternate pipe material information is to be presented in table format.

- If required, soil boring and resistivity information is to be provided and shown in plan view. This information is required for all CIPP. If the soil does not meet geotechnical requirements, CIPP will not be considered as an alternate material. CIPP are not allowed within the curb returns of an intersection involving an arterial street.

- The diameter of CIPP main storm drain will be 6 inches greater than that of pre-cast pipe, with the pre-cast pipe diameter sized for the design discharge. The minimum diameter for CIPP shall be 30 inches.

- HDPE may be used for storm drains 48" or smaller provided Type S pipe is specified with water tight joints meeting or exceeding 10.8 psi water test criteria.

- 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* (Corrugated Steel Pipe Institute, 2007), and the American Concrete Pipe Association (ACPA) *Concrete Pipe Design Manual* (American Concrete Pipe Association, 2011).

6.6 Culverts and Bridges

Technical guidance for the design of culverts and bridges shall be in accordance with the culverts and bridges chapter of the Maricopa County Hydraulics Manual. Specific design standards for the Town needed for application of the technical information contained within the Maricopa County Hydraulics Manual are noted below.

Standard 6.6.1: Requirement to Provide Culverts or Bridges. Except where low water crossings are allowed as specified in Standard 6.6.6, watercourses found to meet the following conditions are to be culverted or bridged:

- A Watercourse with a 100-year peak discharge of 25 cfs or greater,

- A watercourse that is a regulatory area designated as WOTUS under Section 404 of the CWA (refer to Section 4.5), or

- As necessary in order to preserve natural flow patterns and prevent adverse impacts on adjacent, upstream and downstream properties.

- As required for wildlife or cattle crossings.

Standard 6.6.2: Technical Design Criteria. Technical guidance for design of roadway culverts and bridges shall be in accordance with the culverts and bridges chapter of the Maricopa County Hydraulics Manual. Additional guidance may be obtained from *Hydraulic Design of Highway Culverts* (Federal Highway Administration, 2012). Specific design policies and standards for the Town needed for application of the technical information contained in the Maricopa County 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 practical.
- 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.

Standard 6.6.3: Pipe Selection Requirements. Pipe culverts are to be corrugated steel pipe (CSP), rubber gasket reinforced concrete pipe (RGRCP), and reinforced concrete box culverts. 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 the Town. Concrete pipes and box culverts may be either precast pipe or CIPP wall thickness, corrugations, and linings are to be determined based on engineering analysis using the design guidelines provided by the CSPI and the AISI, *Handbook of Steel Highway and Drainage Construction Products* (Corrugated Steel Pipe Institute, 2007). Refer to Section 6.5 for resistivity and minimum pipe thickness or design class requirements for storm drain, which also apply to culverts.

Standard 6.6.4: Design Storms. The flow to be carried in a culvert is that quantity necessary to maintain flows on the street surface within the prescribed depth and velocity limits as described in Standard 6.4.3. They are to be designed to convey, as a minimum, the storm frequency peak discharge listed below by street classification with no flow crossing over the roadway and the ponded water surface elevation shall not exceed the lowest adjacent roadway subgrade elevation unless an alternative design is approved by the Town.

- Arterial and All-Weather Access Streets: 50-year storm frequency
- Collector Streets: 25-year storm frequency
- Local Streets: 10-year storm frequency

Standard 6.6.5: Ponding Outside of Right-of-Way. Backwater ponding limits that extend outside of the roadway right-of-way for the 100-year event 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.

Standard 6.6.6: Low Water Crossings. Low water crossings and dip sections are not allowed without approval by the Town.

Standard 6.6.7: Cover Requirements. Minimum cover of fill over culverts must be per the pipe manufacturers specifications.

Standard 6.6.8: Headwall Requirements. Headwalls are required at the inlet and outlet of all culvert installations unless otherwise approved by the Town. Pipe sizes of 30-inch or greater shall have concrete headwalls. Pipe sizes less than 30-inch shall have concrete headwalls if trash racks are required to comply with requirements specified in Table 6-2. Otherwise, pipe sizes less than 30-inch shall have flared end sections or concrete/masonry headwalls.

Standard 6.6.9: Aesthetic Treatment at Culverts. Where culverts convey flow in undisturbed or naturalistic drainage ways, the culvert headwalls shall have aesthetic treatment to match the surroundings. The area surrounding the headwall (exclusive of the watercourse bottom) shall be revegetated and landscaped using boulders and native stone where indigenous. Railings shall be designed compatible with the colors and form of the development or village theme.

Standard 6.6.10: Minimum Culvert Diameter. The minimum diameter for a culvert is 18 inches. Orifice plates may be added for the outlets of stormwater storage basins. The minimum diameter allowed for CIPP is 30 inches.

Standard 6.6.11: Culvert Requirements for Trail Access. Where a public or private multi-use trail easement/right- of-way is located in a watercourse corridor and intended to go through the culvert, the minimum box culvert width shall be 14 feet and the height 12 feet.

Standard 6.6.12: 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 Town approved easement and shall have a minimum drivable width of twelve feet.

Standard 6.6.13: Velocity Requirements.

- Design velocity shall be a minimum of 5 fps and a maximum of 15 fps.
- Culverts are to be designed with consideration to the guidelines presented in the Culverts and Bridges, and Sedimentation chapters in the Maricopa County Hydraulics Manual.
- The culvert shall be designed so minimum velocities facilitate sediment transport to keep the culvert clean, as specified in Table 6-13. The downstream channel must allow the culvert to flow at appropriate velocity.
- The maximum velocity in the culvert should be consistent with channel stability requirements at the culvert outlet. Aggradation or degradation at culvert crossings must be examined in the design of culverts.

Standard 6.6.14: Outlet Protection Requirements. Culvert outlet requirements shall conform to the requirements set forth in Table 6-14. The size, depth, and lateral extent of outlet protection, including energy dissipaters, shall be designed in conformance with the Culvert and Bridges, and the Hydraulic Structures chapters of the Maricopa County Hydraulics Manual.

Table 6-14. Design Criteria for Culvert Outlets

Outlet Protection	Natural Channel	Artificial Channel
None	Up to 1.3 times existing channel velocity	Up to maximum allowable velocity for channel lining
Riprap or other suitable transition apron	1.3 to 2.5 times existing channel velocity	1.0 to 2.5 times allowable channel lining velocity
Energy Dissipater	Velocities greater than 2.5 times existing channel velocity	Velocities greater than 2.5 times allowable channel lining velocity

Standard 6.7.15: Cut-off Wall Requirements. 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 6-15. 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 the Culvert and Bridges chapter of the Maricopa County Hydraulics Manual.

Table 6-15. Design Criteria for Culvert Cut-off Walls

Pipe Diameter	Minimum Inlet & Outlet Cutoff Wall Depth (feet)
18" to 42"	2.0
42" to 84"	4.0

Standard 6.6.16: Bridge Freeboard Requirements. Bridges should be designed to have a minimum FB of 2 feet below the low chord elevation for the 100-year event. For supercritical flows, see Standard 6.6.21 for additional FB requirements.

Standard 6.6.17: Design Floating Debris Allowance.

Hydraulic Analysis of Bridges: 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 the Culvert and Bridges chapter of the Maricopa County Hydraulics Manual.

Hydraulic Analysis of Box Culverts: 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 the Culvert and Bridges chapter of the Maricopa County Hydraulics Manual.

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 document (American Association of State Highway and Transportation Officials, 2020), unless otherwise directed by the Town.

- 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.
- 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.
- For deep drilled shaft foundations, in the area below the bottom of casing, increase the shaft design diameter by one foot on each side.

Culverts: In high debris/sediment areas, the Town may elect to require the clogging factors in Table 6-16 be applied to the design cross section area of the culvert opening.

Table 6-16. Culvert Clogging Factor in High Debris/Sediment Areas

Culvert Size	Clogging Factor
Equivalent diameter <= 48 inches	Reduce available opening area by 50%
Equivalent diameter > 48 inches	Reduce available opening area by 20%

Trash Racks and Debris Racks: Where trash and/or debris racks are required, culvert design capacity shall be adjusted as defined in the Maricopa County Hydraulics Manual.

Standard 6.6.18: Bridges Requirements to Accommodate Trails. Where a public or private multi-use trail easement/right-of-way is located in a wash corridor, the minimum span between pier(s) shall be 14 feet with a height of 12 feet between low chord and the wash bottom if the trail is to go under the bridge.

Standard 6.6.19: Aesthetic Considerations at Bridges. Where bridges convey flow in undisturbed or naturalistic drainage ways, bridge designs shall incorporate materials, colors, and forms compatible with the theme of the surroundings. Use of native stone is encouraged where it is indigenous at the bridge approaches. Wingwalls shall have aesthetic treatment to match the surroundings. The area surrounding the wingwall (exclusive of the wash bottom) shall be revegetated and landscaped using boulders and native stone where indigenous.

Standard 6.6.20: Bridge Design Erosion Requirements. Bridges crossing undisturbed watercourses with designated erosion setbacks shall span the lateral migration erosion hazard zone which can be estimated by lateral-erosion hazard zone estimation method in the Maricopa County Hydraulics Manual. Alternatively, if structural erosion protection is proposed, the total scour depth should be computed for the erosion protection structures based on Maricopa County Hydraulics Manual to support the design and show that there are no adverse impacts to adjacent properties. If a channel excavation is required to pass the design flow under the bridge, a sediment transport analysis should be required to support that no sedimentation will cause channel capacity loss and violation of required FB. The sediment transport analysis can also quantify the deposition volume for sediment removal by operation and maintenance crew after major flood events. The sediment transport analysis 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 the Town do not result in cumulative adverse impacts within the study reach.

Standard 6.6.21: Supercritical Flow Requirements.

For channels functioning in a supercritical flow regime for the design discharge, there shall be no reduction in cross sectional area at bridges and 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 the Town. 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.

Bridge FB below the low chord elevation shall be the greater of 2 feet or the velocity head ($\frac{V^2}{64.4}$) for channel velocities.

Additional standards pertaining to culverts and bridges are listed in Section 6.2.

6.7 Open Channels

Technical guidance for the design of open channels shall be in accordance with the open channels chapter of the Maricopa County Hydraulics Manual. Specific design standards for the Town needed for application of the technical information contained within the Maricopa County Hydraulics Manual are noted below. The following standards will be employed in all designs of engineered open channels (does not apply to undisturbed drainageways):

Standard 6.7.1: Plan Requirements for Q100>=100 cfs.

- All channel plans must contain a plan and profile as well as adequate cross sections to describe geometry.
- The profile 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 FB as required.

Standard 6.7.2: Plan Requirements for Q100<100 cfs. Engineered channels or ditches designed for flows less than 100 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.

Standard 6.7.3: Floodplain Encroachment Requirements.

- 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.

- In accordance with Chapter 12 Floodplain Management of the Town Municipal Code, a drainage permit shall be required for any development or substantial improvement which may have an adverse effect on existing drainage. Channelization and/or floodplain encroachments may result in adverse impacts on existing drainage and thus are subject to the requirements of the Town Municipal Code.
- Encroachment and/or stabilization on one bank must address 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 Town criteria as a minimum. Although FEMA FB height criteria is the minimum standard, levee design FB 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. The Town strongly discourages the construction of levees for flood control purposes.

Standard 6.7.4: Channel Lining Requirements.

- Use of concrete lined channels shall not be permitted without approval.
- All concrete and shotcrete lined channels must have continuous reinforcement extending both longitudinally and transversely. Shotcrete channels shall be designed to have the same structural integrity as concrete channels.
- All sloping and flat concrete, shotcrete, and soil cement finished surfaces shall have roughened surfaces (e.g., embedded rock, grooves, etc.) to discourage inappropriate recreational use.
- The lining for engineered channel bottoms must be designed for a minimum of 18 kips axle loading assuming one leading per week for the design life of the channel.
- Use of grouted riprap for channel lining is not permitted in any application. Rock embedded into structural concrete linings is acceptable.
- Riprap lined channels shall be designed per Chapter 6.6.3 of the Maricopa County Hydraulics Manual.
- Froude Number Requirements:

    Due to erosion and scour of erodible channels and safety concerns with excessively high velocities, the recommended upper limit of Froude Number (Fr) shall be 2.0.

    The Froude Number for all types of channel linings shall be Fr < 0.86 for subcritical flow regime.

In areas of steeper slopes, where the natural channel is near the critical flow regime, the critical flow regime may be acceptable after the channel is modified and the improved Froude Number is less than 1, the flow depth is less than 2 feet, the sediment transport capacity is maintained, adequate protection from scour is provided, and adjacent structures are elevated above the elevation of a possible hydraulic jump.

For concrete, soil cement, and shotcrete lined channels functioning in supercritical flow regime, the additional range of $1.13<Fr<2.0$ is allowed, provided a sediment analysis is approved that substantiates that sediment loading will not change the flow regime from supercritical to subcritical.

At locations where there are to be planned hydraulic jumps, concrete, soil cement, and shotcrete lined channels may pass through $0.86>Fr<1.13$.

No other linings may be used in channels that fall in the range of 1.13 to 2.0.

A 100-year floodplain delineation based on subcritical conditions will be required if a channel designed to supercritical may change flow regimes unpredictably due to sedimentation issues and flow will exceed the channel banks for the subcritical condition.

- Earthen bottom channels with lined side slopes buried below the depth of expected total scour are allowed with supporting engineering justification including sediment transport analysis, scour analysis, soil boring logs, and long-term watershed yield analysis to support equilibrium longitudinal slopes. Riprap, gabions, turf reinforcement mats, or rock embedded structural concrete may be used to line side slopes.
- Gabions are not allowed on channel bottoms except at grade control, drop structures, or similar hydraulic structures.

Standard 6.7.5: Design Technical Guidelines. Channels shall be designed consistent with the guidelines provided in the Open Channels, Friction Losses in Open Channels, and Sedimentation chapters of the Maricopa County Hydraulics Manual. Material and placement shall be designed per Maricopa Association of Governments' Uniform Standard Specifications for Public Works Construction Specification 220 & 703.

Standard 6.7.6: Maximum channel velocities will be governed by the following tables:

Table 6-17. Maximum Permissible Velocities for Unlined Drainage Channels

(USDOT, FHWA, 1961 AND 1983)

Soils Type of Lining (Earth, No Vegetation)	Maximum Permissible Velocity [1] fps
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 sinuous channels multiply permissible velocity by:	0.95 for slightly sinuous; 0.90 for moderately sinuous; and 0.80 for highly sinuous.

Table 6-18. Maximum Permissible Velocities for Grass-Lined Channels

CHANNELS WITH UNIFORM STAND OF VARIOUS GRASS COVER AND WELL MAINTAINED [1, 2]
(ADAPTED FROM USDOT, FHWA 1961 AND 1983)

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 accented only if an irrigation system is provided. 3. Grass lined channels not recommended for slopes greater than 5%.	

Table 6-19. Criteria for Artificial Channels

Type of Channel Lining [1]	Maximum Side Slope, H:V (%)	Maximum Permissible Velocity [2] fps
Structural Concrete [3]	Vertical	15
Soil Cement	2:1 (50%)	7 [4]
Riprap	2:1 (50%)	9 [5]
Gabion Baskets	[6]	9 [5]
Grass (irrigated & maintained)	4:1 (25%)	See Table 6-18
Earth (non-irrigated)	4:1 (25%)	See Table 6-17
Erosion Protection Mat (reseeded, non-irrigation)	3:1 (33%)	[6]

[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] Shotcrete is allowed provided it is 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 stamped by a PE. Velocities greater than 15 fps are not recommended.
[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 as approved by the Town.

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.

Standard 6.7.7: 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 the following formula:

$$r_{sc} = C\left(\frac{V^2W}{0.5g}\right) \qquad \textbf{Equation 6-25}$$

where:

r_{sc} = minimum radius of channel centerline curvature, in feet

C = coefficient, where C = 1

V = mean channel velocity, in ft/s

W = channel width at elevation of centerline water surface, in feet

g = acceleration due to gravity, in ft/s^2

This equation 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. If the total rise in water surface (superelevation plus surface disturbances) is less than 0.5 feet, the normally determined channel FB from this equation is adequate. If the total rise is greater than or equal to 0.5, that depth shall be added to the FB from Standard 6.7.8. This equation is derived from Section 2-5 of *Hydraulic Design of Flood Control Channels* (U.S. Army Corps of Engineers, 1994).

For curved channels with 0.86<F<1.2 the greater of 3 times the design width and this equation shall be used as the minimum radius of channel centerline curvature. 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 Maricopa County Hydraulics Manual shall be carefully followed and applied.

Standard 6.7.8: Freeboard Requirements.

Required FB is computed according to the following formula :

$$FB = 0.25\left(Y + \frac{V^2}{2g}\right) \qquad \textbf{Equation 6-26}$$

where:

FB= freeboard, in feet

Y= depth of flow, in feet

V= velocity of flow, in ft/s

g= acceleration due to gravity, in ft/s^2

In all instances, the FB required is additive to any increases in water surface due to superelevation or channel curvature.

The minimum FB value for rigid channels shall be 1 foot for subcritical and 2 feet for supercritical flows. The minimum FB 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 Standard 6.7.7 are to be reduced as a part of a more detailed design, the FB 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 *Hydraulic Design of Flood Control Channels* . Using a smaller FB in specific cases requires prior approval by the Town. FB exceeding the minimum standard is strongly recommended.

For sand-bed channels, when the Froude Number is equal to or larger than 0.7, the FB shall be the larger value of $0.027V^2$ or $0.25(y+V^2/(2g))$ where V is the channel velocity and y is the flow depth.

In all FEMA jurisdictional floodplains, the greater of the above equation or FEMA's FB requirement shall prevail for design FBs.

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 Chapter 7, Friction Losses in Open Channels, of the Maricopa County Hydraulics Manual shall be followed. The maintenance plan shall include an agreement, approved by the County/Community/District, for perpetual maintenance of the channel. If this is not feasible, then additional FB shall be required. For this case, standard FB 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.

Standard 6.7.9: Minimum Easement Width Requirement. A town easement, property, dedicated right-of-way, or privately owned drainage tract shall be a minimum of the top width of an appropriately sized open channel plus 2 feet contiguous on both sides. If vehicular maintenance access is not provided within the channel bottom, add 16 feet of width to the top on one side. In no case shall a Town-owned property, right-of-way, or private drainage tract be less than 20 feet wide without approval. Open channels shall be included within a town-owned easement, property, right-of-way, or private drainage tract that must be platted.

Standard 6.7.10: Minimum Roadside Ditch Requirements. Roadside ditches shall be no deeper than 3.5 feet. The side slopes of roadside ditches should be as mild as practical and shall be no steeper than 3:1 on the street side and 1.5:1 on the yard side with a triangular shape.

Standard 6.7.11: Town Maintained Channels. Grass lined engineered channels that are to be maintained by the Town will not be accepted for maintenance by the Town until the grass lining is established to a level satisfactory to the Town.

Standard 6.7.12: Minimum Landscape and Maintenance Guidelines. Landscaping and revegetation must not impede access for maintenance. The vegetation must comply with the design intent of the channel in terms of conveyance and FB. Landscaped channels must be designed using minimum and maximum expected n-values, with minimum FB as specified above.

Standard 6.7.13: Pre versus Post Non-FEMA 100-year Floodplain Delineations. During the course of the Master Planning process, the 100-year runoff will be used to delineate a floodplain for major channels with discharges of more than 500 cfs. The final drainage plan shall show the floodplain delineations for pre-project and post-project conditions. The drainage report shall discuss any disparity in these delineations on adjacent properties.

Standard 6.7.14: FEMA Elevation Certificate. The following note must be added to the Final Site Plan/Final Plat and Setback Exhibit.

A Federal Emergency Management Agency (FEMA) "Elevation Certificate" may be completed for each structure constructed in a Town Flood Prone area prior to an Electrical Clearance for that structure if required, one copy of the "Elevation Certificate" is to be submitted to the General Building Safety Inspector onsite and one copy is to be submitted to the Town of Prescott Valley Floodplain Administrator.

6.8 Hydraulic Structures

Technical guidance for design of hydraulic structures shall be in accordance with the hydraulic structures chapter of the Maricopa County Hydraulics Manual. Specific design standards for the Town needed for application of the technical information contained within the Maricopa County Hydraulics Manual are noted below. The following standards will be utilized in the design of hydraulic structures:

Standard 6.8.1: Trash Rack Clogging Factor Requirement. A minimum clogging factor of 50 percent shall be used in the hydraulic analysis of all trash racks. A more stringent clogging factor may be required when the drainage basin is expected to yield excessive amounts of debris. A clogging factor of 100 percent shall be used in the structural analysis of all trash racks.

Standard 6.8.2: Drop Structure Requirements.

- Hydraulic jump analyses will be conducted for a range of flows from the 2-, 10-, and 100-year peak discharges, since flow characteristics at the drop vary with discharge. These analyses are to be used to support the design of the structure and erosion control measures.
- Drop structures having loose riprap on a sloping face are not permitted due to a high failure rate and excessive maintenance costs.
- Open channels are recommended in lieu of pipes for conveyance of low flows through drop structures. Pipes may plug or frequently overtop, leading to additional maintenance problems. Pipes, if approved for conveying low flows through drop structures, shall be no smaller than 24 inches in diameter.

Standard 6.8.3: Aesthetic Treatment Requirement. Where hydraulic structures are located within or adjacent to undisturbed or naturalistic drainageways, the structures shall have aesthetic treatment to match the surroundings. Trash racks shall have an exterior color to match the surrounding native soil.

Additional standards pertaining to hydraulic structures are listed in Section 6.2.

6.9 Stormwater Storage

Technical guidance for design of stormwater storage facilities shall be in accordance with the stormwater storage chapter of the Maricopa County Hydraulics Manual. Specific design standards for the Town are noted below, whether individual on-site or regional:

Standard 6.9.1: Minimum Design Storm Criteria. All new developments shall make provisions to retain the stormwater runoff from a 100-year, 2-hour duration storm. Retention is currently not required for individual lots that are not part of a larger subdivision development (see Policy 3.10.15). Detention must be based on pre- minus post-development conditions for the on-site area only. If the Rational Method is utilized to size the detention facility the calculation method must take into account rainfall events of a longer duration than the time of concentration of the basin. If a synthetic unit hydrograph method is used, a 24-hour storm duration shall be utilized.

Standard 6.9.2: Sediment Storage Allowance. 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 equal to 100-year flood event sediment yield plus n times the annual sediment yield where n is the maintenance interval in year in addition to the designed stormwater runoff volume required for the stormwater retention basin. The annual sediment yield and the 100-year event sediment yield can be estimated based on the Maricopa County Hydraulics Manual, Chapter 11 Sedimentation.

Standard 6.9.3: Detention Basin Requirements. Detention basins must drain within 36-hours after the runoff event has ended through a low flow outlet facility. Post-development peak discharges may not exceed pre-development peak discharges for the 2-, 10-, and 100-year storm events for the design of detention basins. First flush water quality criteria per Policy 3.5.6 requirements must be met.

The use of retention basins in lieu of a detention basin is not allowed without an approved variance. Possible special cases where retention basins may be considered are as follows:

- 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.
- Riparian vegetation in a downstream watercourse would be adversely affected by application of the retention basin policy.

Standard 6.9.4: Basin Volume Calculations. The required basin volume shall be calculated by the following equation:

$$V = C\left(\frac{P}{12}\right)A$$ **Equation 6-27**

where:

V = calculated volume, in acre-feet

C = runoff coefficient (see Section 6.3.3)

P = 100-year, 2-hour rainfall depth, in inches

A = drainage area, in acres

Standard 6.9.5: Sediment Storage Requirement. Sedimentation basins as required per Section 6.11 are to be located at the upstream (inlet) portions of stormwater storage facilities. All stormwater storage facilities receiving flow from undisturbed watercourses shall incorporate sediment settling basins at the terminus of the undisturbed watercourse(s) as identified in Section 6.11. The sediment settling basins shall be easily accessible by maintenance equipment (such as backhoes) and shall have storage equivalent to the 2-year watershed sediment yield as identified in Section 6.11, in addition to the required stormwater runoff volume. ·

Standard 6.9.6: Storage Basin Design Requirements.

- **Depth**. Stormwater storage basins shall have a maximum water depth of 3 feet for the 100-year, 2-hour design storm event. Deeper water depths for the design event shall require approval and shall address safety issues.

- **Adjacent to Streets or Structures**. The required stormwater storage volume shall be provided on private property and not intrude upon the ultimate road right-of-way or easements. The basin side slope should not begin closer than 10 feet from proposed back of curb or structure. If detention basin is located closer than 10 feet to the existing or proposed back of curb or structure, a 3 foot deep cut off wall or approved waterproof geomembrane will be required to seal the basin.

- **Berms**. Berms shall not be placed closer than 13 feet from the back of the curb or eight-feet from the back of the sidewalk. Berms shall not be higher than 2-1/2 feet above grade on the downhill side. Berms must have a minimum top width of 8-feet. An overflow area (emergency outlet) shall be provided in accordance with Standard 6.9.10. In SFHAs or SFHAs shown on the Flood Management Maps, the high-water level for the 100-year event shall be below the adjoining ground to avoid a levee like structure.

- **Side Slopes**. Side slopes of stormwater storage facilities shall be no steeper than 4:1 unless approval is received for a steeper slope. Stormwater storage basin sides, edges, or top of slopes shall be of irregular geometry, where feasible.

- **Landscaping**. Basins shall incorporate native materials (including native stone and boulders) and be revegetated in a manner consistent with the engineering intent of the facility and conducive to maintenance activities. The landscaping plan must show accommodation for access by maintenance equipment commonly used such as backhoes among other equipment. Landscaping components should not adversely affect the basin hydrologic and hydraulics functions, while integrating local landscape and community desired character and potential water conservation and multiple- use opportunities. Stormwater storage basins are to be privately maintained and located within a designated drainage tract. Decomposed granite is not allowed on slopes steeper than 5:1. A landscaping plan must be approved for the stormwater storage area prior to the issuance of a grading permit.

- **Vertical Retaining Walls**. Where necessary, vertical retaining walls may be utilized for portions of basin sides, subject to public safety requirements (fences, railing, etc.) as needed and described in Section 6.2.

- **Freeboard**. A minimum of 1-foot of FB shall be provided between the crest of the embankment and the water surface elevation required to pass the design storm through the emergency spillway as required by Standard 6.9.10. This may be lessened to 6 inches of FB for developments of less than one acre.

Standard 6.9.7: Within Parking Lots. The maximum depth of ponded water within any parking lot location shall be 0.75 foot with the deeper portions confined to remote areas of parking lots, whenever possible. Planning of areas within a parking lot, which will accept ponding, should be such that pedestrians are inconvenienced as little as possible. No more than 25% of the parking lot area may be used for stormwater storage. The minimum longitudinal slope permitted 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 may be permitted.

Standard 6.9.8: Basin Drain Time Requirement. The design of all stormwater storage facilities shall be such that the stored runoff shall be emptied completely from the facility within 36-hours after the runoff event has ended by controlled bleed-off or discharge pump to an approved facility. The required basin drain time may be extended, with approval, for major storage basins (> 50 acre-feet).

Where bleed-off pipes are used as the primary means of draining a retention-type stormwater storage basin, the outlet pipe shall be designed to drain the 100-year 2-hour (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 demonstrate that the bleed-off discharge does not adversely affect downstream peak discharges or downstream structures. Retention systems using a bleed-off method shall meet the first flush requirements of Policy 3.5.6. The minimum allowable pipe size for primary outlet structures is 18-inches in diameter. The proposed diameter of a basin drain pipe should be rounded up to the nearest standard size made by pipe manufacturers. The bleed-off flow rate will typically be much less than the capacity of an 18-inch diameter pipe. Therefore, a permanently attached, hinged orifice plate shall be used to limit the discharge flow rate in conformance with Figure 9.5 of the Maricopa County Hydraulics Manual. Bleed-off time shall be calculated by the Modified Puls storage routing method. Refer to the Maricopa County Hydraulics Manual for example computations.

Standard 6.9.9: NPDES Requirement. Discharges from stormwater facilities must be in compliance with 40 CFR §122 (NPDES), and AZPDES.

Standard 6.9.10: 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 overflows will follow the downstream predevelopment drainage pattern in a safe manner.

- Emergency spillways must be designed to safely convey the peak discharge from the storm listed in Table 6-20, exclusive of the attenuation effects of the basin.

Table 6-20. 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.	½ 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 24-hour storm, whichever is larger.	

Refer to Section 4.7 for information regarding dams regulated by ADWR.

- Emergency spillways shall be designed to convey the design peak discharge and provide erosion protection in accordance with the Hydraulic Manual.

- Down-gradient properties are to be protected from flow depths and velocities in excess of pre-development conditions.

- A 1-foot minimum FB 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.

Standard 6.9.11: Percolation Test Requirements for Retention Basins. Procedures used to determine the design percolation rate shall be one of the following two methods, listed by order of preference:

Method 1. 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. Drywells 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* (Environmental Protection Agency, 1980). An adaptation of this procedure is outlined in Table 6-21.

Table 6-21. 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 6-22. 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.
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

Step	Description
	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 and more are required based on the retention basin bottom area.
- 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 6-22.

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 6-21, 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 6-22. Minimum Quantity of Soil Log Hole/Percolation Tests Required

Retention Basin Bottom Area, SF	Minimum Number of Test Required
<10,000	2
≥10,000 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 & 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 6-28.

$$CF_{sw} = \frac{1}{\frac{A_b}{(A_b + A_{sw})}} = \frac{1}{\frac{3.1416 * 6^2}{(3.1416 * 6^2) + (3.1416 * 12 * 6)}} = 3.0 \qquad \textbf{Equation 6-28}$$

where:

CF_{sw} = Sidewall correction factor

A_b = Area of the bottom of the test hole, in inches^2

A_{sw} = Area of the test hole sidewalls for a 6-inch water depth, in inches^2

The design factor to be applied shall be selected from Table 6-23 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 6-28. 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 6-23. Percolation Design Factors for Design

Condition	Design factor			
	Method 1 (ASTM) Design Factor	Method 2 (Environmental Protection Agency, 1980)		
		Sidewall Correction Factor	De-rating Factor	Design Factor (3) * (4)
(1)	(2)	(3)	(4)	(5)
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.	2	3	2	6
Groundwater or an impermeable layer is encountered within 4-feet to 10-feet below the bottom of the basin	4	3	4	12

$$P_d = \frac{P}{D_r}$$ **Equation 6-29**

where:

P_d = Design percolation rate, in inches/hr

P = Lowest measure percolation rate, in inches/hr

D_r = Design factor from Table 6-23

Basin drain time is estimated by the following equation:

$$T_d = \frac{V}{A_P \frac{P_d}{12}}$$ **Equation 6-30**

where:

T_d = Retention basin drain time, in hr

A_p = Percolation area (basin bottom), in acres

P_d = Design percolation rate, in inches/hour

V = Retention base design storage volume, in acre-ft

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 the following equation.

$$A_p = \frac{2 * \frac{D}{3} * SS * L}{43{,}560}$$ **Equation 6-31**

where:

A_p = Percolation area (approximate), in acres

D = Design ponding depth, in feet

SS = Basin side slope, in feet/foot

L = Length of retention basin, in feet

Standard 6.9.12: Stormwater Storage Facilities Sited In Town Parks. Stormwater storage facilities sited in conjunction with a park that is to be owned and operated by the Town shall have the following requirements:

- Provide positive drainage out of basin.
- Site to be graded in a manner to provide some high ground and/or non-flooding area, with a minimum acreage to be determined by the Town: 15 acres is preferred for a multi-use basin. The amount of area above the 100-year water level is contingent upon the facilities to be provided, but in no case will less than 20% be accepted.
- Provide for street frontage on at least one side.
- Maximum side slopes: 3:1 (non-turf), 4:1 (turf).
- Ground cover should exist either of all turf or all non-turf material.
- Street nuisance water should not enter park, or should be accommodated with an approved low flow channel that is a minimum of 4 feet wide, flat bottom with 6-inch roll curbs. The maximum and minimum velocity of the low flow channel under normal flow conditions flowing at top of "curb" shall be consistent with the velocity standards in Section 6.7.
- Grading, landscaping, and irrigation to be provided by the developer with Town approval.
- Location of site and street access to be approved by the Town.

- Preserve existing vegetation where possible and appropriate.
- Top 1-foot of soil on bank and basin floor to be free of gravel and rock, suitable for establishing turf.
- Depths greater than 3 feet by Town approval only.

Additional standards pertaining to stormwater storage are listed in Section 6.2.

6.10 Pump Stations

See Policy 3.10.13 regarding the use of pump stations. Technical guidance for design of pump stations shall be in accordance with the pump station chapter of the Maricopa County Hydraulics Manual. Specific design standards for the Town are noted below:

Standard 6.10.1: Water Quality Testing Requirement. Water quality testing and approval from the Town is required prior to the operation of a pump station for every pumping event.

Standard 6.10.2: Pump Capacity. Pump capacity shall be sufficient to empty the facility within 36-hours and shall have a maximum output of 1 cfs, unless otherwise permitted by the Town. The use of pump stations for storm drain facilities is discouraged by the Town.

Standard 6.10.3: Design Requirements.

- Provide a 6-foot wide minimum concrete apron around the inlet opening.
- Provide automatic switch control with vertical float controlled mechanism and installed to the manufacturer's recommendations.
- Pump must be accessible with the basin full.
- Screen pump inlet with 3/4-inch mesh for both vertical and horizontal faces.
- Design/performance redundancy shall be required for all facilities in which failure of the pump station will potentially threaten lives or cause significant property damage.
- Pump stations may not discharge into the street right-of-way. Pumped discharges from stormwater storage facilities to storm drain or channel systems may be done with approval of the Town. Pump stations used to drain stormwater from a stormwater storage facility may not be operated in a manner that would increase runoff above historic levels.

Standard 6.10.4: Clean Water Act. Pump discharges must conform to the requirements of the CWA or other applicable federal, state, and local laws or regulations if discharging into a WOTUS, a tributary to WOTUS, or into a Town or County/Community owned structure.

6.11 Sedimentation

Recognizing that sedimentation and sediment transport is either supply or transport control driven (see the Maricopa County Hydraulics Manual, Chapter 11, Sedimentation) and that stormwater runoff may produce sedimentation or erosion, the following standards shall be applied. See Policy 3.7.1: Riverine Erosion Hazard Zones regarding requirements for sediment analysis.**Standard 6.11.1: Sediment Basin**

Requirement. For all watercourses with less than 50% of the definable watercourse exhibiting engineered improvements, or watercourses with contributing drainage areas less than 75% developed, sedimentation basins/structures shall be designed and constructed as an integral part of stormwater storage and/or conveyance facilities. An exception to this requirement may be granted if the sediment transport analysis of the system demonstrates that sediment is conveyed through the system during frequent events including the 2-year storm event. A definable watercourse is considered to be one that has a 2-foot wide bottom and exhibits hydraulic sorting.

Standard 6.11.2: Sedimentation Basin Design Requirements. Sedimentation basins/structures shall be designed to hold a minimum of 2 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 a geomorphic analysis of the study area. Sediment basins/structures shall be designed with 6:1 side slopes with 16-foot wide access ways on opposing sides. Sediment basins/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 with suitable scour protection per the Maricopa County Hydraulics Manual.

Standard 6.11.3: Culvert and Bridge Design Requirements. For all arterial and major collector streets crossing a distributary flow area or alluvial fan*, the following standards shall apply for the design of culverts or bridges:

- Culverts shall be box culverts, a minimum of 4 feet high (5 feet high is preferred), 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). 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 where the natural grade of the watercourses at the culvert inlets are within 12 inches vertically, an equalizer ditch shall be placed on the upgradient side of the road, between culverts. The bottom width shall be no narrower than 10 feet with side slopes no steeper than 25% (4:1).

6.12 Numerical Models

The purpose of this chapter is to provide general modeling guidelines for the use of one- and two-dimensional (1D, 2D) hydrologic and hydraulic models for floodplain management and drainage design purposes within the Town. Detailed technical guidance is not provided since this depends on the model being applied and the needs and specific topographic characteristics of the project being modeled. Instead, general modeling guidance is provided including which models are preferred, or may be accepted, for use within the Town, when a 1D and/or 2D model should be applied, and for what aspects (hydrology, hydraulics, or both).

6.12.1 1D and 2D Models Accepted for Use

The numerical models meeting the minimum requirement of the NFIP, as approved by FEMA, are also accepted for use for the same purposes within the Town. Lists of approved models can be found on the FEMA website.

A list of preferred numerical models for use within the Town:

- 1D Hydrology – HEC-HMS (most recent version)
- 1D Hydraulics – HEC-RAS (most recent version), HY-8 (most recent version), and EPA-SWMM (most recent version)
- 2D Hydrology – HEC-RAS (most recent version). FLO-2D (most recent version, or as approved by FEMA for an FIS study)
- 2D Hydraulics – HEC-RAS (most recent version). FLO-2D (most recent version, or as approved by FEMA for an FIS study), XP Storm (most recent version, or as approved by FEMA for an FIS study), EPA-SWMM (most recent version, or as approved by FEMA for an FIS study)

Written approval from the Town shall be obtained before using any model other than the preferred for a floodplain delineation or drainage design project within the Town. The engineer/hydrologist shall also obtain written approval before use of any 2D model for such projects. The engineer/hydrologist shall submit technical justification for the use of other than a preferred 1D model, and any 2D model, as a part of the request for approval. The justification shall describe the need and technical basis for the use of an accepted, but not preferred model.

6.12.2 Determining Where to Use 1D or 2D Models

1D models should be applied wherever the basic assumptions for such models can be met. 1D hydrologic models should only be applied to dendritic drainage systems. If diverging flow splits exist, there should be a minimal number and most should be able to be neglected by delineating around them, or modeled using flow diversions based on the results from 1D or 2D hydraulic models. A 1D model is generally not an appropriate choice where:

- There are large areas of sheet flow within the watershed that provide significant attenuation and/or result in significant transmission losses.
- There are distributary flow areas that cannot be readily modeled as flow diversions using hydraulic rating curves.
- Flow is dominantly unconfined.
- Urban and suburban areas where flow patterns can change significantly dependent upon the magnitude of the storm, and where street and drainage system flow patterns are distributary. A 1D model may also not be appropriate when peak discharges are needed at most intersections and/or at mid-block locations and it is not practical to subdivide the watershed to that level of detail.

2D models should be applied in areas where flow is dominantly unconfined, and the following characteristics are present, and the effects need to be accounted for:

- There are large areas of sheet flow that cause significant flow attenuation and transmission losses.
- There are significant obstructions to flow, the local effects of which need to be simulated.

- There are distributary flow areas that cannot be readily modeled as flow diversions using 1D hydraulic rating curves.
- There are channels that can be modeled using 1D methods, but overbank flow is unconfined.
- Unconfined flow hydrology and hydraulics are interdependent and cannot be modeled separately with an acceptable degree of accuracy.

6.12.3 Acceptance of Non-Preferred Models

Any non-preferred, but accepted, hydrologic or hydraulic model that is proposed for use on a project within the Town must have a special capability that is needed for the specific application. The model must produce results favorably comparable to the equivalent preferred model prepared using the Town's methodology as set forth in Section 6.3. See www.FEMA.gov regarding the current standards for the use of 2D models for flood insurance studies used for new or revised floodplain delineations.

6.12.4 2D Integrated Modeling Guidelines

The following are guidelines for various aspects of 2D hydraulic modeling:

Grid Size. No minimum or maximum grid size for a uniform square grid model typically used for 2D hydrology is specified. Instead, the engineer/hydrologist shall evaluate the terrain, project objectives, available topography, and the conveyance system doing the majority of the hydrologic work, and determine an optimal grid size that also considers model run time.

For FLO-2D a maximum grid size of 25-feet is recommended, and the grid size used for the *Area Drainage Master Plan Hydrology and Hydraulic Modeling Technical Support Data Notebook* (WEST Consultants, Inc., 2024) is 15-feet. A grid size of no smaller than 10-feet shall be used. The small time step required for such a small grid may exceed the model technical limits, and model runtimes may be excessively long. See the FLO-2D Input Data Manual, latest edition, for more information.

Rainfall. Point rainfall values from NOAA Atlas 14 shall be spatially-varied and applied directly to the 2D model surface without depth-area reduction. A single hypothetical rainfall distribution shall be used for the entire study area. The hypothetical distribution should be generated using HEC-HMS, with a precision of 4 decimal places. Alternatively, the hypothetical distribution may be generated by hand using a spreadsheet using the approach implemented by HEC-HMS.

Rainfall Losses. Rainfall losses shall be modeled using the Green and Ampt equation and the procedures outlined in Section 6.3. If the 2D model does not have this capability, it shall not be used for hydrologic modeling. If there is insufficient information available to perform model verification, as described below, total infiltration for the model shall be limited to the total infiltration that would be computed from a HEC-HMS model for the study area. This has the conservative effect of eliminating most transmission losses, which are a significant unknown. The rainfall loss volume without transmission losses can be estimated by creating a HEC-HMS model of the study area with actual infiltration loss records and dummy unit hydrograph records. A single basin model can be used if the infiltration characteristics do not vary significantly. If they do, a multiple sub-basin model should be constructed. The depth of total rainfall loss for each sub-basin can then be used to compute an equivalent infiltration depth by dividing by the volumetric soil moisture deficit (DTHETA) parameter. In FLO-2D this infiltrated depth can then be used to set a limiting allowable infiltration depth that is either globally applied for the study area or spatially-varied by sub-basin. Another acceptable approach is to perform a site-specific rainfall loss analysis of the watershed using rainfall simulators and tension infiltrometers to determine more accurate Green and Ampt parameters. This approach shall be implemented using the procedures established by the Desert Research Institute. Consult with the Town before undertaking this level of investigation. The standard Green and Ampt parameters are appropriate for 1D modeling methods but may overestimate infiltration in 2D models when transmission losses are significant.

Land Surface Characterization. A land surface characterization shall be prepared for a 2D model study area. The characterization shall be polygon based in GIS, with each polygon representing a surface feature as listed in Table 6-7.

The level of detail and number of classifications used will depend on the project objectives. Each surface feature should have the following:

- An assigned n-value for 3 feet of flow depth (for the case of FLO-2D; may be different for other 2D models)
- IA
- RTIMP
- Initial soil moisture condition (Initial Saturation), which will be used to assign the DTHETA

Values for various surface feature classifications are included in Table 6-7. The other areas remaining unclassified in a study area are assigned Green and Ampt parameters based on the NRCS Soil Map Unit and Saxton and Rawls (2006) methodology. Obtain Town's written permission to use values other than Table 6-7, including the use of measured hydrologic conductivity.

Surface Roughness. Surface roughness at 3 feet of flow depth should be spatially varied based on the land surface characterization from above. If the model supports it, a depth variable n-value approach should be used.

1D Channels. The modeling of inset 1D channels may be necessary, depending on whether or not the selected grid size can accurately model any significant channels within the study area. If the channels present can be modeled using 1D techniques within the channel banks, that may be the most efficient approach. Use of 1D channels will often allow use of a larger grid size for the overbank areas than is necessary to model the study area without 1D channels. It becomes a balance between model run times for smaller grid models, and the use of inset 1D channels with a larger grid. The level of effort to code any 1D channels may outweigh the benefits.

Obstructions to Flow. A key component of any 2D model is the ability to model flow obstructions at a high level of detail, especially in an urban environment. Building obstructions should be modeled when necessary to produce accurate results and/or meet the project goals. If the model is performing integrated hydrology and hydraulics, the model must be able to simulate runoff from the building as well as act as an obstruction to flow.

Hydraulic Structures. Hydraulic structures such as culverts, bridges and storm drains should be included in the 2D model if they have a significant impact on flow distribution on the surface, and/or if necessary to meet the goals of the project.

Walls and Levees. Masonry and concrete walls and levees can have a significant impact on distribution of flow within the study area. Modeling of these structures should be carefully considered as preparation of input data files can be a significant effort. The 2D model should be run first without such obstructions to determine flow patterns. Then the walls and levees that are hydraulically significant can be determined and the level of effort for data collection and data input file coding can by more accurately estimated. This is often an iterative process. If the model is used for floodplain delineation, walls and levees shall not be included unless they are a FEMA accredited levee system. See Policy 3.9.5 for more information.

Floodplain Cross Sections in FLO-2D/Profile Lines in HEC-RAS. The engineer/hydrologist shall define cross sections/profile line as required to provide hydrographs at key locations such as street crossings and wash confluences, to meet the project goals, and to aid with model verification and/or calibration. Cross sections/profile lines shall be defined perpendicular to the flow direction. Each cross section/profile line shall only cover an area of concentrated flow with a single dominant flow direction. Additional cross sections/profile lines are required for concentrated flow with a different dominant flow direction.

Model Verification/Calibration. Every 2D model shall be verified and calibrated to the extent possible. Historical storm events shall be used whenever possible. The Next-generation radar (NEXRAD) radar data for each storm shall be obtained in 5-minute time intervals, if available, and locally adjusted using any available ground-based rain gage information. The rainfall data should be applied to the model surface as spatially and temporally varied rainfall. The model results should then be compared against any available stream flow gage data available. Any physical evidence of storm flow or ponding depths at specific locations should be collected from roadway maintenance records, field personnel interviews, newspaper records, residents of the area, law enforcement or other government agency personnel who may have been on duty during the event, and television news videos or photographs. This information should then be evaluated, compared with the model results, and the model parameters adjusted if necessary to match the observed results. For undeveloped, unregulated watershed indirect methods can be used (see Section 6.3.11) This effort shall be fully documented. In the event that verification/calibration information is not available, the infiltration depth limit approach shall be taken as described above. For 2D hydraulics only models, the verification/calibration effort should be based on known high water marks and comparison with measured hydrographs and is subject to the accuracy of inflow hydrographs and inflow locations used.

6.13 Hydrology and Hydraulics Reports (non-FIS)

6.13.1 Report Organization

Hydrology and hydraulics reports for purposes other than flood insurance studies shall be organized to include sections as follows (as a minimum):

TABLE OF CONTENTS

1. Completed Hydrology and Hydraulics Report General Checklist

2. Introduction
3. Location
4. Site Description and Proposed Development
5. FEMA Floodplain Classification
6. Off-site Drainage Description
    6.1. Background
    6.2. Proposed Offsite Flow Management
7. On-site Drainage Design Description
8. Hydrology (similar to ADWR SS 1-97)
    8.1. Methodology
    8.2. Parameters
    8.3. Results
    8.4. Confidence Checks and Sensitivity Analysis
9. Hydraulics (similar to ADWR SS 1-97)
    9.1. Methodology
    9.2. Parameters
    9.3. Results
    9.4. Confidence Checks and Sensitivity Analysis
10. Detention Requirements
11. Minimum Finished Floor Elevation Requirements
12. Stormwater Pollution Prevention Plan (SWPPP) Description
13. Sedimentation and Erosion Hazards Discussion
14. Stormwater Permits Requirements (Sections 401 and 404 of the CWA, Floodplain, etc.)
15. Conclusions and Recommendations
16. References

Figures

Figure 1 Area Location Map

Figure 2 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

Appendices

Appendix A Off-site 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 Retention/Detention 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

6.13.2 Hydrology and Hydraulics Report Checklists

Each report will contain the hydrologic and hydraulic analysis checklists as provided by the Town, completed as appropriate for the proposed project.

6.13.3 Additional Report Requirements

Hydrology/Hydraulic reports shall include, but not be limited to, the following items:

- Professional engineer seal signed and dated.
- 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 200-feet. Where drainage areas are large or otherwise inappropriate, other scales may be approved.
- Detailed street hydraulic analysis and storm drain analysis (where required) with labeling of features consistent between the report and the final plans.
- Calculations for the proposed stormwater storage facilities including storage routing, storage volume required, storage volume provided, and first flush requirements. 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 basin dry up within 36-hours after the runoff event has ended is required.

- If adjacent land drains into or is diverted around the development, then 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.
- A lined drawing of the proposed drainage system in plan view showing design flow and capacity. Location and invert elevation at the drainage outfall shall be labeled.
- 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, change the historical flow path, or discharge at erosive velocities above pre-project conditions.
- Average slope and typical cross sections of all streets.
- FEMA floodplains in and adjacent to the project area as an exhibit or figure.
- Summary of previously prepared drainage reports pertinent to the subject area.
- Other requirements as detailed in the Town Standards.

6.14 Hydrology and Hydraulics Reports (FIS)

6.14.1 Report Organization

Hydrology and hydraulics reports documenting floodplain delineation studies for approval by the Town and/or FEMA, shall be prepared in accordance with ADWR State Standard 1-97. The checklists in Tables Table A.2: and Table A.3 shall be used and a completed copy of both provided with the submittal. The Technical Data Notebook (TDN) prepared using ADWR State Standard 1-97 shall be based on the considerations listed in Section 6.14.2.

6.14.2 Technical Data Notebook Additional Requirements

The checklist as provided by the Town shall be used in preparation of the TDN, and a completed copy included with the submittal.

6.15 General Construction Drawing Requirements

1. 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. The seal shall include the date of expiration.
2. All construction shall conform to Maricopa Association of Governments (MAG) and Quad City Standard Details, latest revisions, unless specifically approved by the Town and modified on the plans.

3. Information to determine drainage patterns.
4. Information to determine that an adjacent property drainage pattern will not be adversely affected.
5. A plot of hydraulic and energy grade line profiles for storm drain pipe, both mains and laterals 18 inch or larger shall be provided. The approved hydraulic analysis model, which was used to develop the hydraulic and energy grade line profiles, shall be submitted with the hydrology and hydraulics reports. The profiles shall be submitted in summary form in plan and profile at a reduced scale, intended to highlight the general alignment and hydraulic connectivity of the system herein referred to as the Hydraulic & Energy Grade Line Profile Sheet(s). This information is to be provided with the design data sheet(s) from the hydrology/hydraulics report. The following data shall also be included with the hydraulic/energy grade lines profiles:
    a. The finished street elevation over the storm drain pipe.
    b. Both the existing and proposed pipe profile and size shown with the hydraulic grade lines labeled.
    c. The design peak discharge (cfs) in the storm drain pipe segments.
    d. The velocity (fps) in the storm drain pipe segments.
    e. Invert elevations at pipe entrances and exits.
    f. Appropriate stationing.
6. Plan and profiles of catch basins and connector pipes shall be provided. These profiles shall show gutter elevation, top-of-curb elevation, catch basin type, "V' 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.
7. On the storm drain plan sheets, the engineer shall show the rim and invert elevations and profile of the line at all existing sanitary sewer manholes.
8. The engineer shall identify existing valve nut elevations for all water valves on the project. The valve nut elevations shall be called out in plan view next to the water valve.
9. 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.

10. Clearance with water and sewer facilities requires a minimum of six feet horizontally and one foot vertically. Clearance with other utilities shall be a minimum of one foot (horizontal and vertical) unless approved by the Town.
11. Below ground utilities shall be dimensioned from the road center or monument line.
12. 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, etc. shall be called out including size and pad elevation, and shown in plan, 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'). If no street monument or centerline is involved, the utilities shall be referenced to the centerline of the channel or storm sewer.
13. When below ground appurtenances (utilities, monuments, tanks, valve boxes, etc.) depicted on As-Built or "Record" drawings cannot be field located, they shall be shown and labeled as "not found."
14. 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, trash racks, drop inlets, hand rails, pipe supports, etc.).
15. 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.
16. All storm drain plans shall have the following format:
    a. All plan requirements as defined in the Prescott Valley Design and Construction Standards and Specifications shall be completed with.
    b. Main line storm drain plans shall be 1 inch=20 feet horizontal and 1 inch=4 feet vertical, unless otherwise approved.

c. Scales for connector pipe/catch basin profiles shall be at the same horizontal and vertical scale as the main lines or at a larger scale if necessary to clearly illustrate the improvements and existing utilities, 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 matchlines and at the beginning/end of the storm drain.

f. Centerline stationing shall be shown on the plan and profile sheets. 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. All plans shall use standard Town symbols, if available.

h. Final plan sheets shall be a printed 22-inch x 34-inch, such that a 50% reduction will be 11 x 17 inches.

i. Letter size on full size drawings shall be 14 point minimum.

j. Title blocks shall be located in the lower right-hand corner of the plans and shall include the title "Storm Drain Plans."

k. Storm drain diameters shall be shown in plan and profile with reference to material type.

6.16 Submittals to Development Services

1. All drainage and grading submittals shall be accompanied by a transmittal letter and the Drainage and Grading Plan Checklist provided by the Town, (in addition to the Storm Drain Checklist identified in Section 6.13). The transmittal letter shall include the project name, Development Services project number (if applicable), engineer's name, materials submitted, and purpose of submittal. All plans shall have the CDD project numbers and quarter section number placed in the lower right-hand corner of the plans. An approved preliminary site plan shall be submitted. In addition, a Stormwater Pollution Prevention Plan and Checklist will be required per NPDES criteria, if applicable. All checklists shall be obtained from Development Services.

2. Approved hydrology reports and drainage and grading plans must precede recordation of instruments of dedication when such data is necessary to determine required rights-of-way, Town-owned properties, facilities for drainage, or finish floor elevations. A permit for off-site construction cannot be issued prior to the drainage and grading permit.

3. All submittals must be logged in with the Development Services and shall be accompanied by a letter of transmittal. The project name, Engineer's Name, project numbers, material submitted and purpose of submittal (i.e., preliminary review, review of calculations, revision approval, etc.) shall be given on the letter. Drainage and Grading Plan Review Fees are required at time of Drainage and Grading Permit Approval and Permit Fee Payment. Drainage and Grading Permit Fees are due prior to the issuance of any Drainage and Grading Permit. For drainage and grading plans

submitted as a part of a building permit application, no separate Drainage and Grading plan review or Drainage and Grading permit fee is required.

4. In addition to Section 6.14, plans submitted to Development Services shall include:

- Information to determine drainage outfalls. All drainage outfalls shall be shown on plan and profile, extending until a definite day light condition is established. All temporary outfalls shall be shown in plan and profile, and clearly called out.
- Topographic information showing 2-foot contours (or similar detail) within 100 feet of the project property must be provided.

7 Policy and Standard Revision Process

The Town utilizes a multi-disciplinary multi-division approach to review and adopt proposed changes to drainage policies and standards. This approach utilizes multi-disciplined professionals in order to best reflect the multitude of societal resources influenced by stormwater runoff.

Those seeking changes to policies or standards must make a formal written submittal to the Town stating the present policy/standard, identifying the proposed change(s), and providing comprehensive justification for the change. The Town shall review requested changes.

The Town's policies, regulations, and standards for drainage will be posted as a living document.

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9 Glossary

100-Year Flood: A flood stage or height that, statistically, has one percent chance of being equaled or exceeded in any given year. The 100-year flood is often referred to as the base flood.

Abutments: Walls supporting the end of a bridge or span and sustaining the pressure of the abutting earth. In a drop structure, the walls that form the sides of the crest of the drop. In some structures, wingwalls (transition walls) extend up stream of the abutment walls to create a smooth transition from the upstream channel.

Aggradation: A progressive buildup or raising of the channel bed due to sediment deposition. Permanent or continuous aggradation is an indicator that a change in the stream's discharge and sediment load characteristics is taking place, see Degradation.

Alluvial Fan: A sediment deposit located at a topographic break, such as the base of a mountain, escarpment, or valley side, which is composed of streamflow and / or debris flow sediments and that has the shape of a fan either partially or fully extended.

Antecedent Conditions: Watershed conditions prevailing prior to an event; normally used to characterize basin wetness, e.g., soil moisture. Also referred to as initial conditions.

Apron: A section of concrete or riprap constructed upstream or downstream from a control structure to prevent undercutting of the structure.

Aquifer: A geological formation or structure that stores and/or transmits water, such as to wells and springs.

Areal: Relating to, involving, or of an area, which is a region of land or expanse of space.

Arterial Street: An arterial street system carries large traffic volumes within and through urban areas. The urban arterial system is functionally divided into two classes, major and minor (Town of Prescott Valley, 2022).

Attenuation: The reduction in the peak of a hydrograph resulting in a more broad, flat hydrograph.

Bank Stabilization: A vegetative, structural, or combination of bank treatment designed to stabilize stream and shoreline and to reduce erosion. Methods include bio stabilization and hard armoring.

Base Flood Elevation (BFE): The computed elevation to which floodwater is anticipated to rise during the Base Flood.

Basin Area: The area which contributes stormwater to a concentration point such as a lake, stream, or drainage system. See Watershed.

Basin Floor: The bottom of a stormwater retention facility which has been specifically designed for the purpose of disposing stored runoff following a storm event by the process of infiltration into the subsurface.

Bed Material: Material found on the bed of a stream (may be transported as bed load or in suspension).

Bedrock: The bottom-most layer of the soil horizons. The depth of the bedrock varies. It may be exposed on the surface of the earth or maybe hundreds of meters deep inside the earth. It forms the earth's crust.

Berm: A ridge or barrier constructed of compacted soil, gravel, rocks, and stones to prevent, divert, or direct water away from a particular area.

Bioretention Facilities (e.g., rain garden): Landscaped depressions that treat on-site stormwater discharge from impervious surfaces such as roofs, driveways, sidewalks, parking lots and compacted lawns.

Bleed-Off: A pipe used to discharge stormwater.

Bore Hole/Boring: A method of drilling into the ground to collect soil samples and bedrock to determine the soil's chemical and physical properties.

Braided Wash: A wash whose flow is divided at normal stage by small mid-channel bars or small islands; the individual width of bars and islands is less than about three times the water width; a braided wash has the aspect of a single large channel within which are subordinate channels.

Catch Basin: A chamber or well, usually built at the curb line of a street, for the admission of surface water to a storm sewer or sub-drain.

Channelization: The process of straightening, widening or deepening stream channels to increase water conveyance and provide anthropogenic services (i.e., flood protection, navigation, drainage to facilitate agriculture and development).

Channel Reach: A segment of stream length that is arbitrarily bounded for purposes of study.

Check Dam: A low dam or weir across a channel, for the diversion of irrigation. Also used herein for a low dam to control stream gradient, typically associated with small streams or the low channel of a floodplain or other channel.

Collector Street: Collector streets are public roads that serve moderate traffic volumes. Collector street systems link neighborhoods and industry with the arterial street system. These streets not only serve traffic circulation movements between arterials, local residential streets, and low density areas, but also serve through traffic within local areas. Collector streets provide access to abutting properties consistent with the desired level of service (Town of Prescott Valley, 2022).

Concentration Point: The specific location within a watershed where water from the most hydraulically distant point will reach, essentially marking the outlet of the watershed and representing the point where all runoff from the basin will eventually converge.

Confluence: The point at which two streams converge.

Core Sample: Cylindrical sections of subsurface earth material taken from an existing well and used for geologic analysis. Core samples can provide data regarding particle size distribution, moisture content, bulk density, and shear strength.

Crest: That portion of the drop structure which controls the gradient of the upstream channel. In a vertical drop structure, the crest is a wall typically constructed of reinforced concrete or sheet pile. In a sloping drop structure, the crest is the portion of the drop at the top of the slope and usually incorporates a buried cutoff wall for seepage control.

Critical Depth: The depth at which a given discharge flows in a given channel with a minimum specific energy. For depths greater and lower than critical, the flow is said to be subcritical and supercritical, respectively.

Critical Flow: Flow at critical depth.

Culvert: A hydraulically short conduit which conveys surface water runoff through a roadway embankment or through some other type of flow obstruction. Culverts are constructed from a variety of materials and are available in many different shapes and configurations. Culvert selection factors include roadway profiles, channel characteristics, flood damage evaluations, construction and maintenance costs, and estimates of service life.

Cut-Off Wall: A barrier that prevents water from undermining a culvert's invert slab and prevents piping along the culvert barrel.

Debris Basin: Facilities designed to capture sediment, gravel, boulders, and vegetative debris that are washed out of the canyons during storms.

Degradation: A progressive lowering of the channel bed due to scour. Permanent or continuing degradation is an indicator that a change in the stream's discharge and sediment load characteristics is taking place, see Aggradation.

Delineate: To identify and map the boundaries of a watershed, drainage basin, or other water-related area, typically by using topographic maps and contour lines to determine the dividing lines between different water flow paths.

Denuded Watershed: A watershed area where the vegetation cover has been significantly stripped away, leaving the land bare due to factors like deforestation, overgrazing, or natural disasters, resulting in increased erosion and potential negative impacts on water quality within that watershed.

Design Discharge: Maximum flow a structure or channel is expected to accommodate without contradicting the adopted design constraints.

Detention Basin: A basin or reservoir where water is stored for regulating a flood. It has gravity-flow outlets for outflows during floods.

Deterministic: A deterministic model assumes that the input is exactly known; ignores variation in input by assuming fixed input.

Design Frequency: The nth-year storm for which it is expected that the structure or facility designed for that storm would experience an actual hydrological event of a given or greater magnitude, once, on average, in n years. For example, a 50-year storm has a 2 percent chance of occurring in any given year. Also called the return period, exceedance interval, or recurrence interval.

Design Peak Discharge: The maximum rate of water flow that a drainage structure, such as a culvert or bridge, is designed to handle during a specific storm event, typically based on a chosen recurrence interval, representing the highest flow expected at that location under a defined design criteria; the peak discharge is used as the basis for sizing the structure to safely accommodate floodwaters.

Discharge: Volume of water passing through a channel during a given time.

Distributary Flow: Distributary flow areas have channels which split and rejoin in a complex pattern. The number of channel forks commonly exceeds the number of channel confluences, creating a distributary, rather than tributary drainage pattern.

Drainage Basin: A geographical area which contributes to surface runoff to a particular concentration point. The terms "drainage basin," "tributary area," and "watershed" are used interchangeably.

Drainageway: A route or watercourse along which storm runoff moves, or may move, to drain a catchment area.

Drop Structure: A structure constructed in a conduit, canal, or open channel for the purpose of gradient (bottom slope) control.

Drywell: An engineered subsurface chamber designed to accept surface runoff and allow it to drain into the subsurface strata.

Embankment: A man-made earth fill structure constructed for the purpose of impounding water.

Emergency Spillway: An outflow spillway from a stormwater detention/retention facility that provides for the safe overflow of floodwaters for storm events in excess of the design capacity of the Primary Outlet Structure, or in the event of malfunction or debris blockage of the Primary Outlet Structure.

Energy Grade Line (EGL): An inclined line representing the total energy of the flowing water. For an open channel, the EGL is above the water surface by a value of the velocity head. In a closed pressure conduit, the EGL is above the pressure head line by a value of the velocity head. See Hydraulic Grade Line.

Ephemeral: A stream that goes dry during rainless periods.

Equilibrium: The state of balance of natural channels between hydraulic forces or actions. Equilibrium occurs when the streambed has achieved a graded condition when the slope and energy of the stream are just sufficient to transport material delivered to it. Natural channels which have small changes resulting from periods of low and high flows are considered in equilibrium.

Erosion: Displacement of soil particles on the land surface due to water or wind action.

Filter: Layer of fabric, sand, gravel, or graded rock placed (or developed naturally where suitable in-place materials exist), between the bank revetment and soil for one or more of three purposes: 1) to prevent the soil from moving through the revetment by piping, extrusion, or erosion; 2) to prevent the revetment from sinking into the soil; and 3) to permit natural seepage from the streambank, thus preventing buildup of excessive hydrostatic pressure.

Fine Sediment Load (or Washload): That part of the total sediment load that is composed of particle sizes finer than those represented in the bed. Normally, the fine sediment load is finer than 0.062 mm for a sand-bed channel. Silt, clay, and sand could be considered fine sediment load in a coarse gravel and cobble bed channel. The washload generally comes from the watershed.

First Flush: The initial runoff of rainwater during a storm event, where the concentration of pollutants is significantly higher compared to the rest of the storm.

Flap Gate: A flow control device that functions as a check valve, allowing water to flow through it in only one direction.

Flood Gage or Stream Gage: A method of determining the depth and velocity in an open channel.

Flood Frequency Analysis: A technique used to relate the magnitude of extreme runoff or river flow events to their frequency of occurrence through the use of probability distribution functions.

Flood Insurance Rate Maps (FIRM): Official map of a community on which FEMA has delineated the Special Flood Hazard Areas (SFHAs), the Base Flood Elevations (BFEs) and the risk premium zones applicable to the community.

Flood Magnitude: How much water in the channel is flowing past a certain point in a given period of time (aka. discharge).

Flood Peak: The largest value of the runoff flow that occurs during a flood event, as observed at a particular point in the drainage basin.

Flood Routing: The mathematical simulation of a flood wave as it moves downstream along a watercourse or through a detention/retention facility.

Flood Stage: The gage height of the lowest bank of the reach in which the gage is situated. The term "lowest bank" is, however, not to be taken to mean an unusually low place or break in the natural bank through which the water inundates an unimportant and small area. The stage at which overflow of the natural banks of a stream begins to cause damage in the reach in which the elevation is measured.

Floodplain: A floodprone area of land adjoining or near the channel of a watercourse which has been, or may be, covered by floodwaters. A floodplain functions as a temporary channel or reservoir for overbank flows.

Floodway: A specific regulatory district within the floodplain as identified on FEMA flood hazard boundary maps; or the channel of a river or other watercourse and the adjacent land area necessary to discharge the 100-year flood without cumulatively increasing the water surface by more than one foot and without creating hazardous velocities of floodwaters.

Flow Regime: A system of describing flow that is effected by viscosity and gravity. Flow is described as being subcritical, critical, or supercritical.

Freeboard (FB): The vertical distance above a design water surface elevation that is provided as a contingency or allowance for waves, surges, water-borne debris, or other factors.

Frequency Storm: The frequency storm method is designed to produce a synthetic storm from statistical precipitation data. In the U.S., the data typically comes from the National Weather Service, given in the form of maps, where each map shows the expected precipitation depth for a storm of specific duration and exceedance probability.

Froude Number: A dimensionless number (expressed as $V/(gy)^{0.5}$) that represents the ratio of inertial to gravitational forces. High Froude numbers (values greater than 1) indicate supercritical flow with associated high velocity and scour potential.

Gabion or Wire-Enclosed Basket: A basket or compartmented rectangular container made of steel wire mesh. When filled with cobbles or rock of suitable size, the gabion becomes a flexible and permeable block with which flow control structures can be built.

Geomorphology: That branch of both physiography and geology that deals with the form of the earth, the general configuration of its surface, and the changes that take place due to erosion of the primary elements and in the buildup of erosional debris.

Geotechnical: Relating to the application of geological science to civil engineering; the type of civil engineering that deals with the mechanics of rocks and soil.

Grade Control Structure (sill, check dam): A structure across a stream channel placed bank to bank (usually with its central axis perpendicular to flow) to control bed slope and prevent scour or headcutting.

Gradient: The rate of change of a characteristic per unit of length. The term is usually applied to such things as channel/stream bed slope elevation, conduit invert elevation, velocity, pressure, etc.

Grading: The process of reshaping land at a construction site to create a level base or specified slope.

Green Infrastructure/ Low Impact Development: A management approach and set of practices that can reduce runoff and pollutant loadings by managing runoff as close to its source(s) as possible.

Headcuts or Headcutting: Channel bottom erosion moving upstream along a waterway indicating that a readjustment of the channel's slope and its discharge and sediment load characteristics is taking place. Headcutting is evidenced by the presence of abrupt vertical drops in the stream bottom or rapidly moving water through an otherwise placid stream. Headcutting often leaves stream banks in an unstable condition as it progresses along the channel.

Hydraulics: Fluid in motion; the conveyance of water through pipes and channels (stream, river, lake, ocean).

Hydraulic Conductivity: A measure of how easily water can pass through soil or rock.

Hydraulic Conductivity (effective): The rate of flow of water through a porous medium that contains more than one fluid, such as water and air in the unsaturated zone.

Hydraulic Conductivity (saturated): The rate at which a saturated soil transmits water.

Hydraulic Grade Line (HGL): For an open channel, it is coincident with the water surface. In a closed pressure conduit, it is the line representing the pressure head of the conduit. HGL will always be EGL minus the velocity head. See Energy Grade Line.

Hydraulic Jump: The hydraulic jump is an abrupt rise in the water surface which occurs in an open channel when water flowing at supercritical velocity is retarded by water flowing at subcritical velocity or a stationary pool. The transition through the jump results in a marked change in energy, evidenced by turbulence of the flow within the area of the jump. The hydraulic jump is often used as a means of energy dissipation.

Hydraulic Structures: The facilities used to impound, accommodate, convey, or control the flow of water, such as dams, weirs, intakes, culverts, channels, and bridges.

Hydrograph: The functional relationship between time and flow discharge, as observed at a particular point within a drainage basin. In the case of a detention/retention facility, an Inflow Hydrograph depicts the relationship of time and runoff inflow to the facility, and an Outflow Hydrograph is a graph of flow discharge from the facility versus time.

Hydrology: The study of water; generally focuses on the distribution of water and interaction with the land surface and underlying soils and rocks.

Hydrostatic Load: The force exerted on a surface by a stationary body of fluid.

Impervious/Impermeable: A term applied to a material through which water cannot pass, or through which water passes with great difficulty.

Infiltration: The movement of water into and through the soil.

Infiltration Trench: Narrow, linear ditches with highly permeable soils that collect rainwater from nearby surfaces and allow it to quickly soak into the ground.

Injection Well: A structure that pumps fluids underground into porous geologic formations.

Initial Abstraction: The amount of precipitation that must fall before surface excess (runoff) results.

Intake (structures): A designed structure that allows water to be drawn from a source like a river, reservoir, or lake, diverting it into a conduit (pipe or canal) for further use.

Invert: The lowest point in the channel cross section or at flow control devices such as drop structures, dams, or outlet structures, see Thalweg.

Inverted-crown: The high point in the middle of a road's cross section, from where the road slopes downward toward the edges.

Isopluvial: Greek for "same rainfall," in the precipitation frequency context, a map showing the precipitation depth for the same annual exceedance probability and duration everywhere.

Jurisdiction or Jurisdictional Agency: Town of Prescott Valley

Lateral Migration or Lateral Stream Migration: Change in position of a channel by lateral erosion of one bank and simultaneous accretion of the opposite bank. Movement in which the material has a dominant lateral component.

Local Scour: Scour in a channel or on a floodplain that is localized at a pier, abutment, or other obstruction to flow.

Local Street : Local streets are public roadways that serve relatively low traffic volumes. The local street system provides access to residents, businesses, or other abutting properties. The traffic volume generated by the adjacent land uses are largely short trips, or a relatively small part of longer trips where the local road connects to the collector roadway system. Local streets offer the lowest level of mobility, and usually do not provide access to transit services (Town of Prescott Valley, 2022).

Low Flow Channel: A channel within a larger channel which typically carries low and/or normal flows.

Major drains: Include natural and man-made channels and conduits that serve watershed areas from 160 acres to about 10 square miles.

Master Planning: A 'systems" approach to the planning of facilities, programs and management organizations for comprehensive control and use of stormwater within a defined geographical area or drainage basin.

Minor Drainages or Minor drains: Serve watershed areas up to 160 acres are normally the drains associated with subdivision development.

On-site Detention/Retention: The temporary storage of excess storm runoff in the upper area of a drainage basin. This type of facility is typically within a subdivision, primarily by an individual development and generally irrespective of watershed features.

Orifice: A hole in the outlet structure of a stormwater storage facility sized to drain the facility at a specific rate of flow.

Outfall: A pipe that discharges to a pond, lake, stream, or ocean.

Outlet Structure: A hydraulic structure placed at the outlet of a conduit, open channel, spillway, etc., for the purpose of dissipating energy and providing a transition to the channel or conduit downstream. Outlet structures may consist of culverts, weirs, orifices (gated or un-gated), drywells, or any combination thereof.

Overbank Floodplain Storage: The storage within the floodplain is responsible for attenuating the flood hydrograph and, to some extent, delaying the flood wave.

Particle Size Distribution: The analysis of the range of particle sizes in a soil sample.

Peak Flow/Peak Discharge: The point of the hydrograph that has the highest flow.

Percolation: The movement, under hydrostatic pressure, of water through the interstices of a rock or soil.

Plasticity Index: The range of moisture contents over which the soil deforms plastically (change shape without breaking or crumbling).

Playa: An area of flat, dried-up land, especially a desert basin from which water evaporates quickly.

Porosity: The ratio of pore volume to total rock, sediment, or formation volume.

Pressure Head: In a closed pressure conduit, it represents the energy per unit weight stored in the fluid by virtue of the fluid being under pressure expressed as P/y. Generally having the units of feet. In an open channel, the pressure head is zero.

Primary Outlet Structure: Also known as the Primary Spillway or Principal Spillway, it is the main outlet structure by which stormwater is discharged from the detention/retention facility.

Pump Station: A facility housing stormwater pumps, controls, power plants and their appurtenances.

Regression Equation: A mathematical equation that is fitted to historical data in order to analyze the relationship between variables in the system domain. It is used to make predictions and understand the correlation between different factors.

Regulatory Flood Elevation (RFE): An elevation 1 foot above the BFE.

Resistivity: Measurement of electrical resistance of a conductor of unit cross sectional area and unit length.

Retaining Wall: A structure that holds or retains soil behind it.

Retention Basin: A basin or reservoir wherein water is stored for regulating a flood, however, it does not have gravity-flow outlets for outflows during floods as detention basins do. The stored water must be disposed of by some other means such as by infiltration into soil, evaporation, injection (or dry) wells, or pumping systems.

Return Period: The average interval of time within which the given flood will be equaled or exceeded once. When the recurrence interval is expressed in years, it is the reciprocal of the annual exceedance probability.

Riprap: Pieces of broken stone used as lining to protect the sides of waterways from erosion.

Runoff: The portion of precipitation on land that ultimately reaches streams; especially water from rain or melted snow that flows over the ground surface.

Scour: Erosion due to flowing water, usually considered as being localized as opposed to general bed degradation.

Sediment (or Fluvial Sediment):Fragmental material transported, suspended, or deposited by water.

Sedimentation: The process where particles like sand, silt, and clay carried by the river settle at the bottom as the water flow slows down.

Seepage: The movement of water through pores and voids of pervious material such as soil, gravel, and synthetic filter media, etc.

Seismic Exploratory Operations: Sound waves are sent into the ground to map underground structures and identify potential resources like oil, gas, or minerals based on how the waves reflect back.

Seismic Shot Hole: A drilled hole in the ground used to place small explosive charges or energy sources to generate shock waves for mapping underground rock layers during seismic exploration.

Shotcrete: Mortar or concrete pneumatically projected at high velocities onto a surface.

Sill: A raised edge at the downstream end of a stilling basin. The sill typically has a notch or opening to allow normal stream flows to pass through and/or to allow the basin to drain completely following a storm.

Siltation: The process by which fine particles like silt accumulate and settle in bodies of water, often reducing water flow.

Siphon: A tube or pipe used to move liquid from one container to another, typically using gravity to start and maintain the flow after the liquid is lifted over a barrier.

Sludge: A solid, semisolid, or liquid residue that is created as a byproduct of wastewater treatment.

Soil Cement: A designed mixture of soil and Portland cement compacted at a proper water content to form a veneer or structure which when placed on a streambed or bank can prevent erosion. Also referred to as Cement Stabilized Alluvium.

Soil Horizon: A distinct layer within a soil profile that has different physical, chemical, and biological properties than the layers above and below it.

Soil Moisture Deficit: The soil moisture capacity minus the actual soil moisture, or the amount of water needed to increase the soil's water content to reach field capacity.

Spillway: (a) A low-level passage serving a dam or reservoir through which surplus water may be discharged; usually an open ditch around the end of a dam, or a gateway or a pipe in a dam. (b) An outlet pipe, flume, or channel serving to discharge water from a ditch, ditch check, gutter, or embankment protector.

Stage: The depth of water within a stormwater storage facility, as measured above an established datum.

Station: A specific, precisely located point on the ground whose position has been determined through surveying methods, serving as a reference point for further measurements within a survey project.

Steady Flow: Flow in which the flow quantity does not vary with time at any location along the channel.

Stochastic: Probabilistic modeling of hydrological processes which have random components associated with them; stochastic models embrace random variation by attempting to explicitly describe it.

Storage Reservoir of Pump Station: A reservoir wherein peak flows from storm drains are stored for reducing capacity requirements of the pump station to pump runoff to an appropriate outlet.

Storm Drainage System: A drainage system for collecting runoff of stormwater on roadways and removing it to appropriate outlets. The system includes inlets, catch basins, storm sewers, main drains, storage reservoirs, detention basins and pump stations.

Stormwater: Rainwater or melted snow that runs off streets, lawns, and other sites. When stormwater is absorbed into soil, it is filtered and ultimately replenishes aquifers or flows into streams and rivers.

Stormwater Detention Facility: A stormwater storage facility which is temporarily stores surface runoff and releases it at a controlled rate through a positive outlet.

Stormwater Retention Facility: A stormwater storage facility which stores surface runoff. Stored water is infiltrated into the subsurface or released to the downstream drainage system or watercourse (via a gravity outlet or pump) after the storm event.

Sub-basin: Smaller working units within a larger river basin or catchment area. They represent a distinct geographical area that is delineated based on the natural drainage patterns of rivers and their tributaries.

Subsurface Drainage: Drainage of stormwater runoff into the subsurface by the process of infiltration. This is typically accomplished through the use of drywells, engineered basin floors, etc.

Sump: A low-lying pit or basin designed to collect and store liquids, such as water or wastewater, often for drainage or pumping purposes.

Supercritical Flow: Flow with depth less than the critical depth and velocity greater than the critical velocity.

Surface Retention Loss: The part of the storm that does not contribute to runoff. Retention is made up of depression storage, interception, and evaporation.

Surface Water: Water on the surface of the earth.

Swale: A shallow, vegetated channel designed to collect and filter stormwater runoff from surfaces like streets or parking lots, allowing it to slowly infiltrate the ground and reduce the impact of rapid water flow.

Synthetic Unit Hydrograph: A modeled representation of how a watershed responds to a unit of rainfall, estimating the flow of water over time without requiring direct observation data.

Tailwater: The water surface elevation in the channel downstream of a hydraulic structure.

Thalweg: The line extending down a channel that follows the lowest elevation of the bed, see Invert. Not to be confused with the channel's centerline.

Topography: Physical features such as hills, valleys, and plains that shape the surface of the earth.

Total Sediment Load: The sum of suspended sediment discharge and bedload discharge or the sum of bed material discharge and washload discharge of a stream.

Trash Rack: A metal bar or grate structure designed to prevent blockage of the structure by water-borne debris.

Unsteady Flow: Conditions where the fluid properties (such as velocity, pressure, temperature, and density) at a particular point in space will change with time.

Vadose Zone (or unsaturated zone): The portion of the subsurface above the water table. It contains, at least some of the time, air as well as water in the pores. Its thickness can range from zero, as when a lake or marsh is at the surface, to hundreds of meters, as is common in arid regions.

Valley Gutter: A shallowly-depressed, paved surface (generally concrete), which ordinarily crosses a street at an intersection, which allows continuous discharge of flood flows across an intersection.

Vegetated Filter Strip: Gently sloping, densely vegetated areas that are designed to treat runoff as sheet flow from adjacent impervious surfaces.

Velocity Head: Represents the kinetic energy of the flowing fluid generally expresses as $V^2/2g$ in feet, but actually is the energy per pound of flowing fluid.

Vortex: Local current accelerations which cause a whirling or circular motion that tends to form a cavity or vacuum at its center, thus moving sediment toward the cavity.

Wash: A dry watercourse that fills with water after heavy rain.

Wastewater: a generic term that can include graywater (water from non-toilet plumbing fixtures like taps and showers) and blackwater (water with a high concentration of organic matter and bacteria from toilets, kitchens, and dishwashers)

Watercourse: A body of water that flows in a defined channel with a bed and banks. It can be a natural or man-made body of water. This term includes not only rivers and creeks but also springs, lakes, and marshes through which flowing streams pass or originate.

Waters of the United States: 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.

Watershed: An area confined by drainage divides, often having only one outlet for discharge. See Basin Area.

Watershed Outlet: The lowest point on a watershed's boundary where water flows out of the area.

Weir: A notch of regular form through which water flows. A weir may be a depression or notch in the side of an outlet structure or a depression of specific shape in the embankment of a stormwater storage facility. Classified in accordance with the shape of the notch, there are rectangular weirs, V-notch weirs, trapezoidal weirs, and parabolic weirs.

Wetlands: Areas that are inundated or saturated by surface or ground water at a frequency and duration sufficient to support, and that under normal circumstances do support, a prevalence of vegetation typically adapted for life in saturated soil conditions.

Wetting Front Capillary Suction: The wetting front is the area where wet and dry soil meet and is characterized by a steep hydraulic gradient. Capillary suction is the process by which liquids move through small passages, or capillaries, due to surface tension. The wetting front suction head describes the attraction of water within the void spaces of the soil column.

Appendix A. Sand and Gravel Mining Guidelines

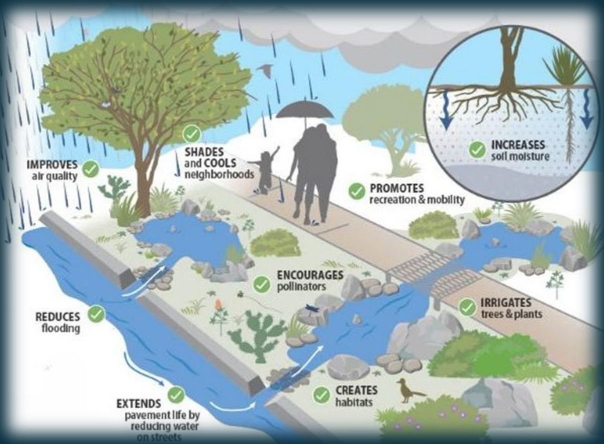
This section is currently being developed and will be added when available.

Until the section is developed, Guideline 3, Evaluation of River Stability Impacts Associated with Sand and Gravel Mining, of the ADWR State Standard for Watercourse System Sediment Balance, SS 5-96, shall be used as a standard. Also, see the ADEQ website regarding permits needed for a sand and gravel operation: https://www.azdeq.gov/permits-needed-sand-and-gravel-operation.

Town of Prescott Valley

Uniform Drainage Policies and Standards Manual

Appendix B: Green Stormwater Infrastructure



FEBRUARY 2025

PRESCOTT VALLEY

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FIGURES

TABLES

1 Introduction

This appendix is intended to facilitate the implementation of Green Stormwater Infrastructure (GSI) best practices in new development. GSI is a set of practices that utilize natural systems of soil and plants to improve water quality, reduce flooding, and enhance long-term water security through deep infiltration to connect clean water resources to underground aquifers.

GSI enhances the local water cycle by slowing water down and allowing vegetation to infiltrate runoff into the soil. Some of this water cools the air through evapotranspiration. As shown in the adjacent graphics, GSI can improve the overall availability of water resources for the Town by making more water available through rainfall while also enhancing quality of life as a result of greater access to lush nature, reduced flooding from water storage in the soil and aquifer, and greater long-term water security for the community through increased aquifer storage.

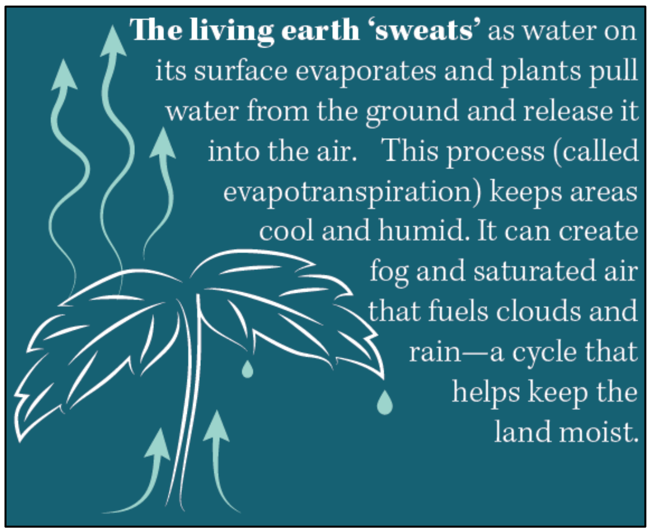


Figure 1-1. Living Earth 'Sweats'

Many communities in a well-intended effort to conserve water, eliminate or reduce vegetation in landscapes. This creates hot and dry areas that will receive less rainfall in the long term. GSI provides an opportunity to create multiple benefits such as enhanced shade, urban cooling, wildlife habitat, and traffic calming while also solving multiple problems such as mitigation of nuisance flooding, conserving water by using stormwater for irrigation, and improving stormwater quality.

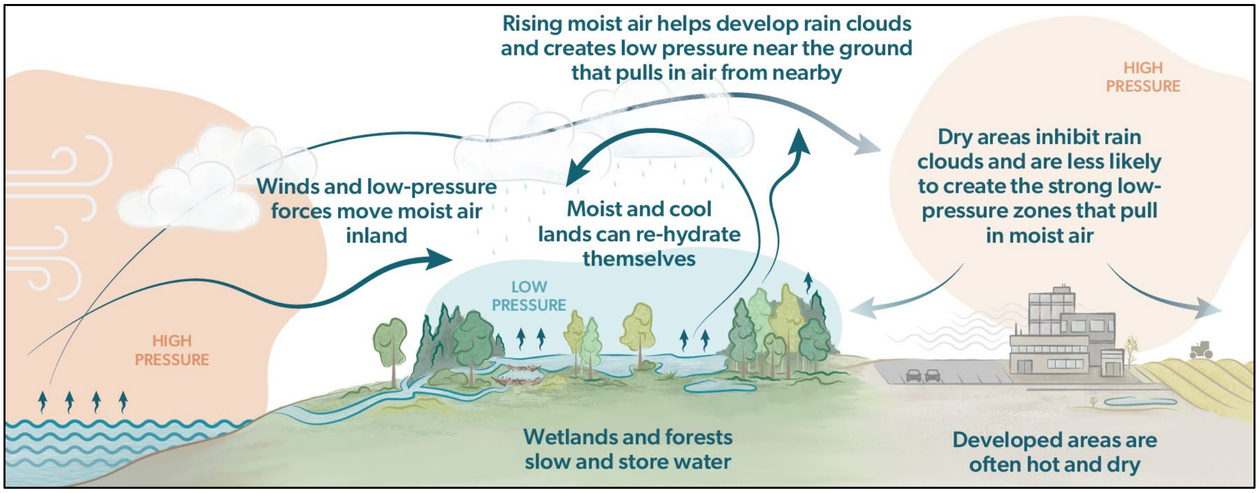


Figure 1-2. Wetlands and Forests vs Developed Areas Effects

The difference between success and failure with GSI can be a matter of inches. Following the guidance in this document will help ensure GSI is implemented effectively.

Images from Beavers Drive Healthy Water Cycles courtesy of Project Beaver (www.projectbeaver.org)

2 Green Stormwater Infrastructure Details

Table 2-1 below summarizes GSI details, including typical applications, and considerations. The descriptions and figures on the following pages explore site and other considerations for these practices in more detail. The details are grouped by basin components (900–908), conveyance features (909–913), active practices (914), in-street basins (915-917), and infiltration technologies (918-919). The 900 series was started as a new GSI series to fit within the numbering system of the Quad Cities Standard Details.

Table 2-1. Summary of GSI Practices and Considerations

Detail	GSI Practice	Description	Application	Considerations
900	Tree Planting	Utilizes soil health and water harvesting practices to maximize health of trees and minimize maintenance and water consumption	Any tree planted within the Town limits	Maximizing basin area to provide irrigation benefit if space is highly constrained
901	Stormwater Harvesting Basin	Captures stormwater flowing from adjacent and hardscape surfaces (streets, driveways, sidewalks, and roofs) to remove pollutants and beneficially irrigate native vegetation	Best used in areas with at least 5 feet of unconstrained (free of utilities) width	Ensure critical elevations for inlet, outlet, and overflow will function as desired, select the right plant for the right place to minimize maintenance
902	Sediment Trap	Dissipates energy of stormwater entering feature to capture sediment	All details with retention areas	Size trap dimensions for sediment load and maintenance schedule
903	Curb Opening	An opening (cut or scupper) in curb that allows stormwater to enter landscapes below street grade safely	Where a curb is obstructing water from entering into landscape	Ensure sufficient drop (at least 2”) behind curb so opening is not obstructed by debris, ensure future paving does not prevent flows from entering opening
904	Flush Curb Inlet	Allows stormwater to enter landscapes below street grade safely	Parking lots, in-street features	Stabilize soil surfaces across entire inlet area impacted by design flow and velocity

Detail	GSI Practice	Description	Application	Considerations
905	Compost Chimney	Creates positive drainage to areas below the elevation of the finished basin bottom and allows for larger volumes of stormwater to infiltrate from the vertical columns of biologically active organic matter to irrigate vegetation and prevent evaporation	All features with retention	Prevent sediment from entering feature with appropriately sized and maintained sediment traps and vegetated buffers. Mulch, compost, and biological soil amendments should be replenished as needed to maximize water conservation and deep infiltration
906	Bioretention Basin	High infiltration soil mix design combining coarse sand, compost (fungal dominant is ideal such as mature leaf & worm compost), and topsoil and/or native soil with low clay content	Areas with high design flows and low landscape area, best used in areas with at least 5 feet of unconstrained width	Enhance survival of vegetation by minimizing sand composition, ensuring sufficient biological activity with high quality compost, native soils with high clay content not suitable
907	Infiltration Trench	Provides underground storage, infiltration (if soils allow or are amended), and conveyance (with optional underdrain)	All features with retention	Prevent sediment and other fines from entering infiltration trench with sediment capture practices such as sediment traps, One Rock Dams, Zuni bowls, and check dams
908	Domed Overflow Structure	Directs ponded water to additional GSI or storm drain system	All features with retention	Ensuring floatable debris expected will not clog drain grate
909	Vegetated Swale	Conveys stormwater and protects soil with the use of vegetation and/or rock dams	Conveyance where design velocities do not exceed 3 fps	Protecting soil while vegetation establishing
910	Check Dam	Rock or concrete structure that slows runoff and prevents erosion	Slopes, eroding water courses, vegetated swale	Ensure erosion does not occur at bottom of spillway or around the sides of dams with hand-placed rock aligned to resist forces from runoff

Detail	GSI Practice	Description	Application	Considerations
911	One Rock Dam	Rock structure the height of one rock to stabilize soil, slow water, and facilitate vegetation establishment	Slopes, eroding water courses, vegetated swale	Top of downstream row of rock must be flush with adjacent grade for stability and to prevent erosion
912	Zuni Bowl	Rock structure used to dissipate energy, manage sediment, and often to arrest a head cut	Slopes, eroding water courses, vegetated swale	Anchoring rocks to build strength opposing the force of runoff flows
913	Sheet Flow Spreader	Rock structure shaped to spread concentrated runoff to reduce erosive forces by creating sheet flow conditions	Slopes, eroding water courses, vegetated swale	Top of downstream row of rock must be flush with adjacent grade for stability and to prevent erosion
914	Cistern	Plastic, fiberglass, concrete, or metal container for storage of rainwater from hardscapes, most often roofs	Sites where supplemental irrigation is needed to establish and maintain vegetation	Utilize a bleed pipe where detention capacity is needed for flood mitigation, careful planning to effectively utilize stored rainwater
915	Traffic Circle	In-street traffic calming feature that utilizes practices from the stormwater harvesting basin or bioretention basin details for landscape areas	Street intersections where utility conflicts are minimal, and traffic calming needed	Locating vegetated traffic circles where topography allows for stormwater irrigation while also minimizing maintenance
916	Chicane	In-street traffic calming feature with practices from the stormwater harvesting basin or bioretention basin details for landscape areas	Crowned streets in need of traffic calming	Managing sediment to ensure long-term infiltration, invasive plants

Detail	GSI Practice	Description	Application	Considerations
917	Street Width Reduction	In-street traffic calming feature with practices from the stormwater harvesting basin or bioretention basin details for landscape areas to reduce street widths	Crowned streets in need of traffic calming and/or width reduction	Managing sediment to ensure long-term infiltration, invasive plants
918	Permeable Block	Concrete blocks with sufficient capacity to allow water to infiltrate, reduce runoff, and improve safety without clogging	Roads, parking lots, and other hardscapes where adjacent landscape is insufficient for design volume	Sediment management and appropriate maintenance to ensure pores do not clog over time, ensure design and installation accounts for all weather conditions
919	Deep Infiltration System	Technology that can effectively bring water below the evaporative zone of at least 30 feet to ensure connection to deep water supplies.	Retention areas including multipurpose fields such as retention basins with athletic fields.	Infiltration goals based on volume, time, depth to groundwater, as well as other planning and design considerations

2.1 Basin Components

2.1.1 Tree Planting

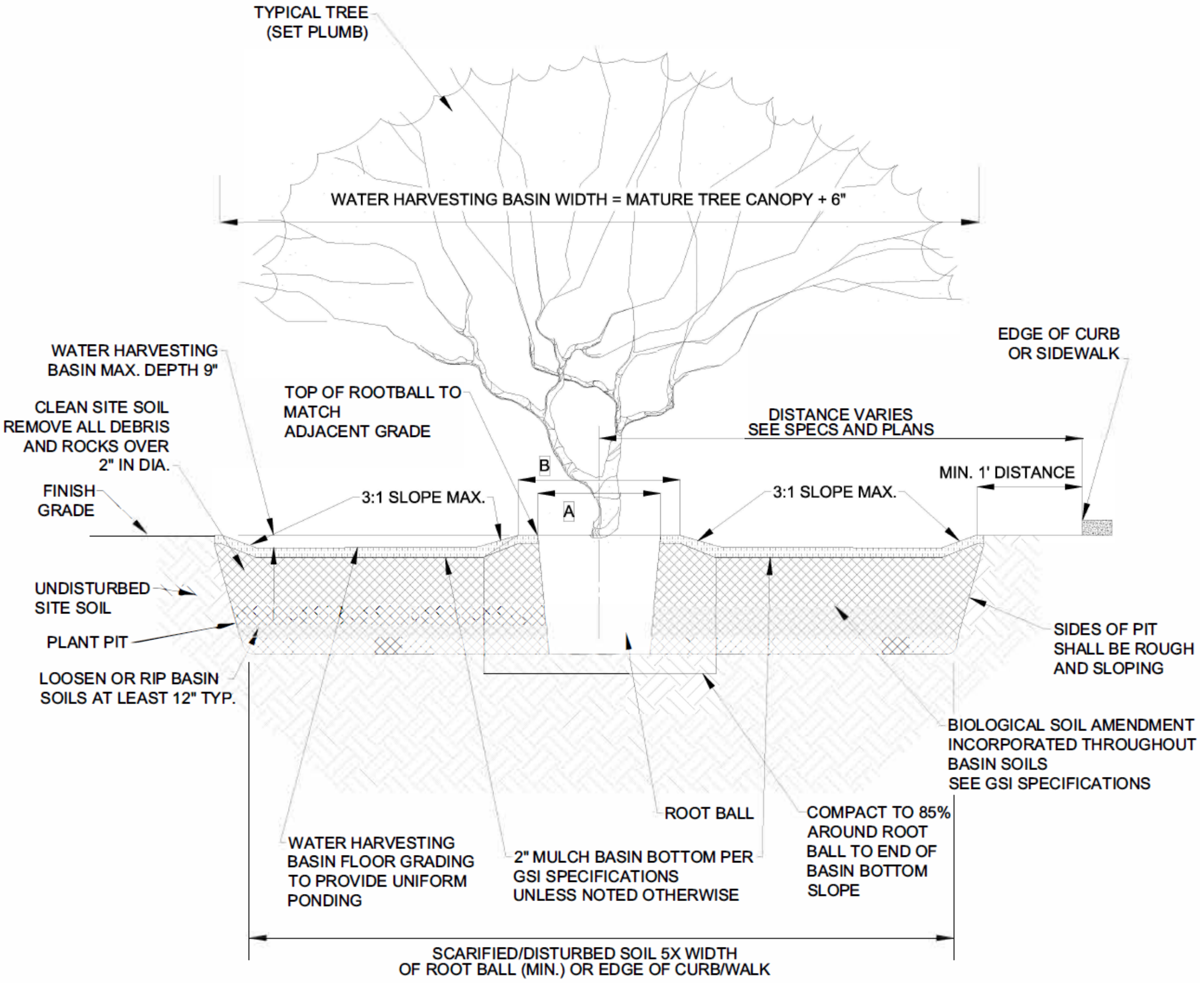
Trees are essential components of urban GSI providing critical shade to prevent water from evaporating, habitat and cool for humans and animals as well as other community benefits. Whether planted in a GSI feature or some other type of landscape, the following practices are critical to ensure a healthy tree and to minimize long-term costs.

Bioavailable water and nutrients through the plant establishment phase as well as effective maintenance once a tree is established are critical to ensure a healthy tree canopy for the Town. Tree death in urban environments is common due to limited resources and hard environmental conditions. In order to minimize resources required for successful tree planting, the following detail addresses these needs through water harvesting with a basin for irrigation and capturing and retaining organic materials such as plant debris and wood mulch as well as biological soil amendments (BSAs) to ensure water and nutrients present in the soil are accessible to the tree.

The basin can be larger than the mature tree canopy but should be graded to ensure irrigation water and rainfall soak into the canopy root zone at the edge of the root ball at the time of planting. To minimize establishment irrigation costs a tree well or small irrigation basin can be graded to provide a direct irrigation benefit during establishment to ensure enough water is provided to the establishing tree. This smaller well must be removed as the tree canopy expands to ensure even distribution of irrigation water and rainfall. Using BSAs with organic wood mulch is critical for tree establishment to extend the benefit of rainfall and irrigation between watering and rain events. Degraded urban soils do not have sufficient soil life, especially the fungal networks that help vegetation get water and nutrients during drought and heat stress. This important biology is destroyed by ground disturbances such as construction, earth moving, tilling, and any other process that removed the native topsoil with established native biology and this biology can also be destroyed by removing ground cover from agricultural practices or as a result of drought (see Figure 2-1 and Figure 2-2).

Figure 2-1. Water Harvesting Basin Around a Mesquite Tree to Capture Rainwater

Source: Oscar Medina.



GREEN INFRASTRUCTURE NOTES:

1. NO PLANTS SHALL BE PRUNED EXCEPT AT THE DIRECTION OF THE AGENCY REPRESENTATIVE.
2. STAKE TREES ONLY IF NEEDED, PER DIRECTION OF THE AGENCY REPRESENTATIVE.
3. DEPTH ("D") OF PLANT PIT TO BE EQUAL TO DEPTH OF ROOT BALL.
4. ROOT PRUNE PRIOR TO PLANTING.
5. LEVEL AND ROTATE TREE FOR BEST APPEARANCE.,
6. WATER HARVESTING BASIN DIAMETER AT LEAST 6 INCHES GREATER THAN MATURE TREE CANOPY, WHERE POSSIBLE.
7. PLANTING SHELF DIAMETER B = A*1.5.
8. NO DECOMPOSED GRANITE (DG) WITHIN 5 FEET OF WATER HARVESTING BASIN.

PV STANDARD DETAIL	TREE PLANTING	REVISED: 09/26/2024	DETAIL No. 900

Figure 2-2. Detail No. 900 Tree Planting

2.1.2 Stormwater Harvesting Basin

Soil health is critical to support short and long-term functioning of a stormwater harvesting basin by improving infiltration rates over time, reducing maintenance and establishment watering needs. These benefits are a result of reduced weed pressure and more effective water use by soil microbiology. Improving infiltration through physical changes that do not preserve moisture for vegetation such as adding conventional engineered bioretention soil, sand, or gravel can result in basin failure and eliminate the multiple benefits of GSI. Installation practices such as physically loosening soil beyond the finished basin depth and adding aerobic soil biology along with appropriate maintenance such as “chop and drop” that preserves soil health by leaving pruning’s on the ground to decompose near the plant they were cut from will ensure basin infiltration rates will improve over time and will be at least 1 inch/hr while also maintaining healthy vegetation.

The basin soil surface should be covered by organic materials where possible, ideally living vegetation with deep roots such as trees and native bunch grasses (e.g., sideoats grama, purple three awn, and deer grass) that can penetrate compacted soils and enhance infiltration rates. When living vegetation coverage is not possible, organic wood mulch is a helpful option to cover the soil surface if runoff velocities will not wash mulch away. Utilizing basins that have the same inlet as the outlet is a simple method to ensure wood mulch will not float away. Compared to rock, organic mulch increases water quality and soil health benefits. Because large storm events can cause organic wood mulch to float, for basins that may be overtopped, consider placing mulch under riprap at least 4 inches in size depending on design storm.

Using the right plant in the right place minimizes maintenance and ensures successful vegetation establishment. Do not place trees near inlets, outlets, and overflows or in basin bottoms. For the health of the tree and enhanced feature performance, where possible, use native trees with healthy tap roots grown in “tall pots” or “tree pots” to ensure a robust root system. Basins in watersheds with high sediment runoff should include sediment traps to minimize clogging of infiltration areas (see Figure 2-3 through Figure 2-6).

Figure 2-3. Rocked Streetside Stormwater Harvesting Basin

Source: Watershed Management Group

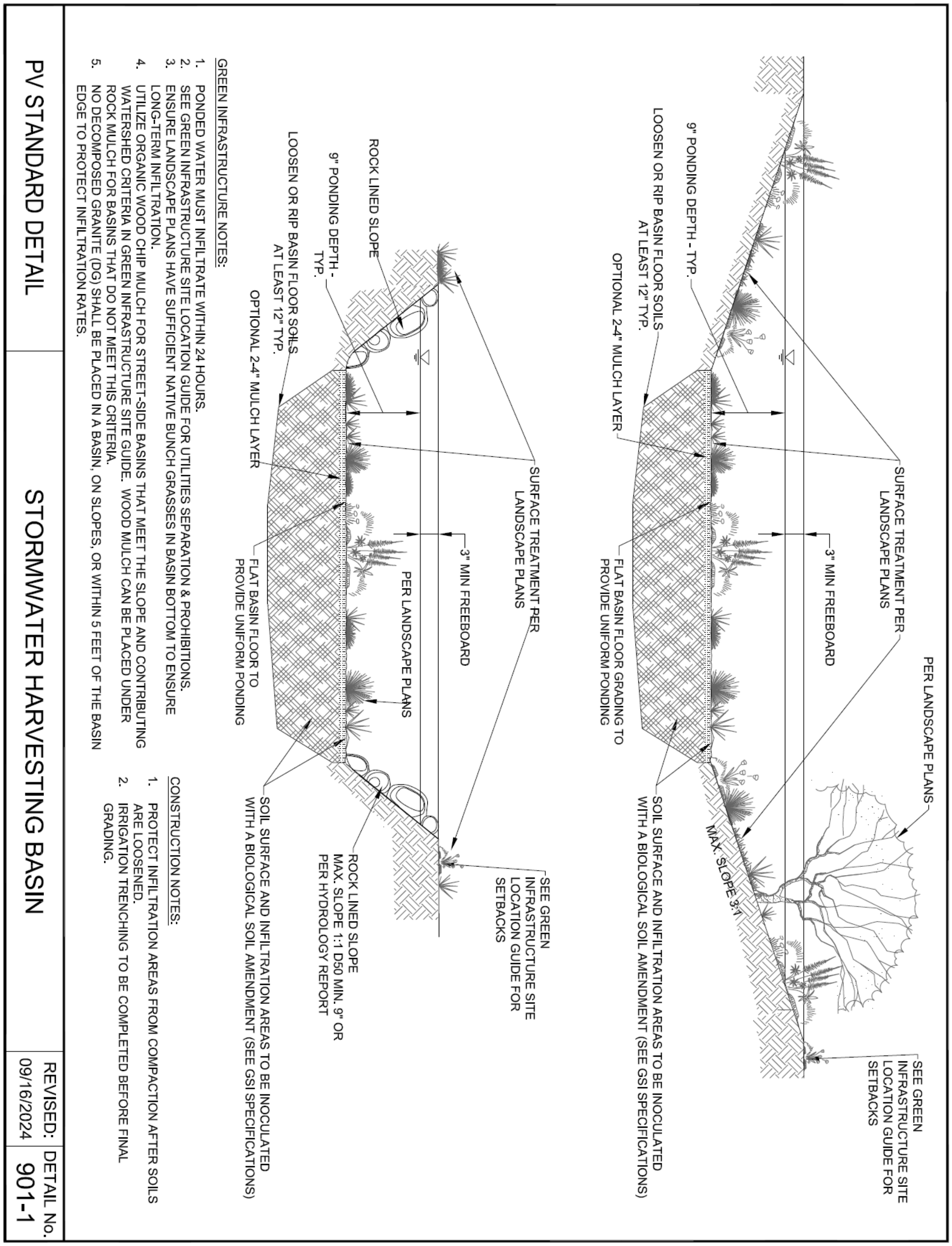
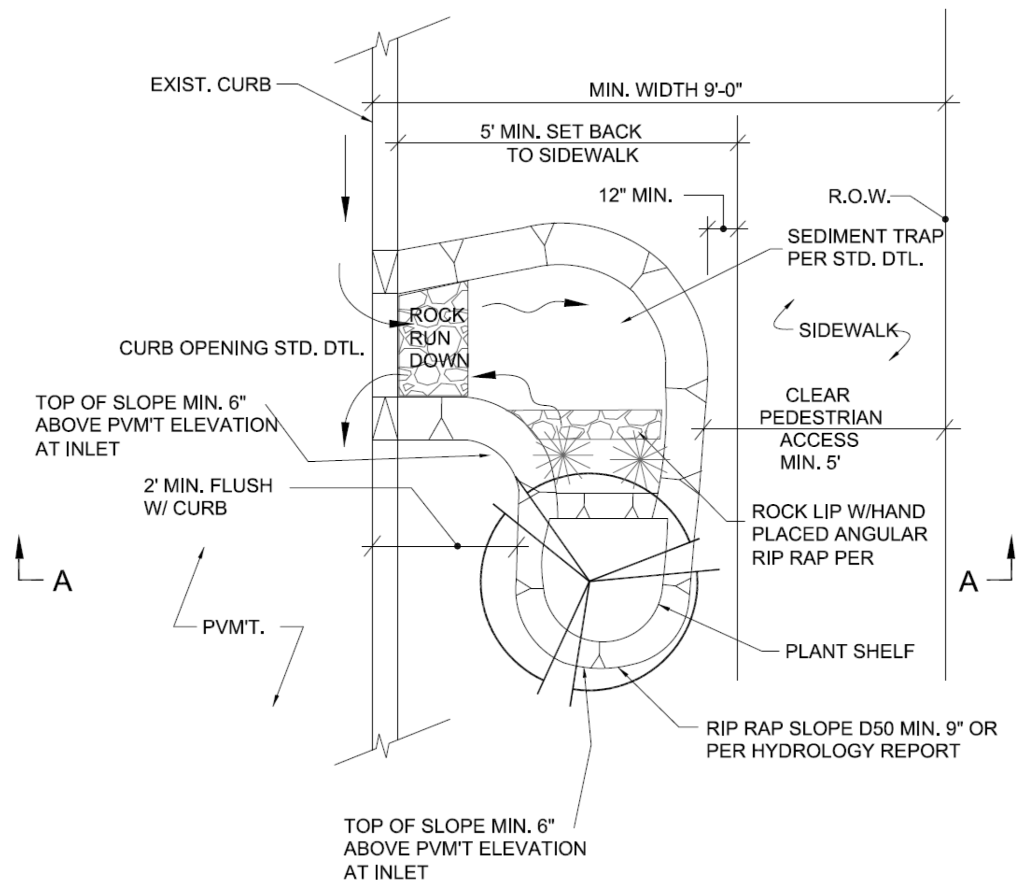


Figure 2-4. Detail No. 901-1 Stormwater Harvesting Basin



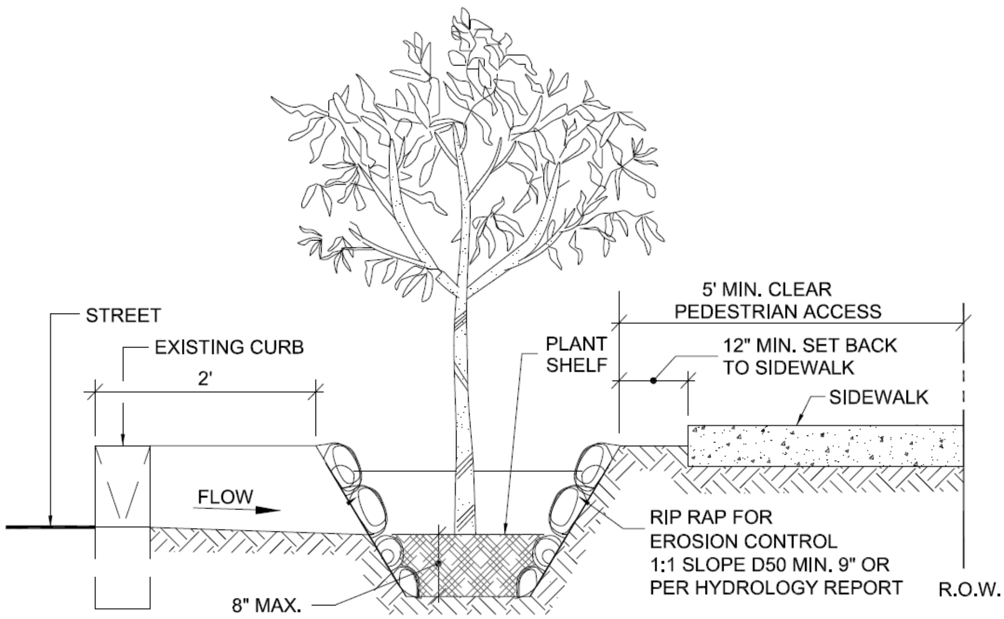
STREETSIDE STORMWATER HARVESTING DETAIL

NOTES:

1. STEP DOWN INTO FINISHED BASIN FROM ADJACENT GRADES NOT TO EXCEED 12".

PV STANDARD DETAIL	STORMWATER HARVESTING BASIN	REVISED: 09/16/2024	DETAIL No. 901-2

Figure 2-5. Detail No. 901-2 Stormwater Harvesting Basin



SECTION A-A

STREETSIDE STORMWATER HARVESTING DETAIL

PV STANDARD DETAIL	STORMWATER HARVESTING BASIN	REVISED: 09/16/2024	DETAIL No. 901-3

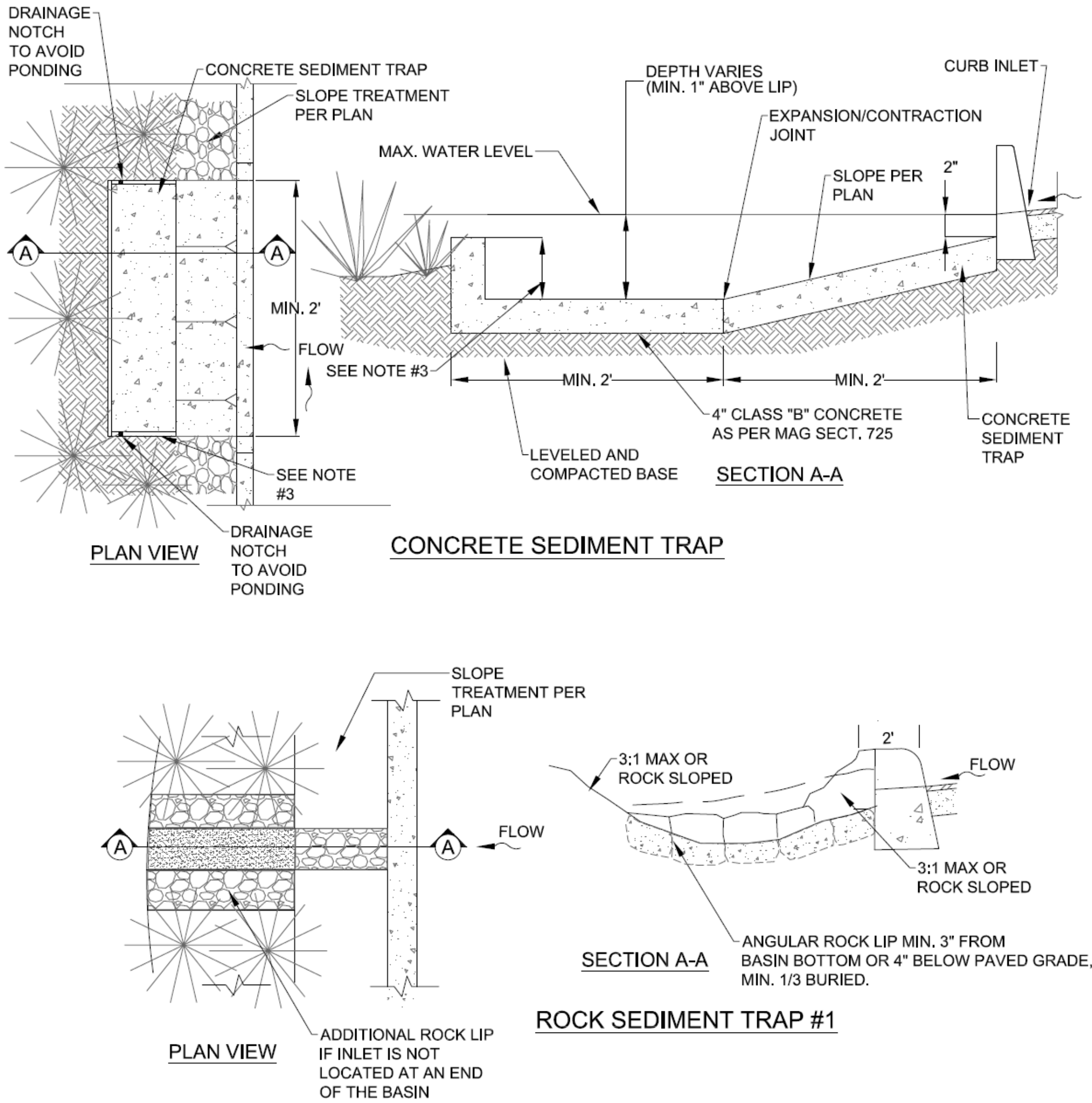
Figure 2-6. Detail No. 901-03 Stormwater Harvesting Basin

2.1.3 Sediment Trap

Any infiltration area that could receive sediment-laden stormwater will benefit from having a sediment trap upstream. Sediment traps minimize or eliminate sediment entering infiltration areas. Their sizing should focus on appropriate energy dissipation, sediment storage based on stormwater characteristics and watershed size, and ease of maintenance. A poorly designed or non-existent sediment trap can cause reduced feature capacity, decreased infiltration rates, and even complete elimination of stormwater infiltration thereby increasing maintenance costs and decreasing basin performance over time (see Figure 2-7 through Figure 2-9).

Figure 2-7. Mesa Urban Garden Rock Sediment Trap

Source: Watershed Management Group.

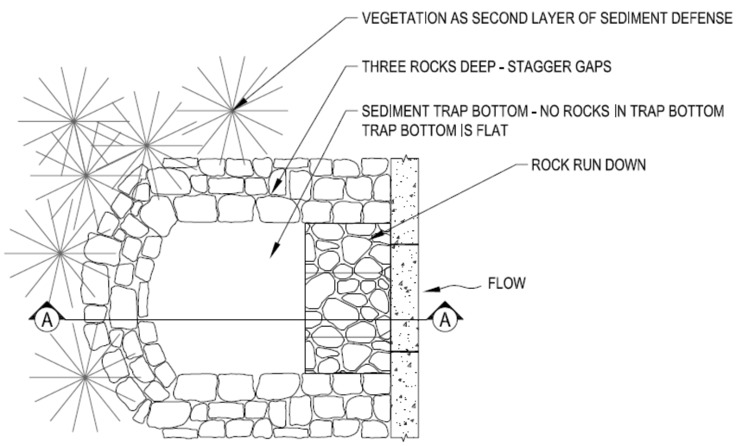


NOTES:

1. DIMENSIONS OF SEDIMENT TRAP SHALL BE DESIGNED BASED ON CONTRIBUTING WATERSHED SIZE AND SEDIMENT LOAD. REFER TO PLANS FOR SIZE OF SEDIMENT TRAPS.
2. SEDIMENT TRAP BOTTOM (THE AREA BETWEEN THE OUTER ROCK CONTAINMENT AND THE ROCK RUNDOWN) SHALL BE 1' WIDER AND 1' LONGER THAN INLET OPENING IN CURB (FOR A 30" CURB OPENING, SEDIMENT TRAP BOTTOM SHALL BE 42" WIDE AND 42" LONG).
3. TOP OF 3" LIP TO BE A MINIMUM OF 4" BELOW THE PAVEMENT SURFACE.
4. SET ALL ROCKS WITH ½ OF EACH ROCK BELOW FINISH GRADE.
5. HAND PLACE ANGULAR RIPRAP, ROCK SIZE 6" - 10" UNLESS NOTED OTHERWISE ON PROJECT PLANS.
6. NATIVE VEGETATION, EMPHASIZING GRASSES AND FORBS, SHOULD BE ESTABLISHED BACK OF SEDIMENT TRAP. PLANTS ARE A SECOND LINE OF DEFENSE IN TRAPPING SEDIMENT AND FILTERING RUNOFF.

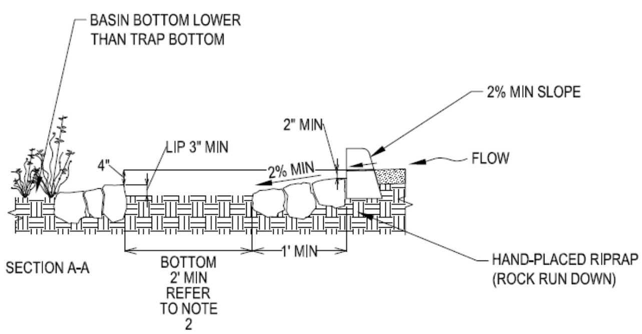
PV STANDARD DETAIL	SEDIMENT TRAP	REVISED: 09/26/2024	DETAIL No. 902-1

Figure 2-8. Detail No. 902-1 Rock Sediment Trap



PLAN VIEW

ALL RIPRAP TO BE CAREFULLY HAND PLACED: KEY ROCKS TOGETHER TO MINIMIZE GAPS, PLACE WITH FLAT SURFACE UP, SET ROCKS ½ MIN BELOW FINISH GRADE.



SECTION A-A

ROCK SEDIMENT TRAP #2

NOTES:
1. DIMENSIONS OF SEDIMENT TRAP SHALL BE DESIGNED BASED ON CONTRIBUTING WATERSHED SIZE AND SEDIMENT LOAD. REFER TO PLANS FOR SIZE OF SEDIMENT TRAPS.
2. SEDIMENT TRAP BOTTOM (THE AREA BETWEEN THE OUTER ROCK CONTAINMENT AND THE ROCK RUNDOWN) SHALL BE 1' WIDER AND 1' LONGER THAN INLET OPENING IN CURB (FOR A 30" CURB OPENING, SEDIMENT TRAP BOTTOM SHALL BE 42" WIDE AND 42" LONG).
3. TOP OF 3" LIP TO BE A MINIMUM OF 4" BELOW THE PAVEMENT SURFACE.
4. SET ALL ROCKS WITH ½ OF EACH ROCK BELOW FINISH GRADE.
5. HAND PLACE ANGULAR RIPRAP. ROCK SIZE 6" - 10" UNLESS NOTED OTHERWISE ON PROJECT PLANS.
6. NATIVE VEGETATION, EMPHASIZING GRASSES AND FORBS, SHOULD BE ESTABLISHED BACK OF SEDIMENT TRAP. PLANTS ARE A SECOND LINE OF DEFENSE IN TRAPPING SEDIMENT AND FILTERING RUNOFF.

PV STANDARD DETAIL	SEDIMENT TRAP	REVISED: 09/29/2024	DETAIL No. 902-2

Figure 2-9. Detail No. 902-2 Sediment Trap

2.1.4 Curb Opening

Areas with curbs that restrict the flow of stormwater to landscape areas can benefit from curb cuts. Curb openings can be incorporated easily into new development and when retrofitting developed communities. Ensuring conveyance of water into landscapes is not prevented at curb openings is critical. Changing street grades over time and/or with street maintenance and paving imperfections commonly create high or low points that prevent flows from entering the curb opening. The optional concrete apron can help address this challenge. A common oversight during design and/or construction is exclusion of a 2-inch drop in grade between curb inlet and water entry into the basin area, causing basin materials such as rock or mulch or incoming debris to restrict the flow of water entering the basin. The most common new construction error is placing rock above the elevation of the opening. This causes a backup of water and can completely prevent water from entering the basin (see Figure 2-10 through Figure 2-15).

Figure 2-10. Curb opening Type 1, Grant Rd., Tucson, AZ

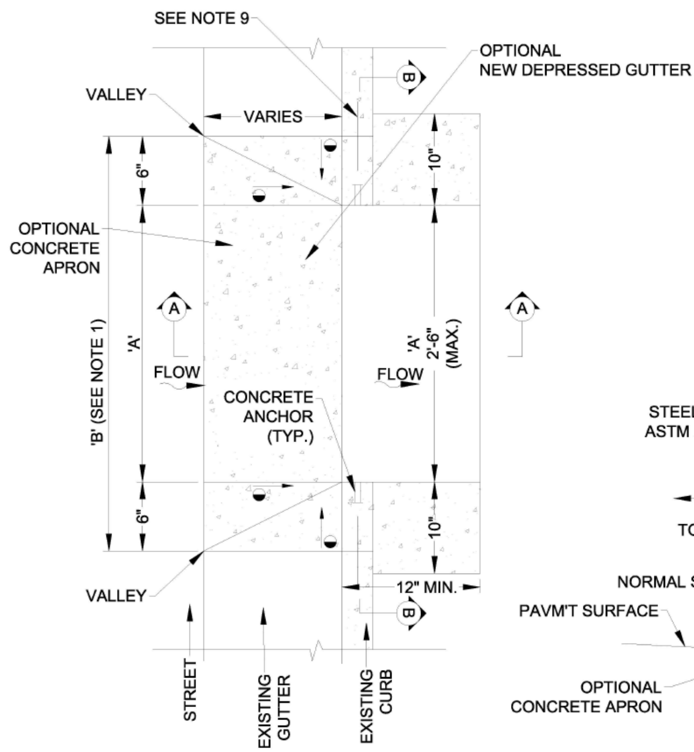
Source: Google Maps Street View.

Figure 2-11. Type 2 Curb Opening for a New Curb, Scott Ave., Tucson, AZ

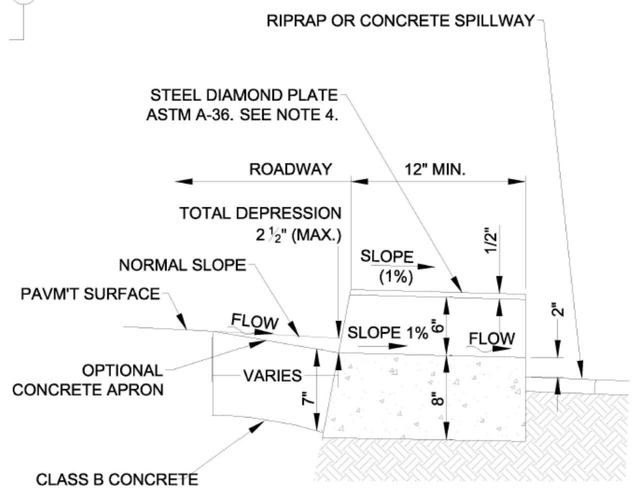
Source: Wheat Design Group.

Figure 2-12. Type 2 Curb Retrofit, Tucson Association of Realtors

Source: Watershed Management Group.



CURB OPENING - TYPE 1



TYPE 1 - SECTION A-A

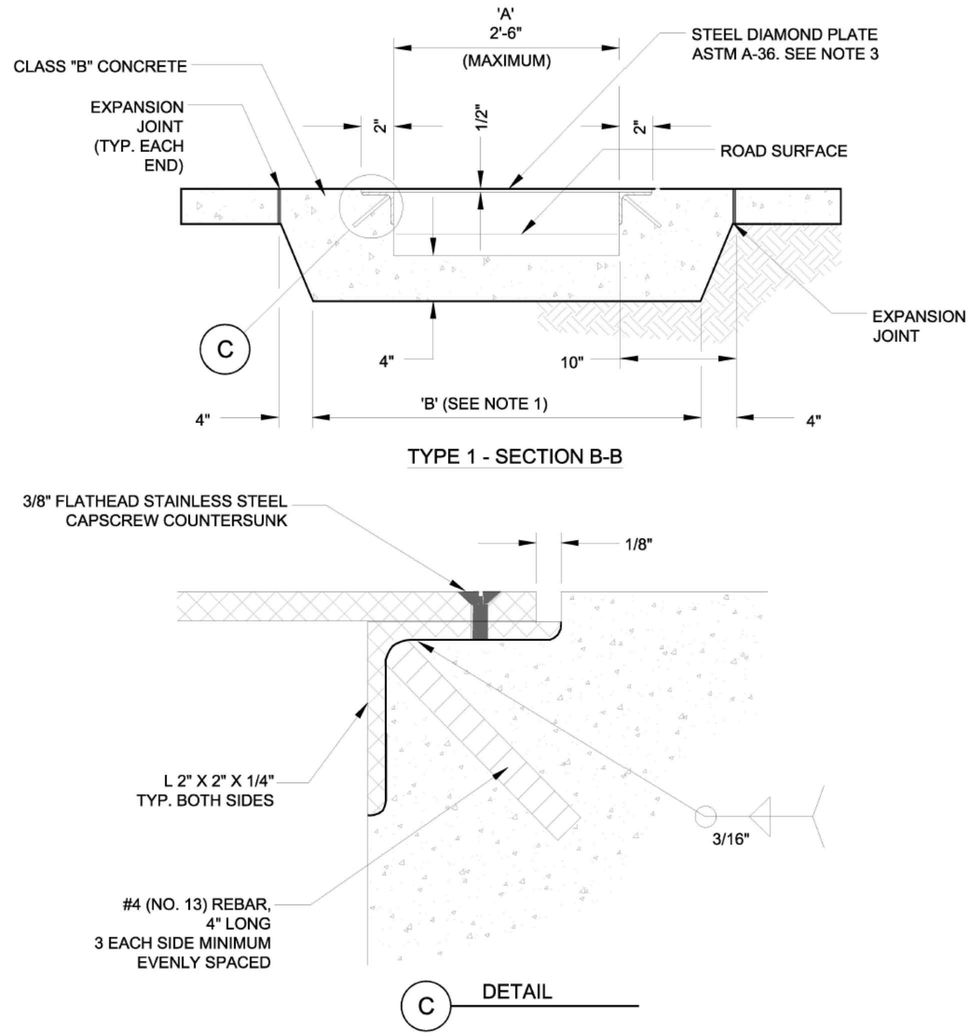
CURB OPENING - TYPE 2 - SEE SHEET 317 SHEET 3 OF 3

NOTES:

1. DIMENSION 'B' EQUALS 'A' + 1'.
2. HEADED CONCRETE ANCHORS SHALL MEET THE REQUIREMENTS OF ASTM A-108.
3. END PLATES SHALL MEET THE REQUIREMENTS OF ASTM A-36.
4. ENTIRE ASSEMBLY SHALL BE HOT DIP GALVANIZED IN ACCORDANCE WITH ASTM A-123.
5. DESIGN VERTICAL WHEEL LOAD IS 8.5 KIPS.
6. SEE MAG SPECIFICATION 340 FOR MATERIAL AND CONSTRUCTION INFORMATION.
7. REFER TO AASHTO ROADWAY DESIGN GUIDE FOR CLEAR ZONE REQUIREMENTS FOR SPEED DESIGNS.
8. GRIND EXPOSED METAL AND SEAL TO PROTECT FROM CORROSION.

PV STANDARD DETAIL	CURB OPENING	REVISED: 09/16/2024	DETAIL No. 903-1

Figure 2-13. Detail No. 903-1 Curb Opening

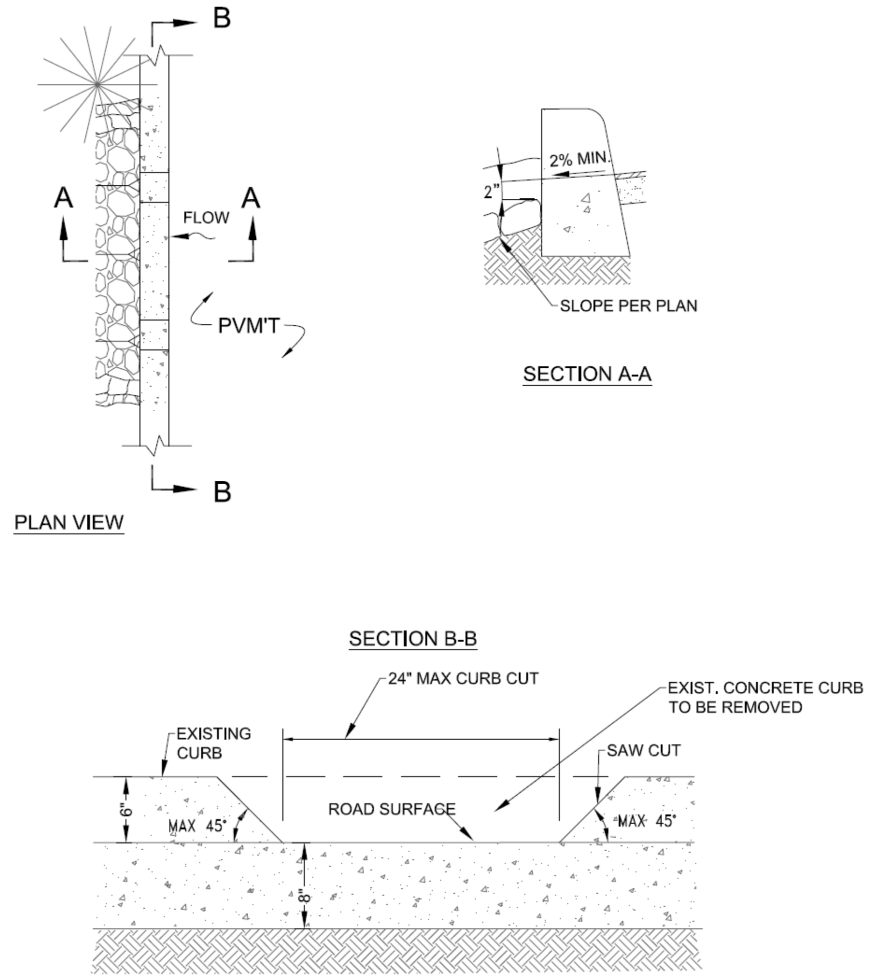


NOTES:

1. DIMENSION 'B' EQUALS 'A'+1'.
2. STEEL DIAMOND PLATE SHALL BE GIVEN ONE SHOP COAT OF PAINT NO.1 PER MAG STD. SPEC. SECTION 790.
3. EXPOSED METAL SHALL BE PAINTED WITH TWO COATS OF SSPC PAINT NO. 5 PER MAG STD. SPEC. SECTION 790.
4. STEEL DIAMOND PLATE SHALL BE DELIVERED AS ONE PIECE, CONFORMING TO THE REQUIRED SCUPPER DIMENSIONS. PLATES SHALL NOT BE SPLICED.

PV STANDARD DETAIL	CURB OPENING	REVISED: 09/16/2024	DETAIL No. 903-2

Figure 2-14. Detail No. 903-2 Curb Opening



CURB OPENING - TYPE 2

NOTES:

1. CURB CUTS MUST HAVE A MINIMUM OF 5' SEPARATION.
2. CURB CUTS MUST BE A MINIMUM OF 5' FROM DRIVEWAY APRON.
3. CURB CUTS MUST BE A MINIMUM OF 20' FROM ALL CORNERS AND & TREE PLANTING A MINIMUM OF 50' BACK OF CORNERS WITH STOP SIGNS.
4. DO NOT CUT DEEPER THAN ROAD SURFACE ELEVATION.
5. RETROFIT CURB OPENINGS SHALL BE MADE BY SAW CUT METHOD. NEW CONSTRUCTION CAN BE CAST IN PLACE.

PV STANDARD DETAIL	CURB OPENING	REVISED: 09/16/2024	DETAIL No. 903-3

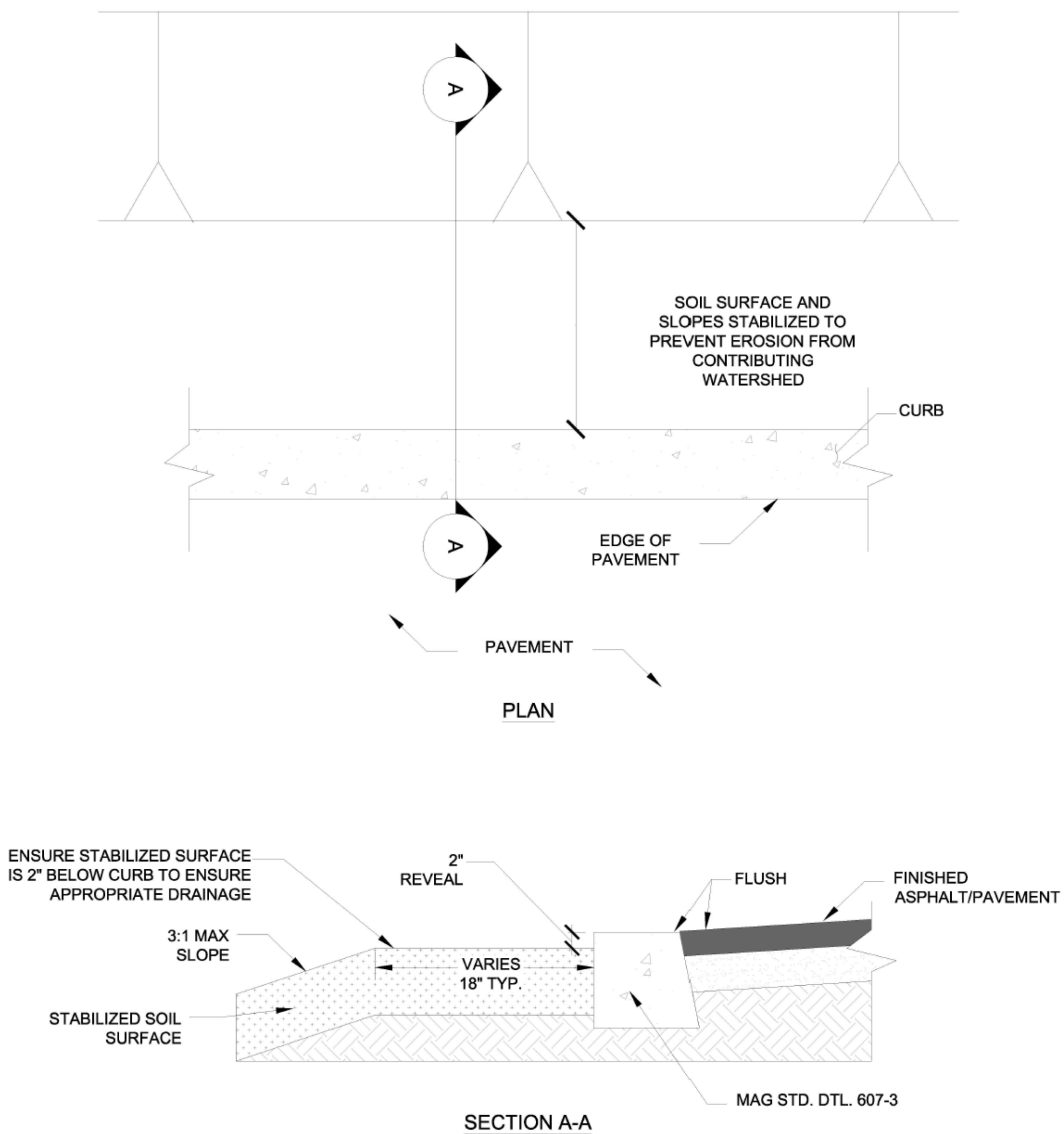
Figure 2-15. Detail No. 903-3 Curb Opening

2.1.5 Flush Curb Inlet

Distributed stormwater management can benefit greatly from flush curbs. Flush curbs can retain sheet flow conditions resulting in lower water velocity compared to the concentrated flows created by raised curbs and curb openings. A common oversight during design and/or construction is exclusion of a 2-inch drop in grade between curb inlet and water entry into the basin area, causing basin materials such as rock or mulch or incoming debris to restrict the flow of water entering the basin. The most common new construction error is placing rock above the elevation of the opening. This causes a backup of water and can completely prevent water from entering the basin (See Figure 2-16 through Figure 2-17).

Figure 2-16. Flush Curb at Edge of Asphalt Parking Lot

Source: Watershed Management Group.

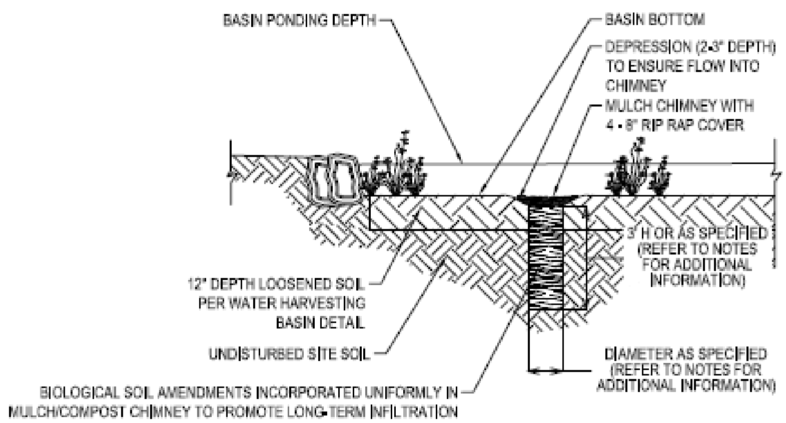


PV STANDARD DETAIL	FLUSH CURB INLET	REVISED: 09/16/2024	DETAIL No. 904

Figure 2-17. Detail No. 904 Flush Curb Inlet

2.1.6 Compost Chimney

By combining the benefits of organic wood mulch and the biodiversity of BSAs, the Compost Chimney is an ideal addition to a retention area to enhance capacity without expanding the basin footprint or ponding depth. The mulch will provide the needed substrate for the biology in the BSAs to establish within the column of the chimney to promote long-term infiltration while also providing an irrigation benefit to surrounding vegetation. BSAs are critical to ensuring plant health by creating new beneficial relationships between the water, soil, and minerals not possible in urban soils devoid of biology as a result of compaction and disturbances from human activities. The number of chimneys and depth of each chimney can be determined by design volumes, infiltration rates, and limiting soil layers or bedrock, and project budget (see Figure 2-18).



CONTRACTOR NOTES:
1. MULCH CHIMNEY(S) SHALL BE PLACED IN LOCATIONS SHOWN ON PROJECT PLANS.
2. CHIMNEY DEPTH AND DIAMETER AS SHOWN ON THE PROJECT PLANS.
3. ADJACENT GRADE SHALL SLOPE TO THE CHIMNEY AS SHOWN IN THE DETAIL, TOP OF MULCH SHALL BE RECESSED AS SHOWN SO AS NOT TO INHIBIT FLOW INTO CHIMNEY.

DESIGNER NOTES:
1. A MULCH CHIMNEY ACTS AS A CARBON SPONGE, ENRICHING THE SOIL, IMPROVING MOISTURE RETENTION, AND ASSISTING IN WATER INFILTRATION IN POORLY DRAINING SOILS. THE CHIMNEY IS PLACED IN THE LOWEST QUADRANT OF THE WATER HARVESTING BASIN AND FILLED WITH A COMBINATION OF POROUS ORGANIC MATERIAL SUCH AS MULCH AND COMPOST (CHIMNEY MATERIAL IS SPECIFIED ON PROJECT PLANS).
2. PLANTS ARE PLACED AT LEAST 6" FROM THE MULCH CHIMNEY. THE GOAL IS FOR PLANT ROOTS TO BE ABLE TO ACCESS MOISTURE AND ENRICHED SOIL.
3. CHIMNEY DEPTH AND DIAMETER: CHIMNEYS SHALL BE SIZED BASED ON PROJECT NEEDS. DEPTH SHALL BE DEEP ENOUGH TO PENETRATE COMPACTION LAYERS (SUCH AS CALICHE) OR TO ACHIEVE BASIN DRAINAGE GOALS. TYPICAL SUGGESTED MINIMUM DIAMETER IS 24", THIS CAN BE ADJUSTED BASED ON AVAILABLE AUGER SIZE, BUT SHALL BE NO LESS THAN 12". A MULCH TRENCH (ALLOWING FOR GREATER INFILTRATION AREA) CAN BE CONSTRUCTED USING THE SAME PRINCIPLES.
4. OPTIONAL COVER (SPECIFIED ON PROJECT PLANS IF INCLUDED): A METAL GRATE (THAT WILL ALLOW FOR INFLOW OF WATER) OR BOULDERS MAY BE PLACED ACROSS THE TOP OF THE CHIMNEY IF THERE IS CONCERN WITH ORGANIC MATTER SETTLING AND A FALL HAZARD RESULTING. UNDER NORMAL CONDITIONS AS THE CARBON SPONGE DECOMPOSES NEW MATERIAL WILL NATURALLY ACCUMULATE WITHIN THE CHIMNEY VOID, ADDITIONAL ORGANIC MATTER MAY ALSO BE ADDED DURING ROUTINE MAINTENANCE.

PV STANDARD DETAIL	COMPOST CHIMNEY	REVISED: 09/26/2024	DETAIL No. 905

Figure 2-18. Detail No. 905 Compost Chimney

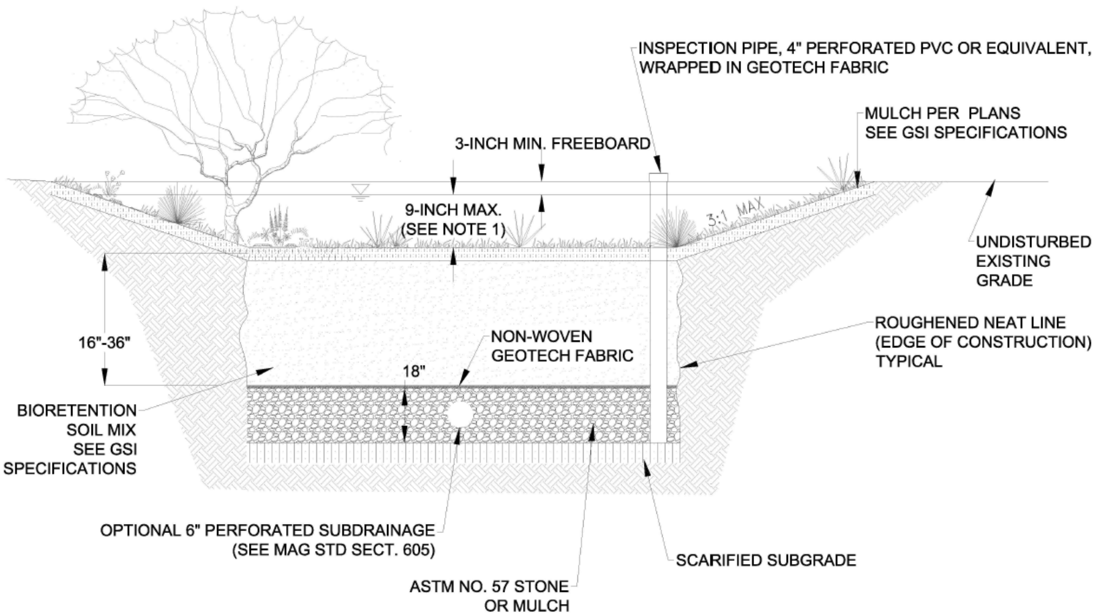
2.1.7 Bioretention Basin

Successful bioretention depends on having sufficient soil biological activity to create the beneficial relationships between the soil food web and vegetation. This is especially true for a bioretention basin intended to provide a water quality benefit with high infiltration rates. Without attention to soil health, focusing exclusively on water quality and high infiltration will harm vegetation, especially in the face of drought and/or extreme temperatures (see Figure 2-19 through Figure 2-20).

The GSI Specification **Bioretention Soil Mix** is designed to address this issue. This constructed soil medium consists of coarse sand (not more than 15% American Society for Testing and Materials (ASTM) C-33 sand by volume); organic mature compost (15-25% leaf, worm, or equivalent) mixed in a drum mixer; and excavated and/or topsoil (70%) per Maricopa Association of Governments (MAG) Section 795.2.

Figure 2-19. Shallow Residential Bioretention Basin

Source: EcoSense Sustainable Landscapes.



GREEN INFRASTRUCTURE NOTES:

1. PONDED WATER MUST INFILTRATE WITHIN 12 HOURS.
2. SEE SITE GUIDE FOR SETBACKS.
3. UNDERDRAIN OR SURFACE OVERFLOW REQUIRED WHEN CONTRIBUTING AREA RUNOFF VOLUME EXCEEDS STORAGE VOLUME.
4. DECOMPOSED GRANITE MAY NOT BE USED IN AREAS SURROUNDING BASIN.

CONSTRUCTION NOTES:

1. CONSTRUCT BASIN AFTER FINISH GRADING OF SURROUNDING SITE.
2. NO EQUIPMENT TRAFFIC ON THE BASIN.

PV STANDARD DETAIL	BIORETENTION BASIN	REVISED: 09/16/2024	DETAIL No. 906

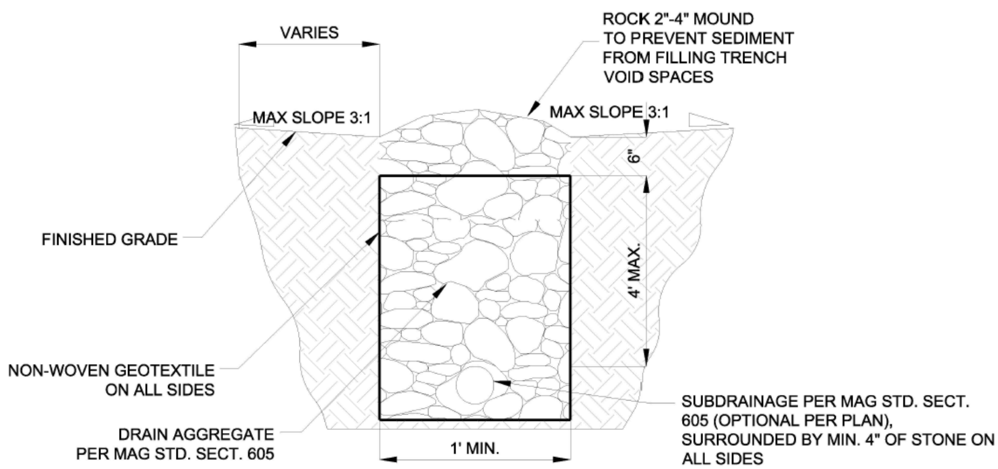
Figure 2-20. Detail No. 906 Bioretention Basin

2.1.8 Infiltration Trench

Infiltration trenches apply to a variety of contexts, whether used on their own or in combination with such elements as stormwater harvesting basins, compost chimney, in-street features, and vegetated swales. Infiltration trenches should only be used in watersheds with sediment-free runoff and/or where sediment management practices like sediment traps, check dams, One Rock Dams, Zuni bowls, or sheet flow spreaders provide sediment management benefits upstream of infiltration trenches. Ensure regular inspection and removal of surface sediment build up (see Figure 2-21 and Figure 2-22).

Figure 2-21. Infiltration Trench

Source: Wheat Design Group.



GREEN INFRASTRUCTURE NOTES:

1. SEE SITE LAYOUT GUIDE FOR SETBACKS.
2. UNDERDRAIN OR SURFACE OVERFLOW REQUIRED WHEN CONTRIBUTING AREA RUNOFF VOLUME EXCEEDS STORAGE VOLUME.
3. DECOMPOSED GRANITE MAY NOT BE USED IN AREAS SURROUNDING THE TRENCH, VEGETATION IS ENCOURAGED ADJACENT TO THE TRENCH. NO VEGETATION TO BE PLANTED WITHIN THE TRENCH.
4. FOR USE IN WATERSHEDS WITH LOW SEDIMENT RUN-OFF.
5. STORMWATER HARVESTING BASINS, SEDIMENT TRAPS, ONE ROCK DAMS, CHECK DAMS, AND ZUNI BOWLS CAN BE USED UPSTREAM TO MITIGATE SEDIMENT.

CONSTRUCTION NOTES:

1. CONSTRUCT THE TRENCH AFTER FINISH GRADING OF SURROUNDING SITE.
2. NO EQUIPMENT TRAFFIC ON THE TRENCH.

PV STANDARD DETAIL	INFILTRATION TRENCH	REVISED: 09/16/2024	DETAIL No. 907

Figure 2-22. Detail No. 907 Infiltration Trench

2.1.9 Domed Overflow Structure

Domed overflow structures capture runoff volumes greater than the designed capacity and direct them to a downstream facility, storm drain, basin, swale, or natural wash (see Figure 2-23 through Figure 2-25). Infiltration rates and design goals for retention/detention determine an appropriate dome elevation. Domes help in watershed conditions with a greater likelihood of floatable debris. For watersheds with minimal trash and floatable debris, a Type "E" Catch Basin, MAG Detail 534, may be appropriate. Figure 2-23 shows a dome at ground level without any resulting basin retention. Raising the dome with a riser creates more retention/infiltration opportunities while minimizing long-term maintenance.

Figure 2-23. Dome Overflow Grate Flush With the Surface

Source: Aaron Volkening

Figure 2-24. Dome Overflow With Riser

Source: City of Lake Oswego, D Avenue Improvement Project.

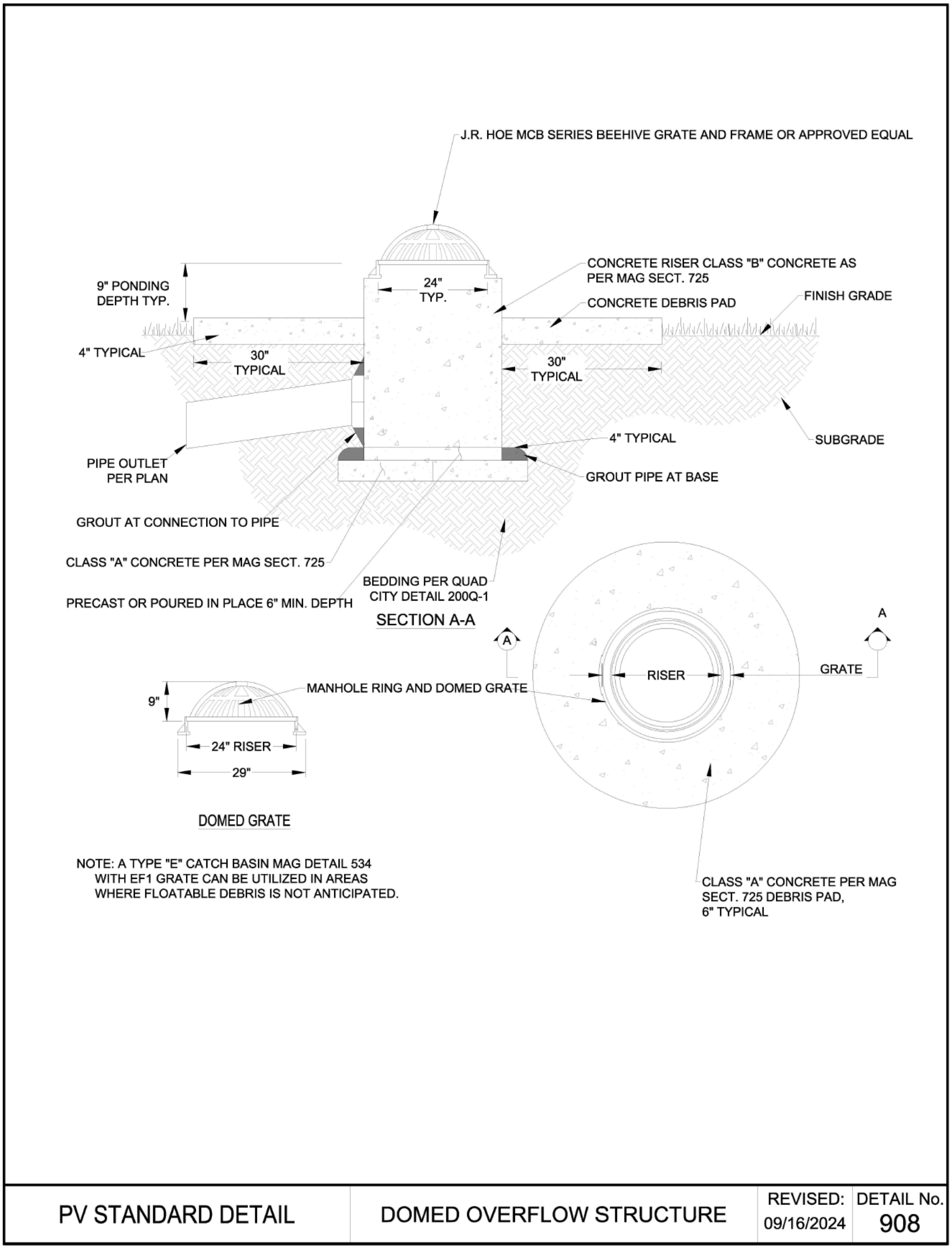


Figure 2-25. Detail No. 908 Domed Overflow Structure

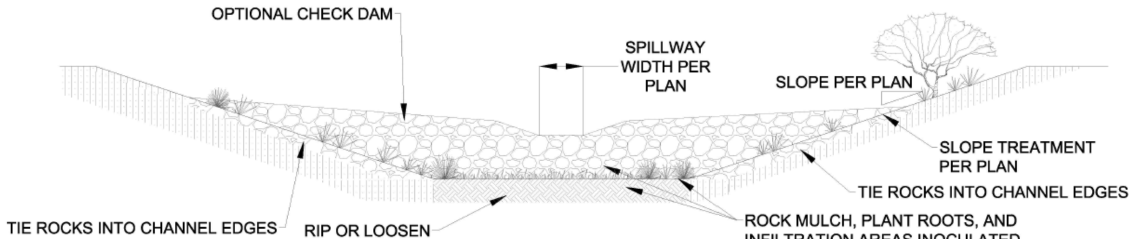
2.2 Conveyance Practices

2.2.1 Vegetated Swale

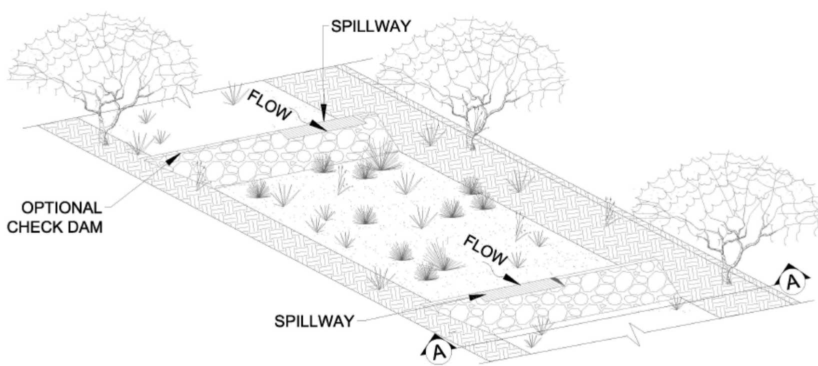
A swale provides conveyance of flows while minimizing erosion with native vegetation and rock (see Figure 2-26 and Figure 2-27). Most residential applications do not require rock unless velocities and/or slope are greater than 3 feet/sec. Where possible utilize living vegetation such as native bunch grasses and trees along with BSAs to stabilize soils. Consult an engineer to determine the necessity of rock and its appropriate sizing for expected high-flow conditions.

Figure 2-26. Vegetated Swale Near Glendale Arizona Public Library Entrance

Source: Joanne Toms, Glendale Water Conservation.



SECTION A-A



SWALE ISOMETRIC VIEW

GREEN INFRASTRUCTURE NOTES:

1. USE NATIVE GRASSES TO STABILIZE SOIL IN SWALE BOTTOM.
2. SPILLWAYS SHALL BE OFFSET FROM ONE ANOTHER.
3. WHEN DESIGN VELOCITY EXCEEDS 3 FEET/SEC., UTILIZAE RIP RAP SIZED PER DRAINAGE REPORT.

PV STANDARD DETAIL	VEGETATED SWALE	REVISED: 09/16/2024	DETAIL No. 909

Figure 2-27 Detail No. 909 Vegetated Swale

2.2.2 Check Dam

Check dams slow surface runoff to reduce erosion (see Figure 2-28 and Figure 2-29). Alternating low-flow notches in check dams with native bunch grasses on the downstream side provides meandering flows while protecting soils and basin edges from erosion.

Check dams have some potential drawbacks, including their difficulty of construction relative to smaller One Rock Dams, their potential for failure, and their potential to increase erosion and flood risks. Ensure check dams direct the force of water away from check dam edges/swale banks by raising rock that interfaces with the soil banks above the main level of the check dam. If this is not done, water will cut around the edges of dams.

Figure 2-28. Saint Mary's Road Concrete Check Dam

Source: Wheat Design Group.

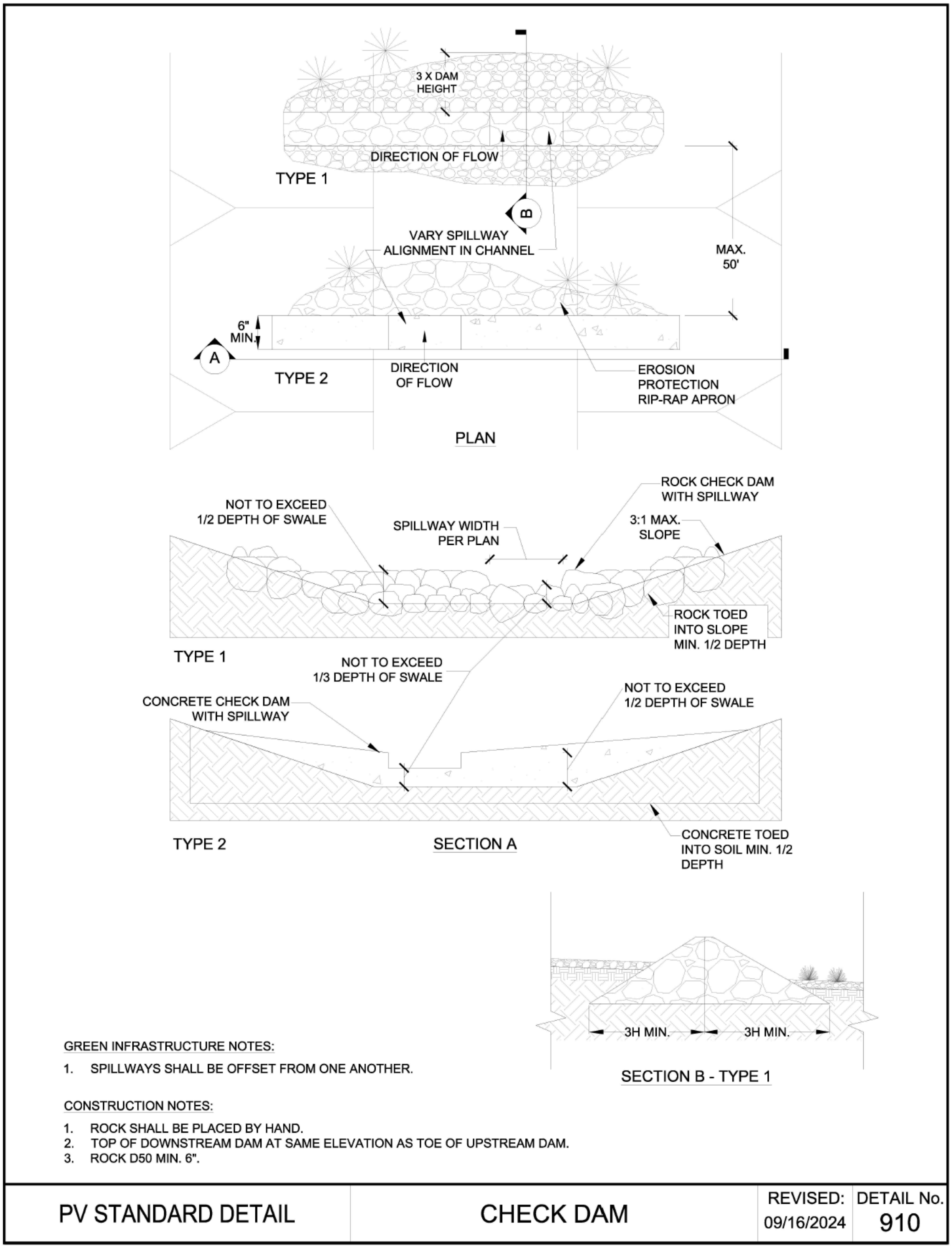


Figure 2-29. Detail No. 910 Check Dam

2.2.3 One Rock Dam

One Rock Dams are an important tool for erosion control, especially as an alternative to a check dam (see Figure 2-30 through Figure 2-33). In a wash or erosive flow path, a series of well-built One Rock Dams can be much more effective than difficult-to-construct and failure-prone check dams.

To construct a One Rock Dam, tie rock into the adjacent banks and secure the low flow rock below the channel bed as shown in Quivira Coalition's conceptual diagram shown in Figure 2-32 and Figure 2-33 to create sufficient strength to resist the expected scour/erosion forces. Size rock for the largest expected flows to ensure features remain in place. This will minimize erosion in all expected design conditions. Typical D50 ranges from 8" in small, shallow sloped watersheds to 18-24"+ in larger and/or steeper sloped watersheds.

Figure 2-30. One Rock Dam

Source: Holistic Engineering and Land Management.

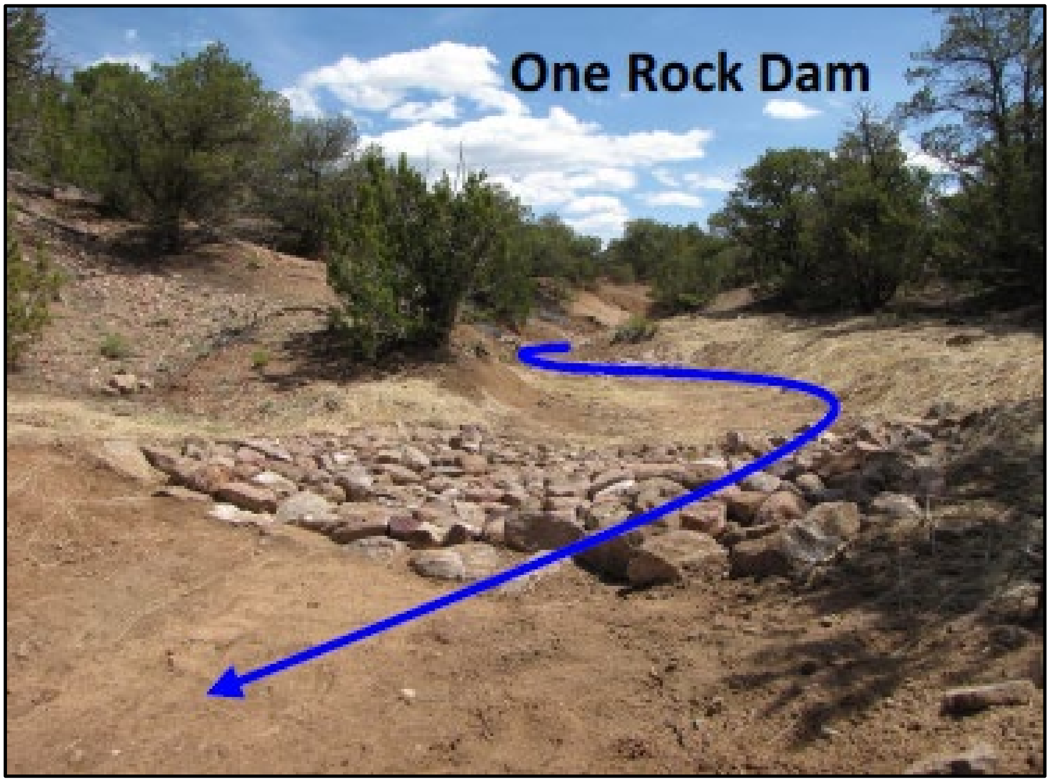


Figure 2-31. One Rock Dam Showing Flow Path

Source: Quivira Coalition.

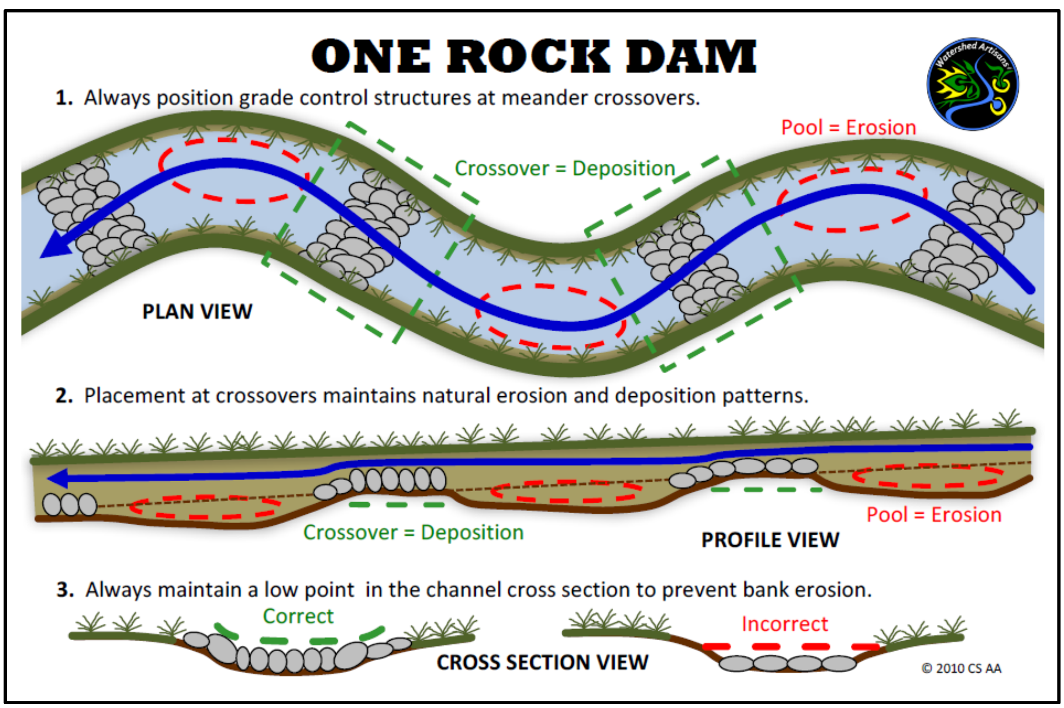
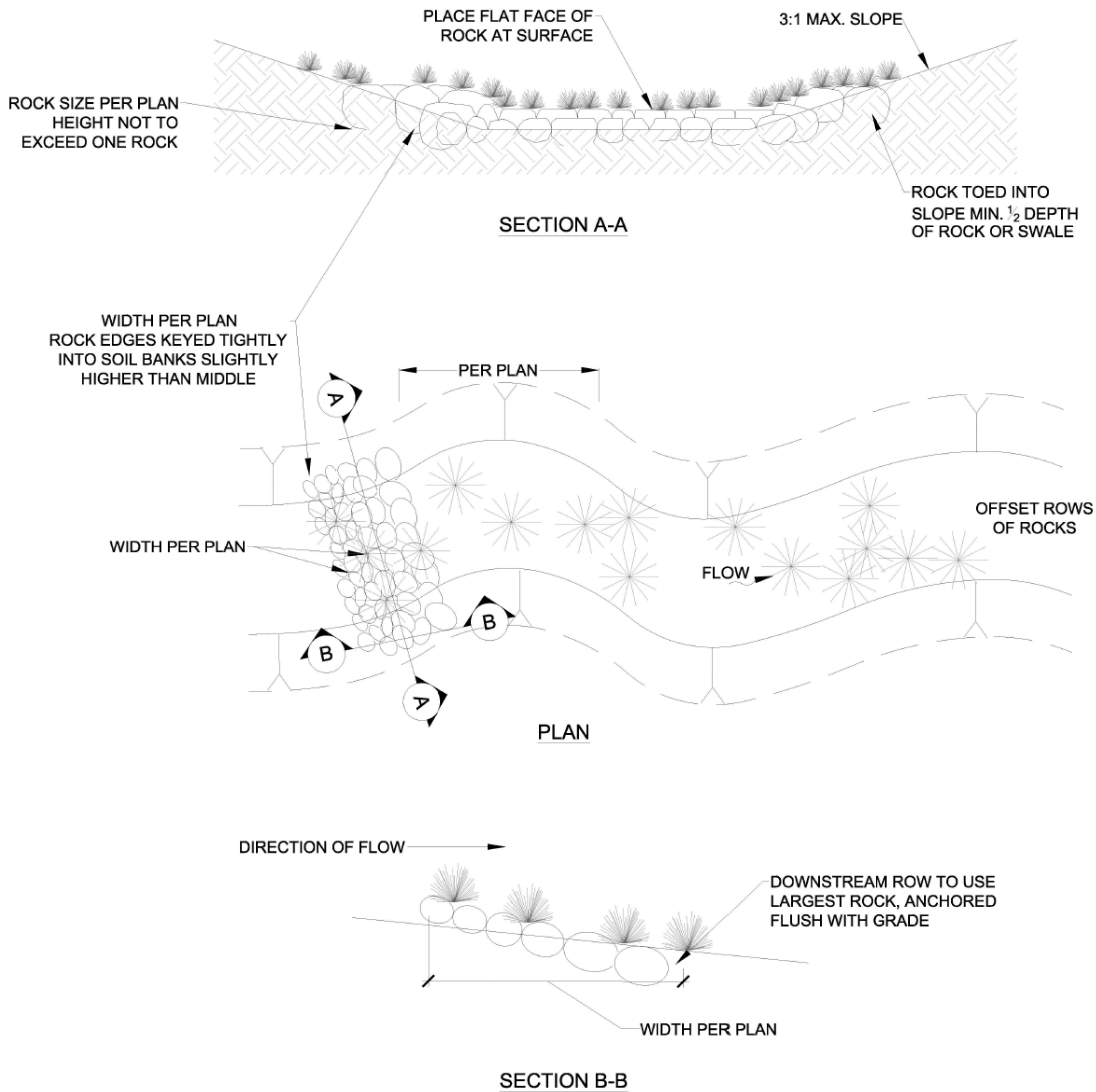


Figure 2-32. Diagram of a One Rock Dam

Source: Quivira Coalition.



GREEN INFRASTRUCTURE NOTES:

1. DAMS NOT TO BE PLACED ON BENDS.
2. SEED MIX PER LANDSCAPE PLAN UNDER ROCK.

CONSTRUCTION NOTES:

1. ALL ROCK HAND-PLACED.
2. ROCK D50 MIN.6".

PV STANDARD DETAIL	ONE ROCK DAM	REVISED: 09/16/2024	DETAIL No. 911

Figure 2-33. Detail No. 911 One Rock Dam

2.2.4 Zuni Bowl

Used in washes/channels or in the uplands, Zuni bowls can dissipate energy, manage sediment, and stop erosion and head cutting (see Figure 2-34). Always use experienced, skilled labor to effectively place rock to resist the forces of flows and to ensure large rocks key in at the bottom of the bowl and at the edges of the banks where erosive forces will be greatest.

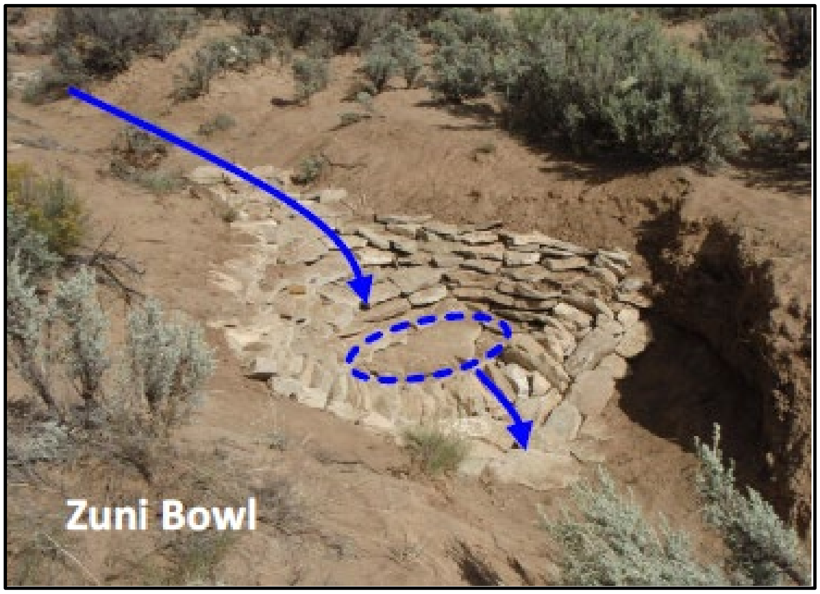


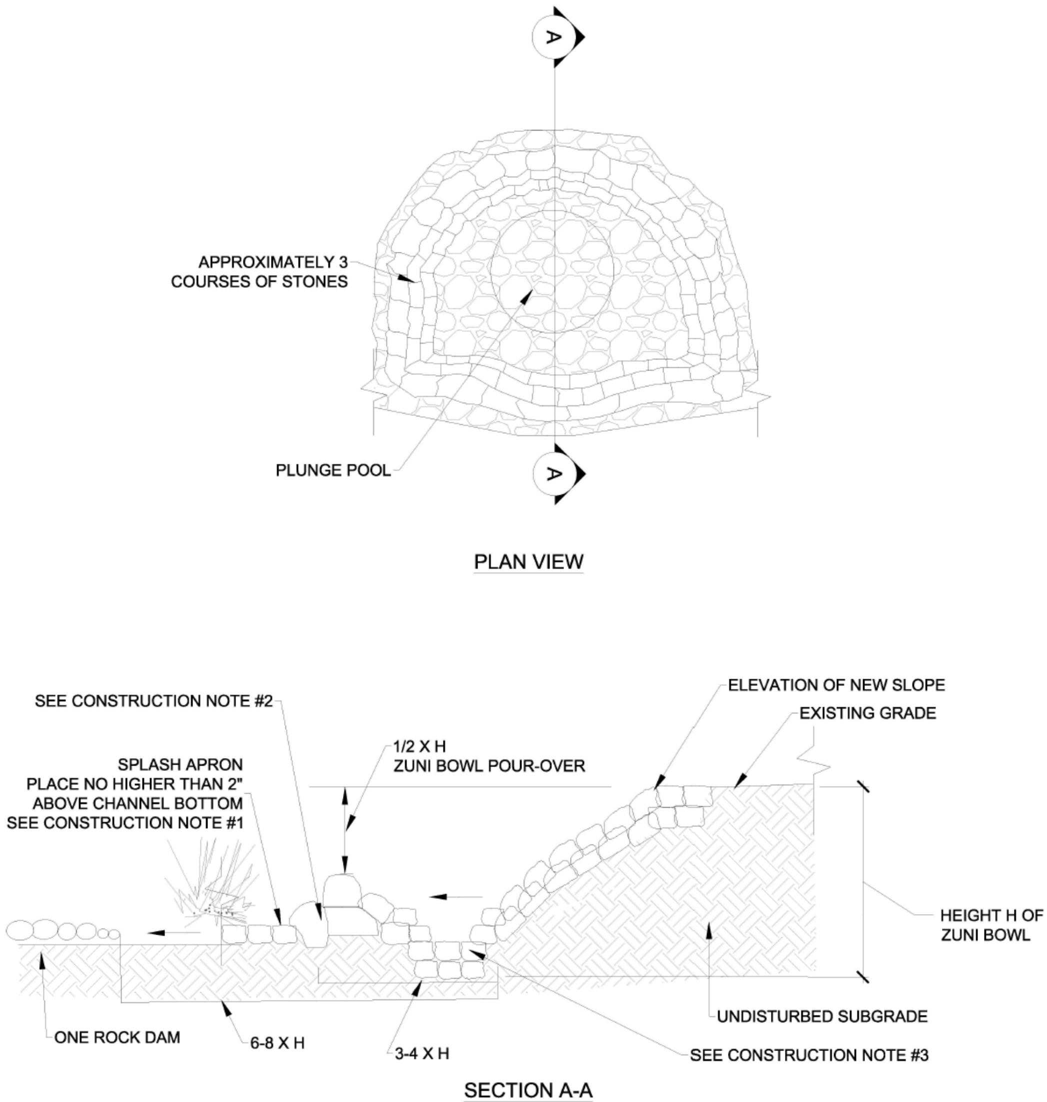
Figure 2-34. Photo of Zuni Bowl Showing the Flow Path

Photo Credit: Quivira Coalition.

Figure 2-35 shows a Zuni bowl during and after construction, respectively. The design guides water in three ways: down the creek over the grade control and into both the right and left diversion basins. This is a Rosgen cross-vane with an "A" arm and a Zuni bowl (see Natural Resources Conservation Service (NRCS) *Part 654 Stream Restoration Design National Engineering Handbook* Chapter 11 Rosgen Geomorphic Channel Design 11–59). The cross vanes retain water, irrigating new vegetation while also protecting the edges of the Zuni bowl. The design defines a riffle-run-pool-glide sequence in the creek (Van Clothier, Stream Dynamics, Inc. streamdynamics.us/blog-entry/san-vicente-creek-project-action).

Figure 2-35. During- and Post-Construction of a Large Zuni Bowl

Source: Stream Dynamics, Inc.



CONSTRUCTION NOTES:

1. PLACE SPLASH APRON ROCKS FIRST.
2. SECOND ROW OF ROCKS AS LARGE AS POSSIBLE.
3. ADD PLUNGE POOL ARMOR AT UNIFORM HEIGHT.
4. LAY ROCK COURSES UPSTREAM FROM PLUNGE POOL.
5. ROCK D50 MIN. 10".

GREEN INFRASTRUCTURE NOTES:

1. THIS PRACTICE IS INTENDED FOR NATURAL DRAINAGES AND BENEFITS RESTORATION PROJECTS FOR STREAM HEADCUTS.
2. NATURAL STONES FROM THE PROJECT AREA SHOULD BE UTILIZED.
3. PLACE NATIVE SEED BEFORE ROCK INSTALLATION.

PV STANDARD DETAIL	ZUNI BOWL	REVISED: 09/16/2024	DETAIL No. 912

Figure 2-36. Detail No. 912 Zuni Bowl

2.2.5 Sheet Flow Spreader

A sheet flow spreader, also called a Media Luna, creates sheet flow from concentrated flows (see Figure 2-37 through Figure 2-39). To build a strong structure capable of resisting the forces of runoff, ensure that the most downstream anchor rock is fully buried to be flush with the surface of the adjacent grade.

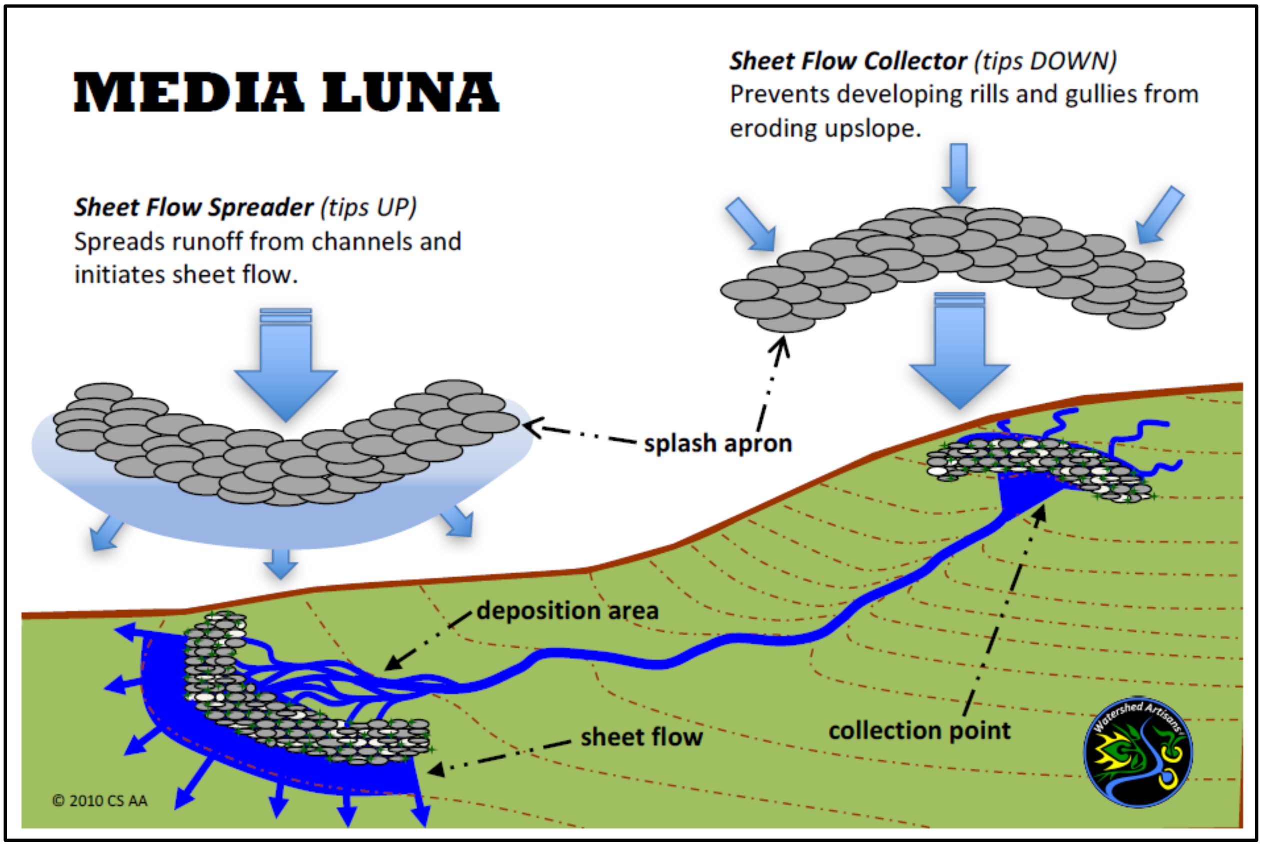


Figure 2-37. Diagram of Media Luna

Source: Quivira Coalition.

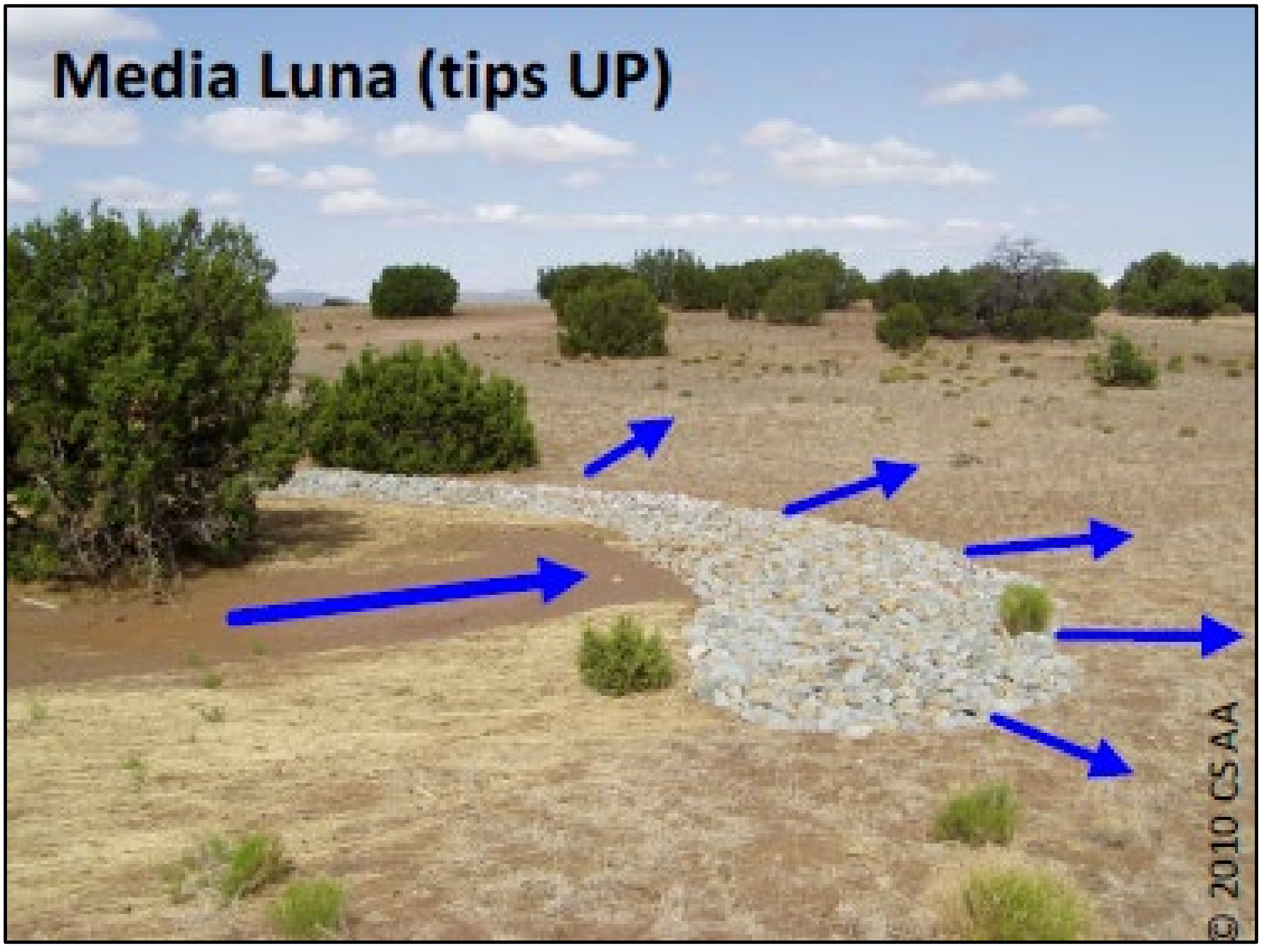
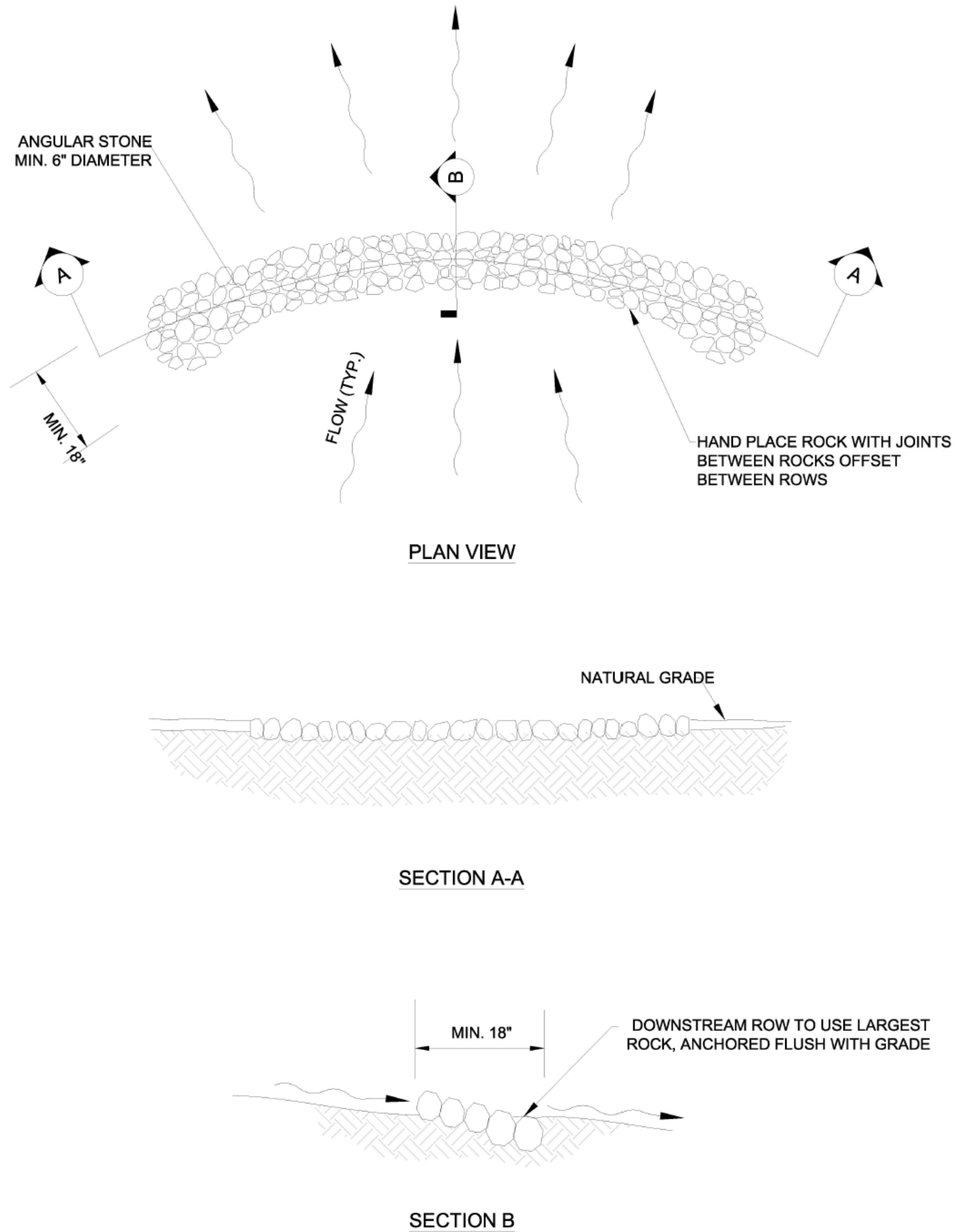


Figure 2-38. Photograph of Media Luna/Sheet Flow Spreader That Tips Up

Source: Quivira Coalition.



NOTES:

1. LENGTH AND WIDTH PER PLAN.
2. SHAPE OF SPREADER "ON CONTOUR" WITH EDGES SHAPED UPSLOPE TO PREVENT WATER MIGRATING AROUND.

PV STANDARD DETAIL	SHEET FLOW SPREADER	REVISED: 09/16/2024	DETAIL No. 913

Figure 2-39. Detail No. 913 Sheet Flow Spreader

2.3 Active Practices

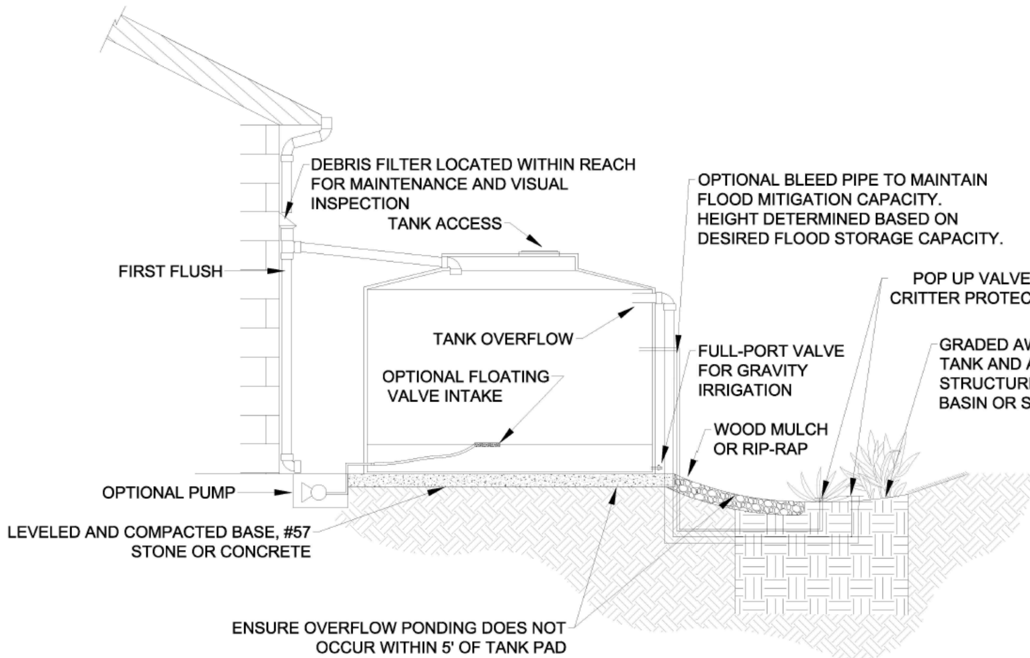
2.3.1 Cistern

During dry periods in arid regions, cisterns provide important water storage for outdoor water use and indoor uses (if treated appropriately) (see Figure 2-40 and Figure 2-41). Typical indoor rainwater treatment systems use a combination of sand, activated carbon, and ultraviolet processes. When installing a cistern, follow these guidelines to ensure optimal system function:

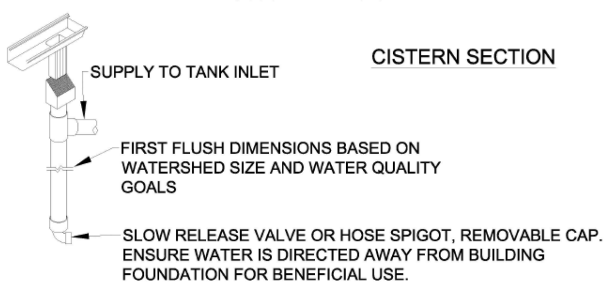
- Design first flush systems and debris filters to be self-cleaning and easily accessible.
- Remove internal debris screens included in any plumbing or tank fitting. Any screens the homeowner cannot see will eventually clog and create a flood risk.
- Locate tank overflow popup at least 10 feet away from any structures and cisterns.
- Maintain existing drainage patterns with overflow location.
- Maximize benefits of tank overflow by directing flows to basins where possible.
- If flood risk mitigation is a primary goal, install a bleed off pipe to ensure availability of a minimum flood storage capacity when back-to-back storms occur.

Figure 2-40. Metal Cistern

Source: Southern Arizona Rain Gutters.



CISTERN SECTION



FIRST FLUSH DETAIL

NOTES:

1. PUMP TO IRRIGATION AND/OR INDOOR WATER FILTRATION SYSTEM PER INTERNATIONAL BUILDING CODE SECTION 611.
2. CISTERN SETBACK 2' MIN. FROM ADJACENT STRUCTURES.
3. A BRAINBOX OR OTHER AUTOMATED MAKE-UP WATER SUPPLY CAN BE USED TO REFILL CISTERN WHEN RAINWATER SUPPLY IS EXHAUSTED.
4. BACKFLOW PREVENTION DEVICE REQUIRED FOR A PUMP SYSTEM.
5. FIRST FLUSH OPTIONAL WHEN WATER USED FOR IRRIGATION.

PV STANDARD DETAIL	CISTERN	REVISED: 09/16/2024	DETAIL No. 914

Figure 2-41. Detail No. 914 Cistern

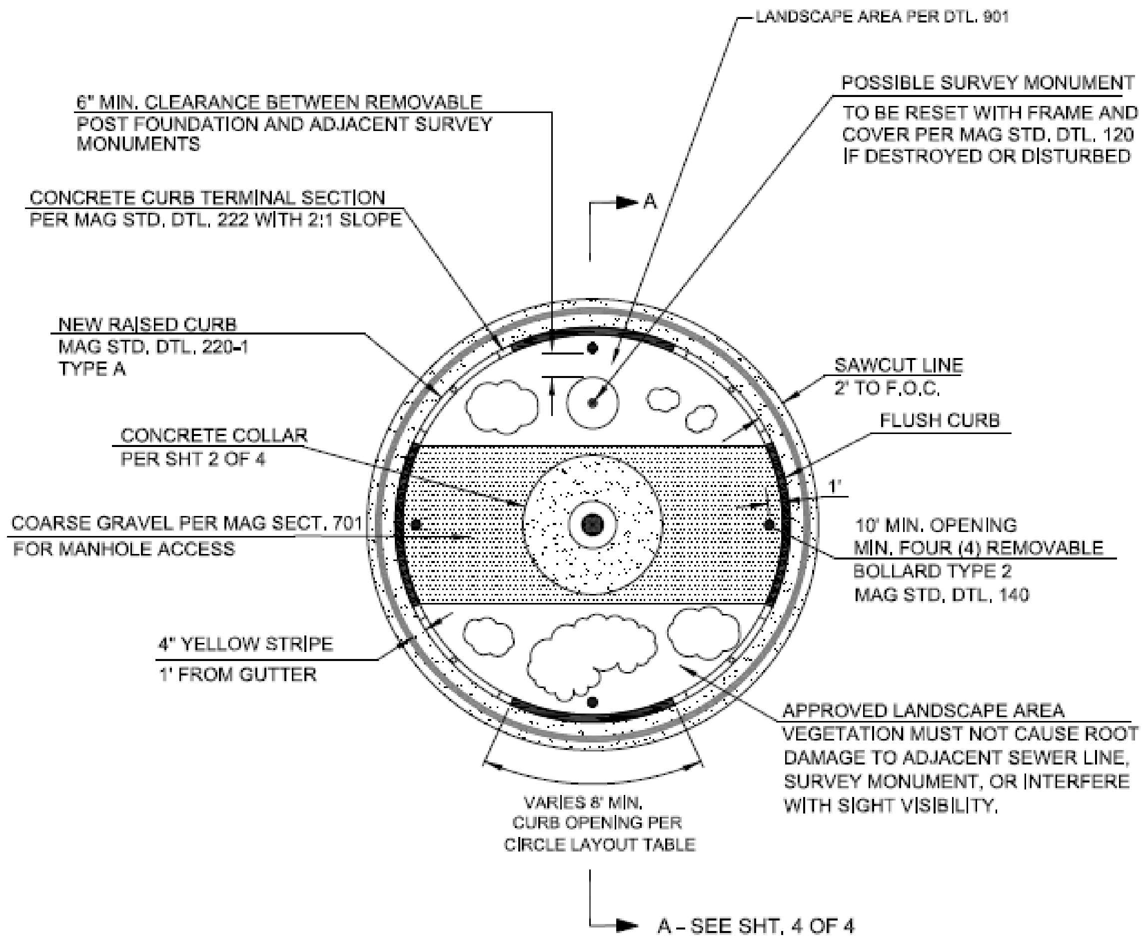
2.4 In-Street Practices

2.4.1 Traffic Circle

Traffic circles with stormwater harvesting basins in the planting areas can provide important traffic calming and nuisance flooding benefits (see Figure 2-42 through Figure 2-46). Many utilities located in roads create potential conflicts with traffic circle plantings. All designs and plantings must comply with local guidance for utility offsets and setbacks. Plant locations must also comply with appropriate safety and visibility requirements.

Figure 2-42. Newly Planted Traffic Circle

Source: Tucson Clean and Beautiful.

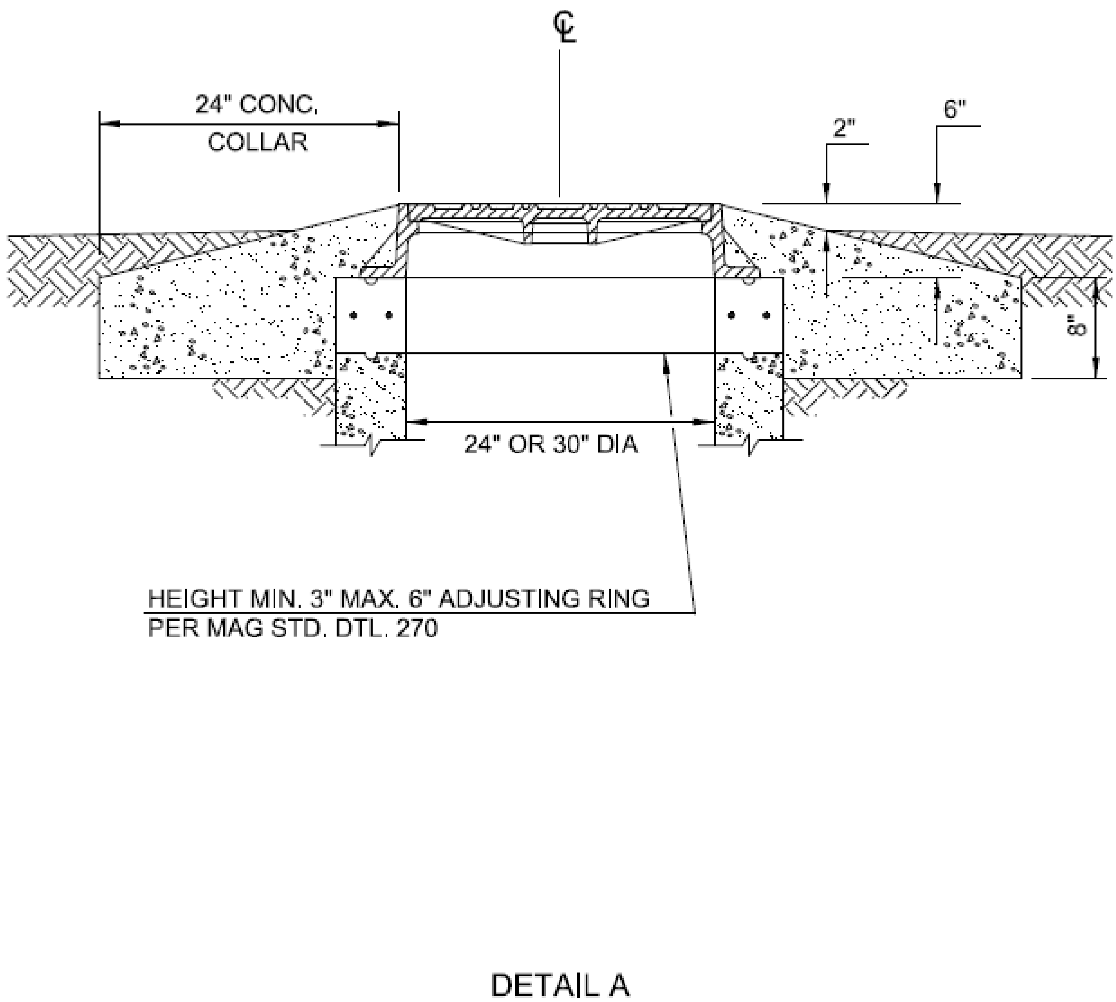


NOTES:

1. LANDSCAPING TO BE DETERMINED BY PROJECT ENGINEER AND NEIGHBORHOOD IN ACCORDANCE WITH THIS DETAIL.
2. TRAFFIC CIRCLE NOT TO BE USED IN FLOOD PRONE AREAS OR INTERSECTIONS WITH INVERTED ROADS.
3. TRAFFIC CIRCLE CANNOT BE AT CONSECUTIVE INTERSECTIONS TO ALLOW FOR EASE OF USE FOR COMMERCIAL VEHICLES.

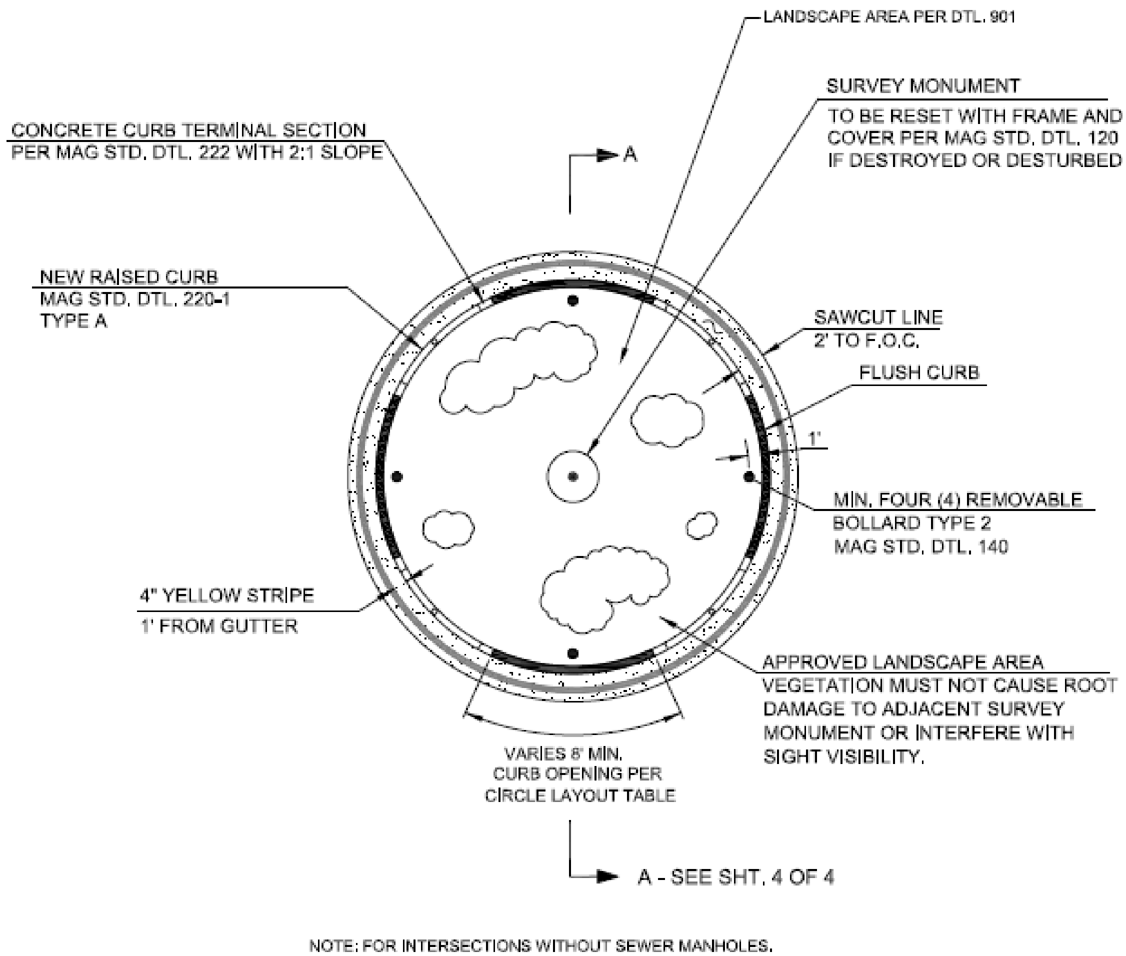
PV STANDARD DETAIL	TRAFFIC CIRCLE	REVISED: 09/26/2024	DETAIL No. 915-1

Figure 2-43. Detail No. 915-1 Traffic Circle



PV STANDARD DETAIL	TRAFFIC CIRCLE	REVISED: 09/26/2024	DETAIL No. 915-2

Figure 2-44. Detail No. 915-2 Traffic Circle



NOTES:

1. LANDSCAPING TO BE DETERMINED BY PROJECT ENGINEER AND NEIGHBORHOOD IN ACCORDANCE WITH THIS DETAIL.
2. TRAFFIC CIRCLE NOT TO BE USED IN FLOOD PRONE AREAS OR INTERSECTIONS WITH INVERTED ROADS.
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PV STANDARD DETAIL	TRAFFIC CIRCLE	REVISED: 09/26/2024	DETAIL No. 915-3

Figure 2-45. Detail No. 915-3 Traffic Circle

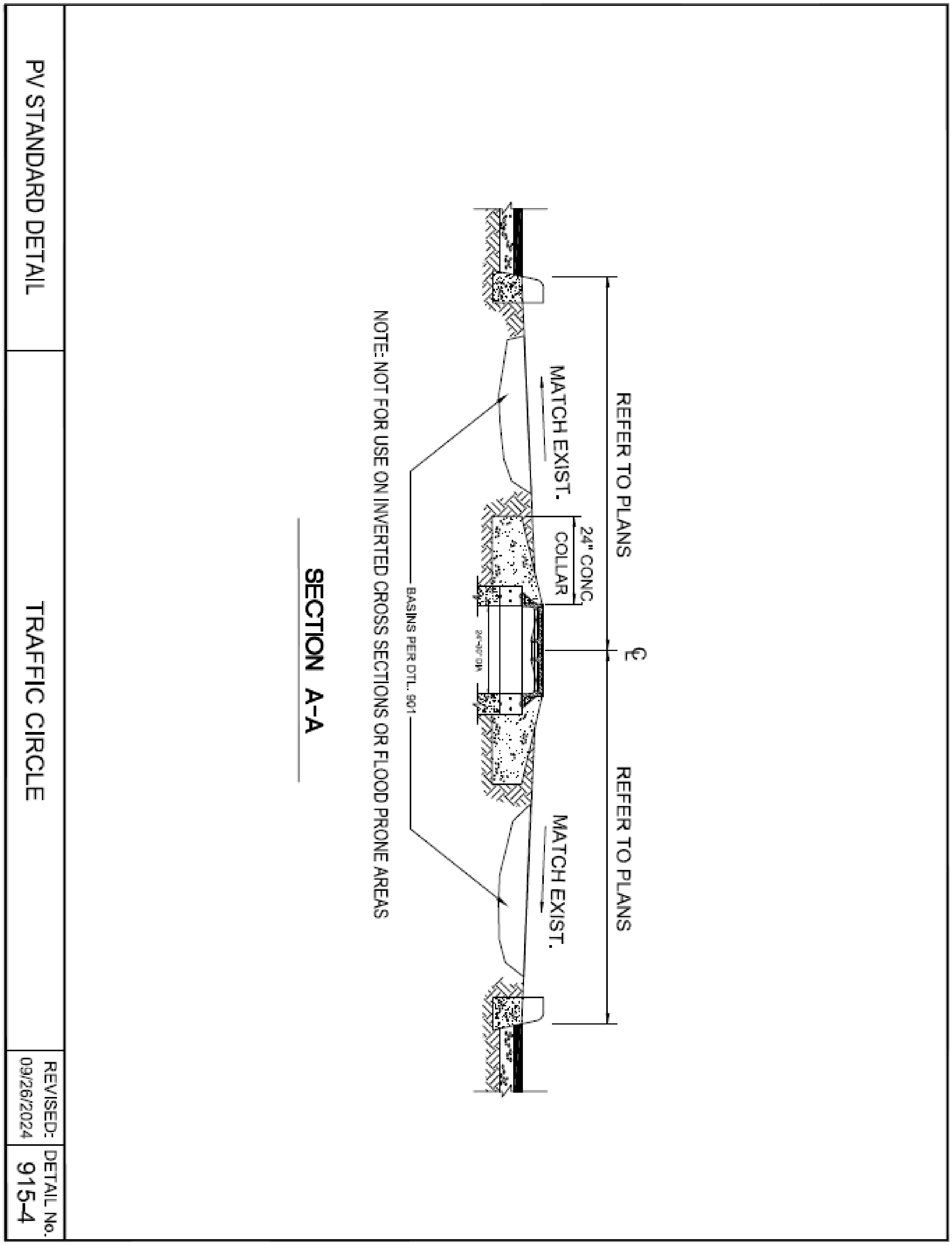


Figure 2-46. Detail No. 915-4 Traffic Circle

2.4.2 Chicane

Streets represent a significant source of runoff. They also add thermal heat, contributing to the urban heat island. Chicanes mitigate these negative impacts by reducing hardscapes and creating cooling shade (see Figure 2-47 and Figure 2-49). A chicane can also be used with sediment traps, compost chimneys, bioretention, infiltration trench, and check dams. Chicanes are applicable for more constrained spaces and/or where traffic calming by meandering the flow of traffic is desired. For larger in-street features use the Street Width Reduction Detail. If using domed overflows for conveyance, ensure low flows are retained and/or will flow downstream. This maximizes the irrigation benefit while minimizing maintenance by effectively managing sediment and debris.

Figure 2-47. Newly Planted Chicane

Source: Tucson Clean and Beautiful.

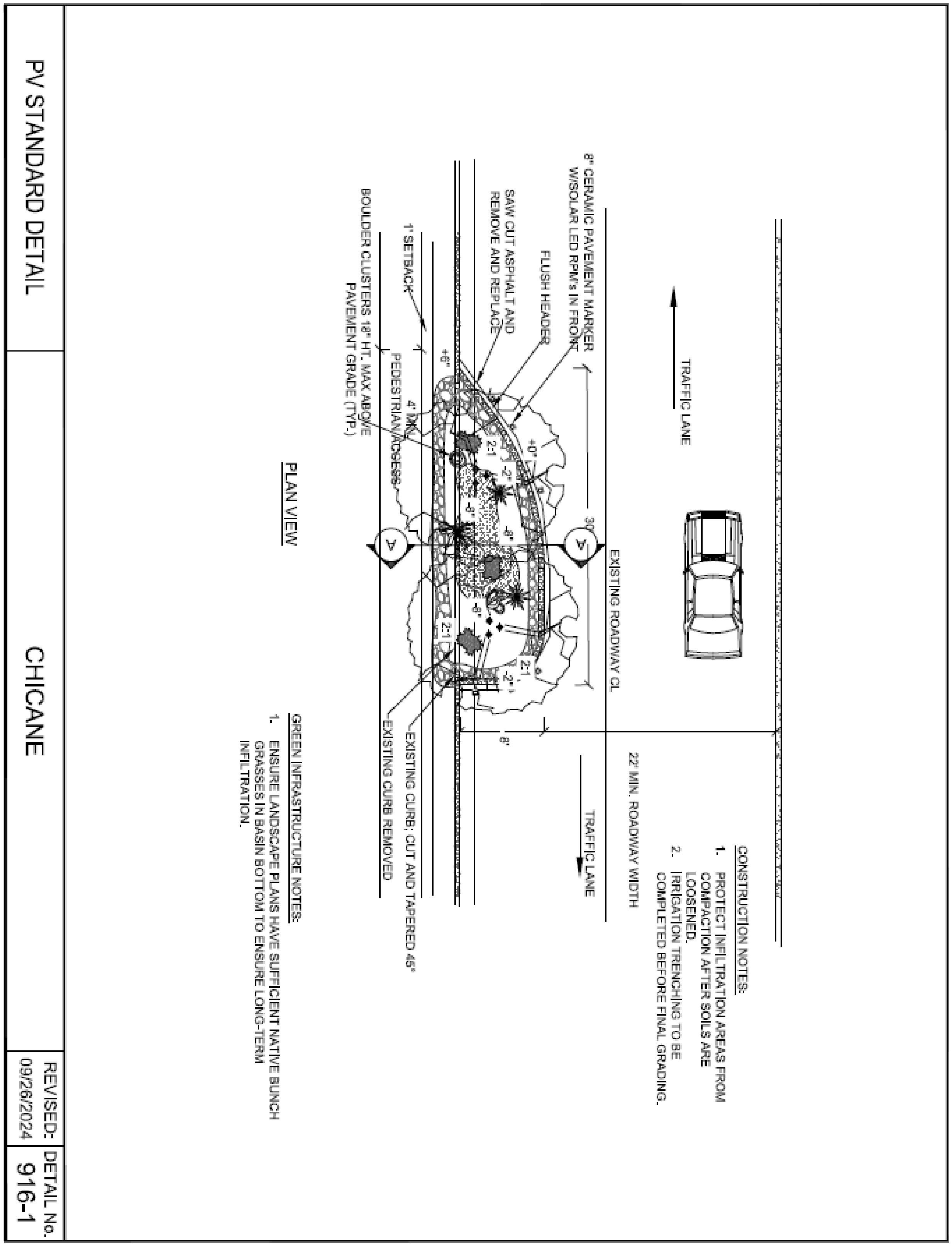
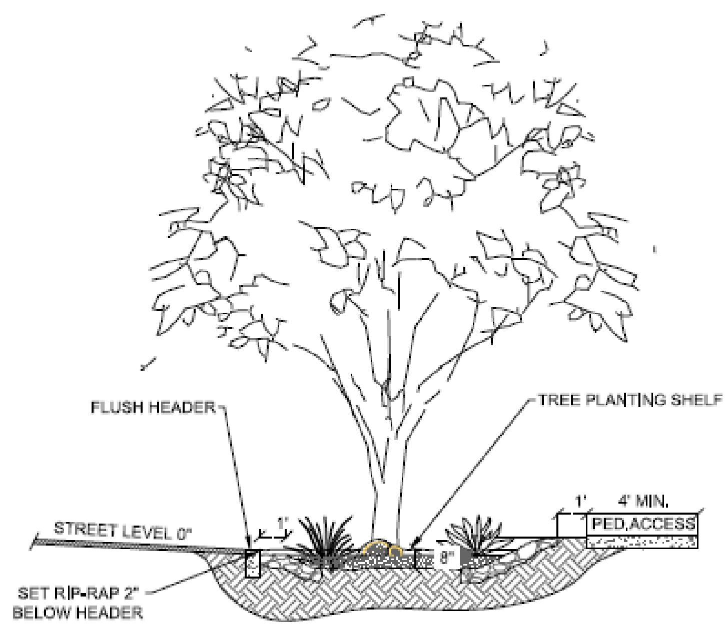


Figure 2-48. Detail No. 916-1 Chicane



SECTION A-A

CONSTRUCTION NOTES:

1. PROTECT INFILTRATION AREAS FROM COMPACTION AFTER SOILS ARE LOOSENED.
2. IRRIGATION TRENCHING TO BE COMPLETED BEFORE FINAL GRADING.

GREEN INFRASTRUCTURE NOTES:

1. ENSURE LANDSCAPE PLANS HAVE SUFFICIENT NATIVE BUNCH GRASSES IN BASIN BOTTOM TO ENSURE LONG-TERM INFILTRATION.

PV STANDARD DETAIL	CHICANE	REVISED: 09/26/2024	DETAIL No. 916-2

Figure 2-49. Detail No. 916-2 Chicane

2.4.3 Street Width Reduction

As discussed above, streets contribute substantially to runoff and the urban heat island. Reducing street width mitigates these negative impacts. Incorporating sediment traps, compost chimneys, bioretention, infiltration trenches, and check dams into a street width reduction project maximizes benefits (see Figure 2-50 through Figure 2-52). Street Width Reduction is applicable for long and wider in-street interventions. Chicanes better serve more constrained sites or locations requiring meandering traffic calming features. If using domed overflows for conveyance, ensure low flows are retained and/or will flow downstream. This maximizes the irrigation benefit while minimizing maintenance by effectively managing sediment and debris.

Figure 2-50. Street Width Reduction

Source: Tucson Clean and Beautiful.

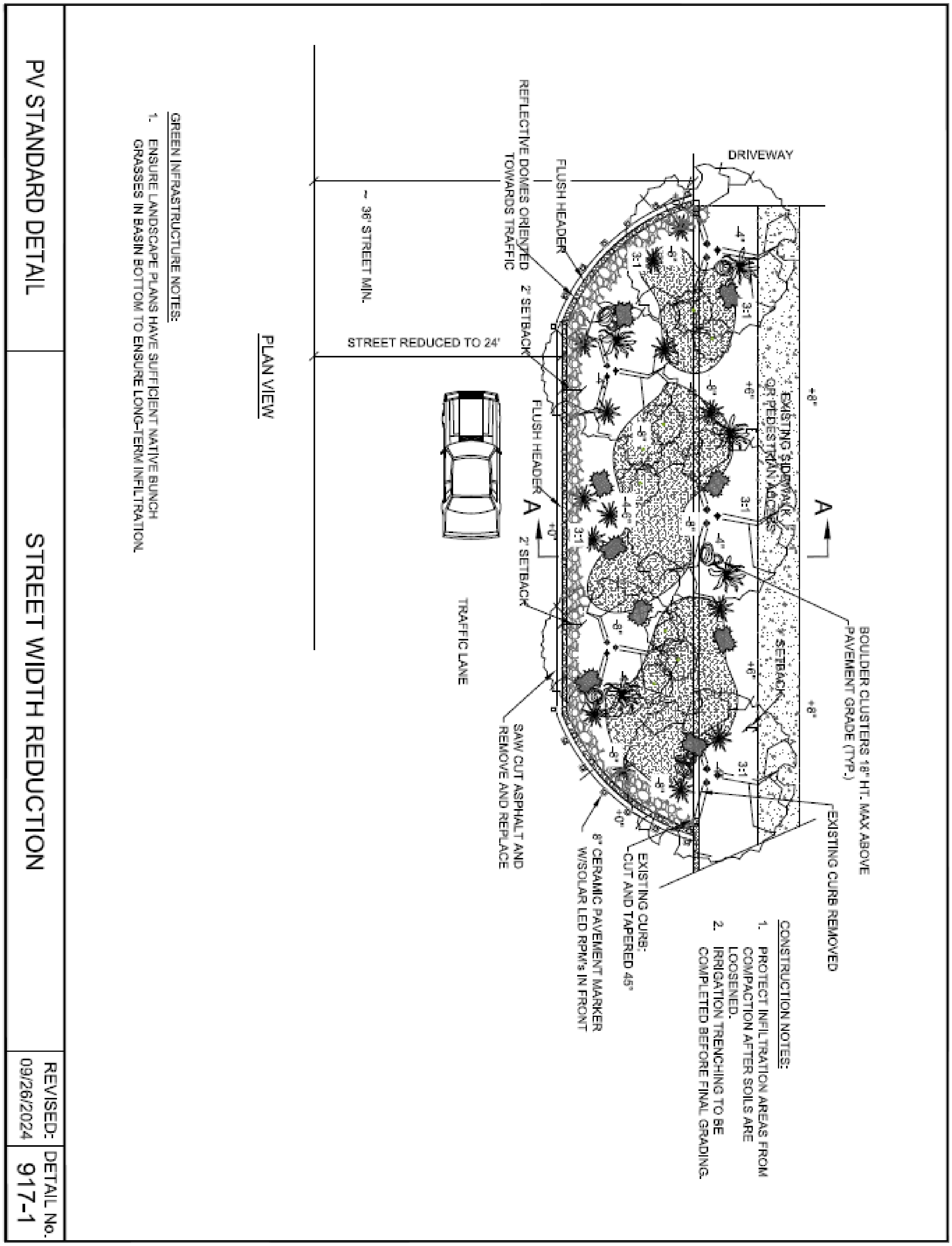
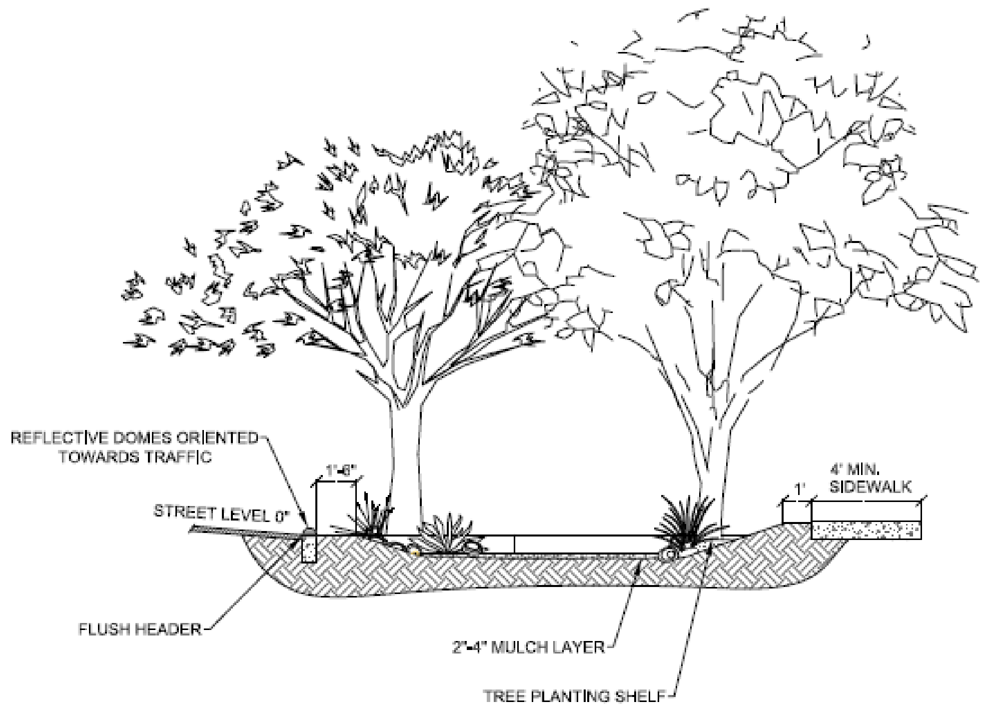


Figure 2-51. Detail No. 917-1 Street Width Reduction



SECTION A-A

CONSTRUCTION NOTES:

1. PROTECT INFILTRATION AREAS FROM COMPACTION AFTER SOILS ARE LOOSENED.
2. IRRIGATION TRENCHING TO BE COMPLETED BEFORE FINAL GRADING.

GREEN INFRASTRUCTURE NOTES:

1. ENSURE LANDSCAPE PLANS HAVE SUFFICIENT NATIVE BUNCH GRASSES IN BASIN BOTTOM TO ENSURE LONG-TERM INFILTRATION.

PV STANDARD DETAIL	STREET WIDTH RECUCTION	REVISED: 09/26/2024	DETAIL No. 917-2

Figure 2-52. Detail No. 917-2 Street Width Reduction

2.5 Infiltration Technologies

2.5.1 Permeable Block

Permeable block provides a suitable surface for traffic movement while allowing rainfall to infiltrate where it falls, preventing runoff accumulation on roads enhancing driver, cyclist, and pedestrian safety. Clogging and reduced performance are risks of permeable surfaces in places with airborne dust and sediment laden water, however the block in Figure 2.53 through Figure 2.55 have capacity to manage dust and sediment while also maintaining high infiltration rates of over 1,000 in/hr. Ideally, pavement surfaces can be maintained with conventional street maintenance equipment, can infiltrate high rates of rainfall for large watersheds with high storm intensity, and can be combined with subsurface conveyance and/or storage.

Figure 2-53. Example Permeable Block Installation

Note: This shows a subsurface capacity from strong arch shape with high performance infiltration design that prevents clogging by infiltrating between the blocks.

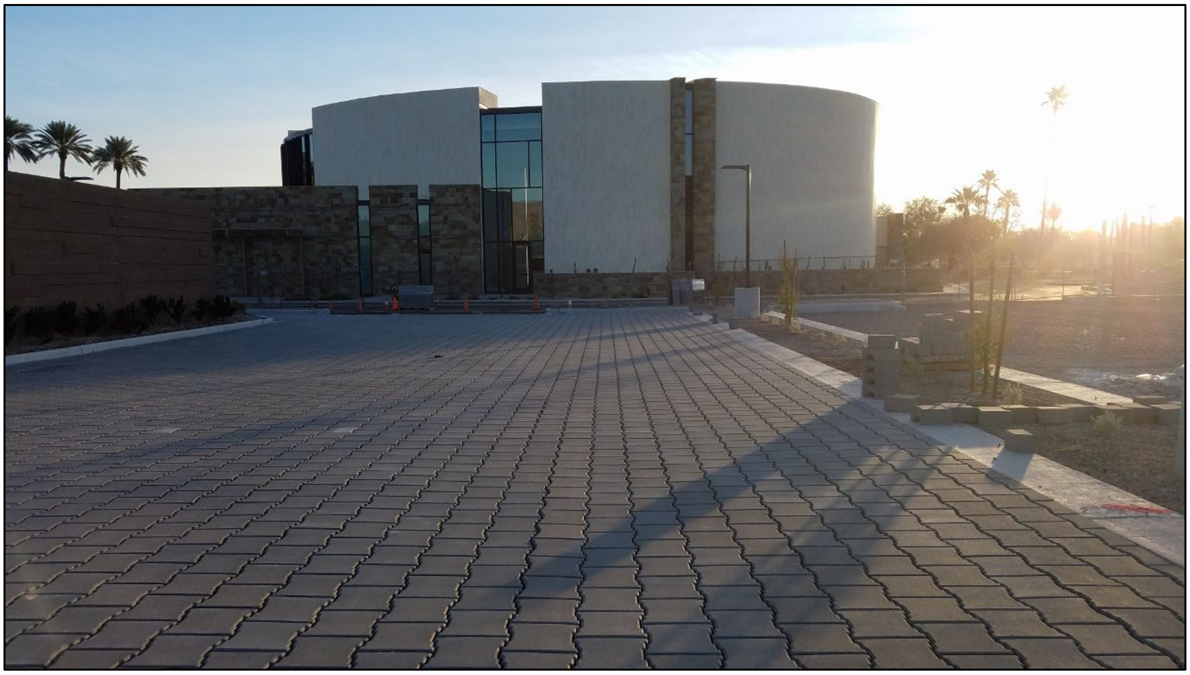


Figure 2-54. Example at the Franciscan Renewal Center

Note: Example showcases[1] a parking lot at the Franciscan Renewal Center with 26,910 square feet of permeable block

[1] Project case study recognized by the Pope through his global campaign whose aim is to encourage and promote integral ecological conversion and sustainability.
https://www.laudatosirevolution.org/project/energy-transition-in-the-franciscan-renewal-center/

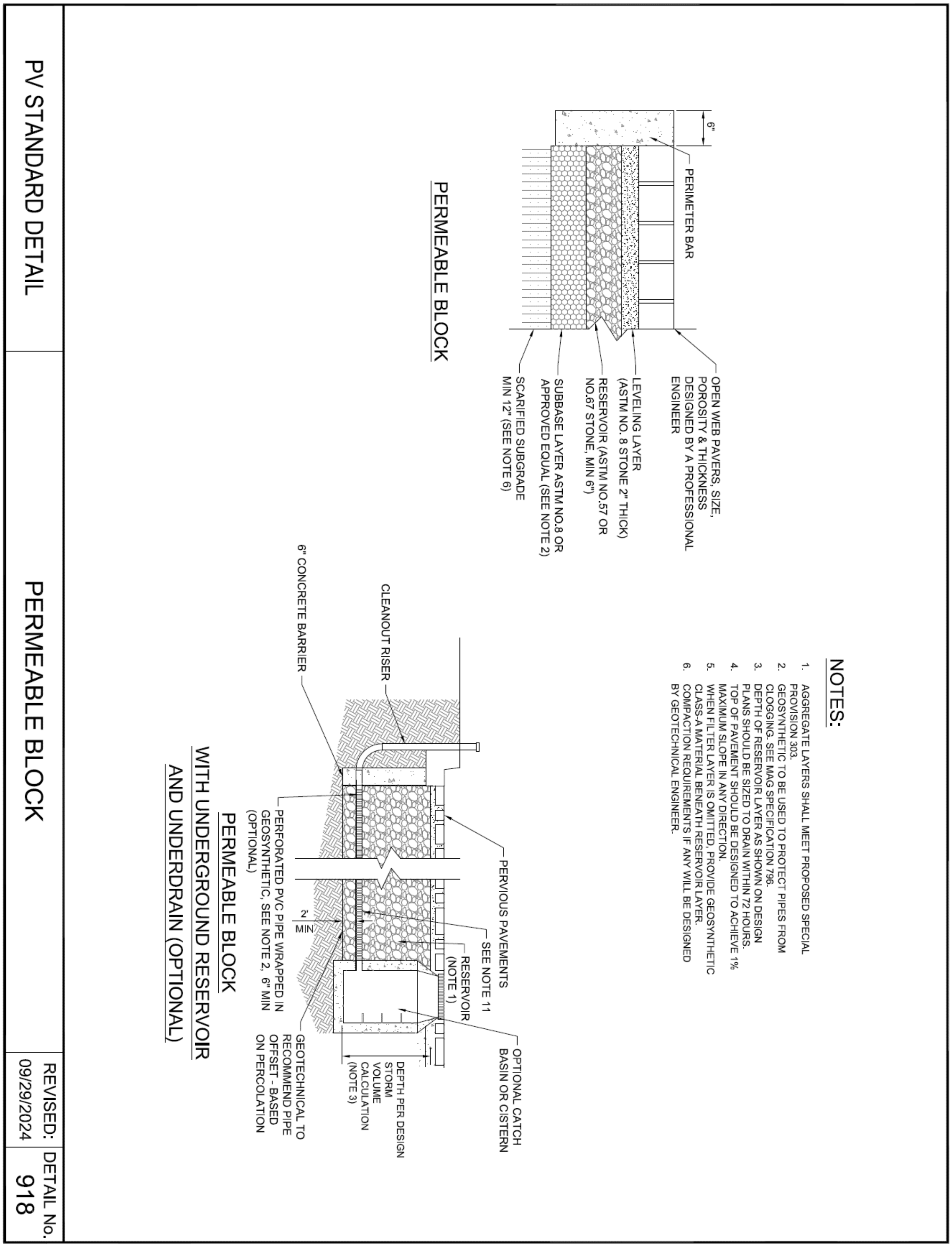


Figure 2-55. Detail No. 918 Permeable Block

2.5.2 Deep Infiltration System

Critical to water security is preventing evaporation within the Town through deep infiltration. This can only be accomplished by getting water below the evaporative zone estimated to be 10–30 feet or more below the surface depending on the soils and evaporation rates.

One successful technology shown in Figure 2.56 through Figure 2.58 enhances soil infiltration capacity with its unique geometry. This system can be installed within stormwater harvesting basins to allow for deep infiltration of stormwater while also creating a positive irrigation benefit for vegetation.

Figure 2-56. Unique Geometry of Parjana's IRIS Technology Ensures Successful Deep Infiltration of Stormwater

Source: Parjana Engineering

Figure 2-57. Installation of Parjana's IRIS to Promote Infiltration in an Athletic Field

Source: Parjana Engineering.

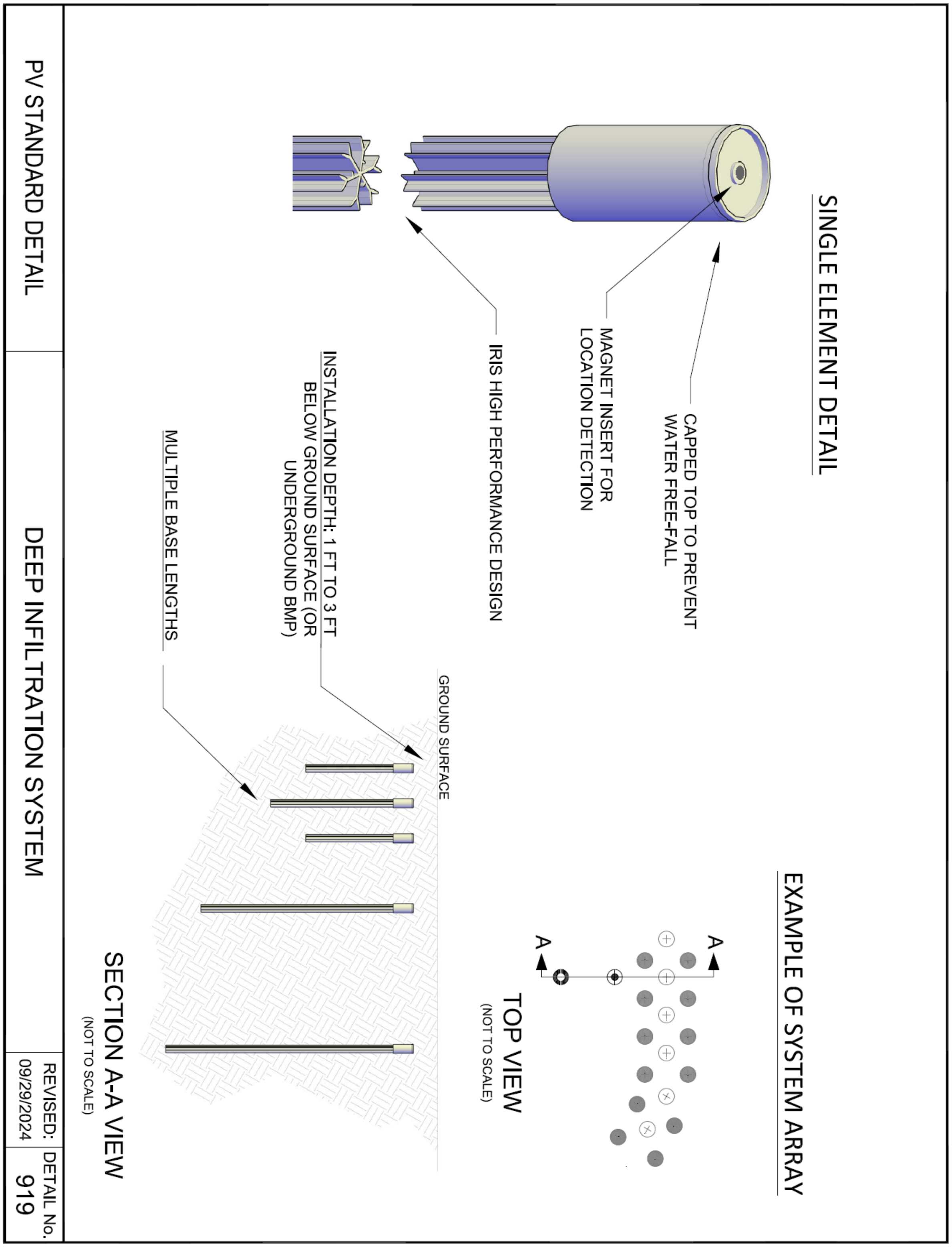


Figure 2-58. Detail No. 919 Deep Infiltration Technology.

3 Site Guidance Considerations

The distributed use of GSI to manage stormwater close to where it falls improves quality of life in the desert and reduces urban heat islands by maximizing shade from native trees. Despite available guidance and policies, new developments have not always effectively implemented GSI as intended by local best practices. The City of Tucson and Pima County have required new development to manage the first inch of rainfall or first flush volume through distributed GSI. GSI construction details, site planning guidance, and training will help both adoption of GSI in new development and the use of best practices. This section outlines several tools and resources for appropriate GSI planning and design.

3.1 Planning

Identifying GSI opportunities must be done strategically and opportunistically. Over the long-term GSI specific capital improvement projects (CIP) programs can be developed as funding allows. In the short-term, GSI must be opportunistically implemented where it is most easily built in order to demonstrate community and educational benefits with the lowest costs and greatest chance of success. Education and training are critical in order to ensure planning, design, construction, and maintenance practices are utilized and so the community understands the full range of benefits to support GSI.

GSI opportunities can be identified and prioritized by a variety of criteria depending on regional priorities and challenges as well as available data. The criteria below are a few examples of relevant data for planning GSI projects.

- Topography – Does water flow to potential GSI inlets?
- Utility locations – Can minimum required distances (based on local ordinances between GSI and overhead and underground utilities) be maintained?
- Available area – Is the potential area at least 5 feet wide without any utility, pedestrian, or other conflicts? (most relevant in rights-of-way)
- Planned construction – Is new construction being planned? GSI can often be easily built into a project if it is being planned or if it is still within the design concept phase.
- Drainage complaints & flood risk – Can GSI address local flooding challenges?
- Community resources – Can GSI enhance shade for community centers, parks, churches as well as other important community gathering spaces?
- Pedestrian and bike corridors – Can GSI be utilized to improve the comfort of alternative transportation infrastructure?

3.2 Design

Integrated design provides an opportunity to blend the full range of GSI practices based on observations of existing conditions to address the needs and constraints of a site. When possible, work with an engineer, designer, landscaper, contractor, or consultant who has appropriate experience in Arizona and accreditation for GSI design and implementation and maintenance for the relevant context.

A common phrase among experienced GSI practitioners is "slow, spread, and sink stormwater." Integrated design is the process of examining flows of water across a landscape and looking for the best opportunities to utilize that water to achieve the design goals by implementing GSI practices to slow, spread, and sink stormwater. Cisterns, curb cuts, bioretention, and stormwater harvesting basins can be used to slow water. one rock dams, check dams, sheet flow spreaders, and Zuni bowls can spread water. stormwater harvesting basins, compost chimneys, bioretention, and infiltration trenches can sink and infiltrate water.

For more details on the integrated design process, the books Rainwater Harvesting for Drylands and Beyond Vols 1 & 2, are excellent resources for homeowners and design professionals. The website associated with these books[2] has many useful resources freely available.

The eight principles of integrated design from Brad Lancaster follow the steps below:

1. Long and thoughtful observation
2. Start at the top of the watershed
3. Start small and simple
4. Spread and infiltrate the flow of water
5. Plan to manage all overflows as a resource where possible
6. Maximize living and organic ground cover
7. Maximize benefits and efficiency
8. Continually reassess and improve system

This process can be utilized by residents and professional designers. A critical component, whose importance should not be overlooked, is long and thoughtful observation. Many flaws in GSI projects can be avoided by better understanding the context and problem. While the design process can be sped up by working with a trained professional, utilizing observations from a property owner, who has spent multiple rainy seasons observing water flows, is ideal.

[2] Rainwater Harvesting for Drylands and Beyond, Brad Lancaster, http://www.harvestingrainwater.com/.

3.2.1 Setbacks

Table 3-1. Suggested Utility Setback

Source: City of Tucson

Underground Utility	Large Tree over 20 feet Tall	Plants Under 20 feet Tall	Plants Under 3 feet Tall
Gas	8	5	3
Sewer	16; 10 if sewer line is deeper than 8’	10	5
Electric	3	3	3
Drinking Water	10	5	3
Cable/Fiber	5	5	3

3.2.2 Design Calculations

To quickly assess the potential for GSI and size features, the following sections outline simplified calculations for rapid initial GSI sizing.

3.2.2.1 Stormwater Harvesting Basins

Sizing stormwater harvesting basins based on roof runoff can be done with this simple calculation: for every 1,000 square feet of roof, 600 gallons of water are produced per inch of rainfall, which roughly translates to 100 square feet of stormwater harvesting basin based on a depth of 9 inches. See Table 3-2 for a range of stormwater harvesting basin areas for different roof areas and rainfall events.

Table 3-2. The Approximate Stormwater Harvesting Basin Area Needed for Various Rainfall Events and Roof Areas

Roof Area (sf)	500	1,000	2,500	3,500
Rainfall	Stormwater harvesting basin Area (sf)			
0.25”	13	25	63	88
0.50”	25	50	125	175
0.75”	38	75	188	263
1.00”	50	100	250	350
1.50”	75	150	375	525
2.50”	125	250	625	875
3.50”	175	350	875	1,225
5.00”	250	500	1,250	1,750

3.2.2.2 Cisterns

Cistern sizing should be based on the appropriate goals defined by the property owner and can include:

- supplemental irrigation needs for fruit trees and vegetable gardens,

- primary/sole irrigation source for food production,
- supplemental water source for indoor and outdoor water needs, and
- primary/sole water source for property.

These various goals will require sizing based on available water from roof surfaces, budget, space for tanks, topography, and roof height. Supplemental irrigation needs and indoor/outdoor water needs can be met as project budget allows. In order to meet the needs of all irrigation for food, it is important to ensure food production has been planned and sized appropriately. Given the amount and pattern of rainfall in the Town, it is important to reduce water consumption through water conservation measures in order to maximize the use of rainwater and minimize the cost of storage needed. Given the interest and available budget ($20K+) for storing, treating, and utilizing rainwater as the sole water source for a property, sizing a tank or tanks to capture half the annual rainfall for the area is recommended. See Table 3-3 for cistern sizing based on rain events and roof areas.

Table 3-3. Cistern Capacity for Different Rain Events and Roof Sizes

Roof Area (sf)	500	1,000	2,500	3,500
Rainfall	Cistern Capacity (gal)			
0.25”	75	150	375	525
0.50”	150	300	750	1,050
0.75”	225	450	1,125	1,575
1.00”	300	600	1,500	2,100
1.50”	450	900	2,250	3,150
2.50”	750	1,500	3,750	5,250
5.00”	1,500	3,000	7,500	10,500
10.00”	3,000	6,000	15,000	21,000

Exact cistern capacity and dimensions will vary depending on the brand and type of material. It is not recommended to plan or utilize cisterns to harvest water off surfaces other than roofs as the maintenance and pre-treatment of water makes tanks cost prohibitive for most residential users. Roofs typically supply more water than needed for the goals of a property owner.

3.3 Water Budget

Central to water conservation practices with GSI is ensuring water demands of plant material can be met through stormwater runoff after establishment. A well-designed street harvesting GSI feature stormwater supply will easily exceed plant demand, given the large watersheds relative to plant demand. A 100-square-foot GSI basin with two native trees, 4 grasses, 2 groundcovers, and 2 shrubs would consume approximately 6,000 gallons of water per year at maturity. 6,000 gallons of water would be produced for every inch of rainfall over 10,000 square feet. This is equivalent to a 400’ asphalt roadway, with two 12’ lanes, and 1’ of gutter.

Given the rainfall patterns in the Town, it is possible to design GSI projects without irrigation systems, as long as appropriate plants are selected, soil health best practices are implemented, and monitoring is done to ensure plants are watered with water trucks during dry periods while plants are establishing (2-3 years after planting). With appropriate design and construction planning to account for ideal planting time to receive winter rainfall, long-term maintenance costs and cost benefit ratios can be maximized by not investing in irrigation systems.

3.4 Soil Health

Soil health best practices are critical to address the challenges presented by the various soil types in the Town. Regardless of soil type, human activity has created significant compaction and degradation of soil biology leading to high rates of runoff, erosion, and flooding. Integrating the following best practices will improve soil health and reduce compaction during construction and maintenance, allowing successful performance of GSI in any soil type:

- Establish native bunch grasses and/or other shrubs or plants with deep fibrous roots.
- Inoculate infiltration areas and roots with beneficial microbes (see GSI Specification Biological Soil Amendment) to improve soil structure and infiltration rates.
- Over excavate and uncompact stormwater harvesting basin areas to break through difficult soil layers.
- Ensure construction scheduling will not compact infiltration areas.
- Cover infiltration areas with plants; organic wood mulch; and/or rock, compost, wood mulch mix.

Soil health is the single most important component of GSI. It is also the most commonly overlooked part of GSI projects in arid environments. For every 1% increase in organic matter, the soil can retain an additional 20,000 gallons of water per acre[3]. It is imperative that healthy soil best practices are central to GSI projects and that communities learn what are the most appropriate practices for local climate, vegetation, and GSI goals.

GSI goals across the southwest and the nation are diverse and sometimes do not work well together. For example, managing large volumes of water in GSI for flood control or water quality benefits with high flow soil mixes will not allow vegetation to thrive in the face of drought or even regular dry periods due to low moisture holding capacity. Technologies exist to address this challenge, but often exceed project budgets of communities that do not have well-funded stormwater or GSI programs and/or the regulatory drivers of communities with combined sewers.

Given the importance of shade and water conservation needs of the arid southwest, an emphasis on simple improvements to soil, that can establish and maintain a healthy tree canopy as well as deep rooted grasses and shrubs, is essential. This allows for improved infiltration rates, increased water pollution reductions, and can improve moisture retention for the right plant palette. It is important to address soil health in both design and maintenance of GSI. Our model for soil health is based on healthy forest and grassland desert ecosystems that create major benefits for humans beyond the often-degraded landscapes created as a result of past human activities. These systems thrive with healthy nutrient cycles that create a thick layer of

[3] “Unlock the Secrets in Soil,” Natural Resources Conservation Service, 2014 Soils Primer.

topsoil rich in organic matter from decomposing plant and animal litter. Evidence of healthy soils can be observed with the presence of mosses, lichens, fungi, and mycelium. Fungi and mycelium have shown the potential to improve water quality in GSI practices with little or no additional costs (see Figure 3.1)[4,5].

Figure 3-1. Mulch Shown With a White Mat of Mycelium

Photo courtesy of Fungi Perfecti.

Soil health benefits can be maximized in design and construction with the following practices.

1. Compaction is minimized by avoiding infiltration and planting areas with heavy equipment while also breaking up physical compaction by ripping soils 12-24" below the basin bottom depth.
2. Healthy native plants, especially bunch grasses or other native grasses, are planted in basin bottoms that can thrive with periodic inundation and have significant root mass to improve infiltration rates and provide a substrate for microbial treatment of stormwater pollutants.
3. Organic mulch ground cover is utilized where possible. Wood mulch is critical to jumpstarting soil health and is preferred over rock mulch. Larger mulch can be used in basin/rain garden bottoms. The depth can be varied based on local conditions and design goals, but typical depths are two to four inches. Rock should only be used in areas where mulch will float away and/or hydrology dictates the need for rock to eliminate scour. Overuse of rock in basin bottoms and as ground cover increases urban heat island impacts and can also cause water quality issues.
4. Soil inoculants that provide healthy mycelial and microbial communities can be added to mulch, under mulch, in the soil, and on plant roots when planting to provide many benefits to soil and vegetation including improved infiltration, water retention and water quality improvements[6]. Worm castings, compost, and compost teas can be made with local organic

[4] Stamets, P., Taylor, A. Implementing Fungal Cultivation in Biofiltration Systems. USDA Forest Service Proceedings, RMRS-P-72. 2014
[5] Taylor, A., Flatt, A., Beutel, M., Wolff, M., Brownson, K., Stamets, P. Removal of Escherichia coli from synthetic stormwater using mycofiltration. Ecological Engineering. Volume 78, May 2015, Pages 79-86.
[6] The Drought Tolerant Garden: Los Angeles County Handbook, 2012.

matter to provide inoculants if purchasing commercial products is not possible. Utilizing local organic materials to produce inoculants or commercial products tailored for the local plant and soil conditions is preferred in order to jumpstart the native soil biology most effectively.

5. Native tree canopy coverage over entire project footprint, and ideally beyond, is created to protect the ground and plantings from the erosive forces of high intensity rainfall while providing microclimates for understory plant establishment, decreasing soil temperatures, and reducing the urban heat island impacts.

3.5 Rock Work

Rock work is important for stabilizing soil in a variety of conditions. Throughout southwestern landscapes and GSI, rock is overused as a primary ground cover, and this creates significant water quality and infiltration challenges. Appropriate rock work in GSI should focus on stabilizing soil with the long-term intention of establishing vegetation.

Rock in GSI bottoms should only be utilized when water velocities create a scour potential that does not allow for vegetation to establish, and organic mulch would wash away. Ensuring rock of appropriate size and quality for the context, in addition to minimizing fines, is critical to ensure long-term infiltration.

Careful rock placement is essential to prevent scour and erosion around rockwork. Quivira Coalition's *Erosion Control Field Guide* provides several best practices for rock work.

Best practices for rock work include:

1. ensuring rock is sized appropriately for design conditions,
2. providing adequate support for rock features where the erosive forces of water are typically most destructive (the edges and bottoms of features), and
3. utilizing skilled and experienced labor for the context.

3.6 Vegetation

The right plant for the right place is an important phrase for GSI. Vegetation selection and placement decisions should:

- enhance infiltration rates,
- create shade for people, other vegetation, and soil,
- stabilize soils,
- block nuisance views and noises,
- maintain lines of sight for vehicle, bike, and pedestrian safety,
- maintain safe access to GSI facilities,
- meet plant material water demands, and

- maintain a safe distance from other infrastructure.

The Prescott Active Management Area 5th Management Plan Low Water-Use and Drought Tolerant Plant List[7] is a good starting point for GSI plants, however, there are many plants that are not appropriate for GSI included on this list. The two types of plants that are important for GSI are trees and grasses given their function to modify the local microclimate to promote healthy soils that enhance infiltration and reduce evaporation.

Table 3-4 includes a selection of trees and grasses highly recommended for GSI based on local experience as well as experience. These plants are recommended as core infrastructure plants for the Town that can be expanded upon as local GSI experience develops.

Table 3-4. Trees and Grasses Recommended for GSI

Botanical Name	Common Names
TREES	
Celtis reticulata	Netleaf Hackberry, Western Hackberry, Canyon Hackberry
Chilopsis linearis	Desert Willow, Desert Willow 'Warren Jones,' Desert Willow 'Lucretia Hamilton'
Fraxinus velutina	Velvet Ash, Arizona Ash
Juglans major	Arizona Walnut, Arizona Black Walnut, New Mexico Walnut
Quercus arizonica	Arizona White Oak
Sapindus Sponaria	Western Soapberry
GRASSES	
Aristida purpurea	Purple Three-Awn
Bouteloua curtipendula	Sideoats Gramma
Bouteloua gracilis	Blue Gramma
Muhlenbergia emersleyi	Bullgrass, Bull Muhly
Muhlenbergia rigens	Deergrass, Deer Muhly
Sporobolus airoides	Alkali Sacaton, Alkali Dropseed
Sporobolus cryptandrus	Sand Dropseed

Trees that should NOT be included in GSI due to their non-native characteristics that reduce their infrastructure function within a water harvesting basin are included in Table 3-5.

[7] Prescott Active Management Area 5th Management Plan Low Water-Use and Drought Tolerant Plant List: https://www.azwater.gov/sites/default/files/media/5MPPrescottLWUPL_Final_0.pdf

Table 3-5. Trees Not Recommended for GSI

Botanical Name	Common Names
Albizia julibrissin	Silk Tree, Mimosa, Nemu Tree
Calocedrus decurrens	Incense Cedar
Cedrus atlantica	Atlas Cedar
Cercis canadensis var. texensis ‘Oklahoma’	Oklahoma Redbud
Cotinus coggygria	European Smoketree, Purple Smoke Tree
Crataegus laevigata	Smooth Hawthorn, English Hawthorn
Pinus heldreichii	Bosnian Pine
Pinus nigra	Austrian Pine
Pinus sylvestris	Scots Pine, Scotch Pine
Pistacia chinensis	Chinese Pistache
Prunus padus	European Bird Cherry, Mayday Tree
Prunus serotina	Black Cherry
Styphnolobium japonicum	Japanese Pagoda Tree
Ulmus parvifolia	Chinese Elm, Evergreen Elm

A local Town GSI plant list needs to be developed to break down suitable GSI plants into “Above the inundation elevation” and “Below the inundation elevation.” It is important to develop and refine a list of local, native plants that thrive in the conditions created by GSI features. Additional categories such as basin bottom, side of basin, top of berm could also expand the above list. Given that evaporation is a major source of water loss, native plants that can thrive with periodic inundation, create shade, and provide a source of organic matter over time to the soil are critical to improve moisture holding capacity while also decreasing long-term maintenance needs and costs. Work with local nurseries to select native seed, propagate, and establish plants in a way that will maximize their success in the harsh conditions of the desert and unique conditions of GSI infrastructure. Pots, such as “Tall Pots,” that allow native trees to establish a healthy tap root are critical to healthy trees in GSI projects that can withstand drought, maximize infiltration, and water quality benefits, and withstand strong winds with saturated soils.

3.7 Installation

The difference between success and failure for GSI can be a matter of inches. Construction observation and contractor training and/or experience are critical for the success of GSI projects. The art and act of GSI construction must embrace all the best practices of design as outlined in this section. Having an experienced contractor and/or construction observer involved in GSI projects, programs, and processes, who understands how to communicate these best practices in ways that address needs and knowledge of equipment operators, general contractors, laborers, and community members, is critical. In many arid areas where GSI has not been broadly accepted and implemented, there is the challenge to bring the planning, design, installation, and maintenance professionals up to a common baseline of GSI best practices.

Education, training, and demonstration sites are critical to building support for GSI and to create the knowledge and tangible understanding of best practices through hands-on training.

3.8 Maintenance

It is important that communities reassess the value of - and need for - the typical neat and tidy landscape. Conventionally maintained landscapes have tremendous direct and indirect negative impacts on communities. Pollution from noises, chemicals, dust, runoff, and fossil fuels are some of the results of having manicured landscapes that are not healthy for nature or people. Research shows the lack of nature in urban environments is having detrimental impacts on human physical and emotional health. Additionally, the negative public health impacts and many lawsuit rulings against the use of chemicals like glyphosate, found in the commonly used weed killer Roundup®, have resulted in many bans and commitments to eliminate the use of those chemicals in cities and countries across the globe[8]. Conventional maintenance practices will guarantee the need for chemical-based practices that negatively impact community health while ensuring long-term costs will increase over time. Alternatives to conventional chemical management described below are cost-effective as the resources needed to maintain the landscape decrease over time relative to chemical management as a result of building soil health.

Maintenance practices that are closer to the fun of gardening in your lush garden are needed to change the perception of maintenance and change the aesthetic to a more natural beauty that will decrease the direct costs of maintenance over time and will decrease the indirect costs of health impacts created by conventional maintenance.

Often overlooked and under or unfunded, maintenance is critical to successful GSI programs in arid environments. The only offense greater than deferred maintenance is inappropriate maintenance, as shown in photo below. Education and training programs are critical to ensure the GSI knowledge base in a region is healthy and can grow with the demand for projects and maintenance needs. The *Phoenix GSI Handbook* outlines the basic, standard maintenance practices necessary for each GSI practice. The Tucson Storm 2 Shade Program has a well-designed maintenance guide in English and Spanish that can be found here: https://climateaction.tucsonaz.gov/pages/gsimaintenanceguide.

Critical best practices and common practices that need to be avoided are shown in Figure 3.2. This is an important challenge to address early in the development of a GSI program. GSI must be maintained as infrastructure within an asset management program. Often, GSI is treated as a typical landscape with the result looking like this:

[8] https://www.baumhedlundlaw.com/toxic-tort-law/monsanto-roundup-lawsuit/where-is-glyphosate-banned/

Figure 3-2. A Commercial GSI Installation Sprayed for Weeds and Excessively Pruned

With the organic mulch removed, native bunch grasses and shrubs sprayed, native trees severely pruned into a lollipop, all the infrastructure functions of the GSI feature have been impaired, and if continued, will eliminate all functionality. GSI managed with current conventional landscape maintenance techniques, will result in a stormwater hazard and nuisance that will pond, creating mosquito habitat while also decreasing water quality by negating the beneficial functions of the soil and plant root ecosystem. When practices that impact soil health negatively are employed, maintenance needs will remain consistent and will lead to higher longer-term costs over time.

It is important to understand the function of weeds in nature. They are intended to protect and heal soil. When we continue to damage soil through conventional maintenance practices, we guarantee weeds will persist and increase long-term maintenance needs. By creating conditions for healthy soil, native plants will thrive as the presence of weeds decreases with time. The weeds that do appear in GSI features will be easier to eradicate with healthy soils. Weeds will likely not be eliminated given that, primarily by stormwater, seeds will be brought into GSI features; however, with a mature tree canopy and a healthy native understory and groundcover, weeds will no longer be needed to protect the soil and germination rates will diminish over time. The ease of removal will also improve as soil is naturally aerated by root growth and organic mulch decomposition.

In order to maintain healthy soils, ensure the guidelines below are followed:

- Utilize physical removal of undesired plants. Animals can be an advantageous way to remove undesirable weeds and invasive plants, while building soil health.
- Do not use chemicals.
- Minimize raking where possible and do not use leaf blowers.
- Use trimmings from native vegetation as soil cover/mulch replacement after chopping it to an acceptable size to drop where needed.

It is important that maintenance guidance be developed with simple language that appeals to the contractor industry, community decision makers, and private property owners. An example of a simple guide is the

NMDOT GSI Maintenance Field Guide[9]. An introduction to desirable and undesirable plants in the desert, pruning, and regular stormwater harvesting basin care activities is presented.

When addressing a problem site or a GSI feature than has not been maintained adequately or at all, ensure adequate time and resources are allocated to address the issue up front. GSI features are too often blasted with chemicals and then weed wacked to remove the nuisance vegetation growth. However, this only destroys any soil life and resets the conditions for new weed growth. If problem plants, like Bermuda grass, are choking features, it is important to try physical removal with a hoe. If a nuisance plant like Bermuda grass is well-established, a major retrofit of the GSI soil may be necessary with heavy equipment. Ensure tree roots are adequately protected by using an experienced operator and involving a certified arborist as needed.

Many conventional landscape maintenance companies are not typically equipped or trained for appropriate GSI maintenance. For example, the Sustainable Landscape Management (SLM) certification provided by the Arizona Landscape Contractors Association does not fully meet the needs of GSI. Often the certification is possessed by someone behind a desk who does not always impart the relevant knowledge to crew leaders and landscape technicians. The City of Tucson developed a GSI maintenance training and certification program that could be adapted for the Town.

[9] New Mexico Department of Transportation GSI Maintenance Field Guide*:*
https://www.dot.nm.gov/wp-content/uploads/2023/09/2023-08-14-GSI-Maintenance-Field-Guide_Final-for-Print.pdf

4 Green Stormwater Infrastructure (GSI) Specifications

- **Mulch**. Mulch shall consist of composted, ground, or shredded wood from native trees with at least 85 percent able to pass through a 3-inch screen. The pH shall not exceed 7.5. The mulch shall be hygroscopic, free of chemicals, weed seeds, debris, and trash.

- **Biological Soil Amendment**. The amendment shall contain a minimum of aerobic beneficial components as outlined in Table 4-1. The water used to apply amendment shall be free of chlorine and disinfection byproducts.

Table 4-1. Biological Soil Amendment

	Minimum Biological Requirements
Bacterial Biomass (ug/g)	135
Fungal Biomass (ug/g)	135
F:B ratio	>3:1
Protozoa (ug/g)	10,000
Beneficial Nematodes (count per g)	100
Ciliates (count per g)	<5

- **Bioretention Soil Mix**. A constructed soil medium that is a mixture of coarse sand (not more than 15% ASTM C-33 sand by volume), organic mature compost (15-25% leaf compost, vermicompost or equivalent) mixed in a drum mixer; and topsoil (max 70%). Topsoil shall be fertile, friable soil obtained from well drained arable land which has or is producing healthy crops, grasses, or other vegetation. It shall be free draining, non-toxic and capable of sustaining healthy plant growth. Topsoil shall be reasonably free of subsoil, refuse, roots, heavy clay, clods, noxious weed seeds, phytotoxic materials, coarse sand, large rocks, sticks, brush, litter, and other deleterious substances.

5 Successful GSI Incentives from other communities

Policies and incentives are powerful tools to drive GSI in new construction and retrofit scenarios. A key component to any policy or incentive is providing resources for inspection, enforcement, follow up and maintenance to ensure GSI is constructed and maintained appropriately.

- Parking lot tree canopy and water harvesting policy

  A performance-based tree canopy policy is recommended. City of Tucson requires one tree for every four spaces. Parking lots have sufficient surface area to provide all needed irrigation for 100% canopy coverage from mature native shade trees with stormwater runoff from GSI given appropriate design and grading. Incentives could be offered if additional hardscapes such as streets, sidewalks and roofs are designed to direct stormwater to GSI before storm drains.

- Residential rainwater harvesting rebates

  Tucson and many other cities offer rebates for water harvesting. Tucson has recently expanded its rebate program to include funding street side GSI that results in cost-effective water conservation and flood mitigation benefits.

- Commercial water harvesting policy

  All commercial landscapes must meet 50% of landscape irrigation needs with rainwater harvesting GSI. Best practices have not been enforced and landscapes have become gravel covered pits that get sprayed with pesticides and herbicides and native trees are severely pruned. A performance-based policy revision to include canopy goals is preferred. Additional outcomes regarding water quality can be specified to meet Las Cruces' MS4 permit requirements.

- Green streets policy

  For all new construction of streets in the City of Tucson, the Green Streets Policy requires that GSI meet 50% of irrigation needs and support a 25% tree canopy.

- Parking alignment for optimal shade and water harvesting

  Aligning parking spaces to be oriented East/West so that GSI tree plantings can be North/South will allow for maximum cooling. Parking lots should be graded for distributed water harvesting. By spreading stormwater throughout GSI landscapes, irrigation benefits and flood potential reductions can be maximized. The Metropolitan Area Planning Council in Massachusetts developed a toolkit[10] that addresses best practices and policies for parking lot design.

- Potential development code revisions

  The following items are potential opportunities to revise development codes for new development to reduce future water supply impacts exacerbated by extreme weather and water supply unknowns. This list is not exhaustive, and specific recommendations can be made with a complete review of existing codes.

  - Neighborhood street narrowing
  - Excessive pavement at intersections

[10] Metropolitan Area Planning Council Fact Sheet #2: https://www.mapc.org/wp-content/uploads/2017/11/LID_toolkit_factsheets_1-3.pdf

  - Cul-de-sac dimensions for landscaped median
  - Detention pond retrofit for shade tree irrigation

  The City of Sierra Vista conducted a comprehensive review of opportunities to modify development code to enhance the health of the watershed and San Pedro River[11].

- Vacant properties

  Vacant properties present a tremendous opportunity to convert underutilized land into Town assets with GSI providing stormwater management and flood mitigation benefits. GSI features can also be community assets by creating shaded community gardens, pocket parks and other community gathering spaces. The City of Baltimore hosted the Growing Green Design Competition[12] to develop community support and implementation of GSI projects on vacant lots.

[11] Appendix D from A Stormwater Action Plan for Sierra Vista: https://watershedmg.org/document/appendices-stormwater-action-plan-sierra-vista
[12] U.S. EPA Restoring Vacant Lots to Control Stormwater, Revitalize Neighborhoods: https://www.epa.gov/md/restoring-vacant-lots-control-stormwater-revitalize-neighborhoods

6 Implementation Plan

GSI provides significant financial and environmental benefits over traditional stormwater management. In the face of climate change and increasing urban heat island impacts, GSI provides an ideal infrastructure solution to utilize wasted stormwater to benefit the community while reducing overall costs to provide infrastructure services relative to traditional stormwater management methods.

In order to implement a GSI program, it is recommended the Town focus on the following key areas in order to maximize cost savings and benefits:

1. Develop key demonstration sites that maximize the hands-on community education benefits;
2. Incorporate GSI into all relevant CIP projects and processes supported by relevant policies;
3. Utilize parks and open space adjacent to hardscape surfaces for GSI to maximize benefits to park users and Town water security needs;
4. Develop Town policies for a GSI based parking lot tree canopy and water harvesting requirement, commercial water harvesting policy, and parking alignment for optimal shade and water harvesting to promote sound design of parking infrastructure to reduce property owners' maintenance and operations costs;
5. Analyze the full range of costs and benefits to promote the lifecycle benefits of GSI in new construction and redevelopment;
6. Evaluate the full range of funding options to promote GSI capital projects and to develop a GSI program; and
7. Develop a policy to utilize vacant properties for GSI.