PINAL COUNTY
DRAINAGE MANUAL

Volume I

Design Criteria

August 2004

VOLUME 1 – DESIGN CRITERIA

PINAL DRAINAGE ORGANIZATION

The drainage policies used in Pinal County are set forth in the Ordinance to Regulate Drainage in Pinal County. The Pinal County Drainage Manual sets forth design criteria, methodology and procedures and is comprised of two volumes; Volume 1: Policies and Design Criteria and Volume 2: Design Methodology and Procedures. The table of contents for both Volume 1 and 2 are included in each volume for easy accessibility.

Ordinance to Regulate Drainage

The ordinance document establishes general drainage policies and provides the minimum standards for the design of drainage and storm water management facilities within unincorporated Pinal County.

Drainage Manual Volume 1: Design Criteria

Volume 1 establishes minimum standards and criteria for the design of drainage and storm water management facilities within unincorporated Pinal County. It is desirable that the policies and standards set forth in this manual be adopted by local jurisdictional entities so that uniform drainage policies and practices will be established throughout the County. However, each entity has the authority to establish its own policies within its jurisdiction; therefore, the user is encouraged to review the policies and standards for the jurisdiction in which the project is located.

Drainage Manual Volume 2: Design Methodology & Procedures

Volume 2 is intended to serve as an aid in the design of drainage and stormwater management facilities. The manual provides a convenient source of technical information and presents methodologies and procedures acceptable to the County. However, the methodologies and procedures presented in the manual are not comprehensive and are not intended to replace or inhibit sound engineering judgment.

PINAL COUNTY DRAINAGE MANUAL
VOLUME I: DESIGN CRITERIA
TABLE OF CONTENTS

Chapter 1: Introduction

Chapter 2: Drainage Report and Design Submittal

Chapter 3: Design Criteria

VOLUME 1 – DESIGN CRITERIA

Chapter 1: Introduction

1.1 PURPOSE

The purpose of the Pinal County Drainage Manual (referred to as Manual in these documents) is to establish general drainage policies, provide the minimum criteria, and to serve as an aid in the design of drainage and stormwater management facilities within Pinal County. The Manual recommends design standards and criteria that if adopted by local jurisdictional entities will establish uniform drainage policies and practices throughout the County. The Manual comprises two volumes as described in Section 1.3.

It is the overall and primary objective of Pinal County to provide drainage design criteria which serve to protect the health, safety, and general welfare of the citizens of the community, with regards to flooding and drainage issues.

1.2 SCOPE

The Pinal County Drainage Manual must be used for any project being reviewed and approved by the County. This includes projects on County property as well as within County rights-of-way. Projects on private property that must be approved by the County must also follow the requirements set forth in this Manual.

Projects under the jurisdiction of towns or cities in Pinal County may also be subject to the provisions of this Manual. Check with the appropriate jurisdiction for specific requirements, as they may not use this Manual or they may have adopted local modifications to the Manual provisions.

This manual is not intended to conflict with any other Pinal County design standards or ordinances. If a conflict does arise, it is the intent of the County to require the more stringent or restrictive standard to apply.

This document provides general engineering guidelines and is not intended to be a substitute for sound engineering judgment when dealing with specific design problems. Specific engineering procedures and methodologies are not always dictated within this manual. Often the manual references other widely accepted design manuals published by the U.S. Army Corps of Engineers, Arizona Department of Transportation, Federal Highway Administration and other regulatory agencies within Arizona. This approach is intended to provide the engineer with the flexibility to apply engineering methods most appropriate to an individual project. Other objectives of this manual include (1) minimizing the review time for drainage reports, (2) providing the design engineer with the County's drainage requirements prior to initiating a project; and, (3) providing for drainage infrastructure that is functional, durable and aesthetically pleasing.

1.2.1 Applicability

This manual is to be used by Civil Engineers in preparing drainage reports for stormwater planning, analysis and design within Pinal County, Arizona. Many procedures that are presented or referenced within this manual have a limited range of applicability. An attempt has been made in this manual to specify these ranges whenever possible within the manual. However, it is the responsibility of the practicing engineer to utilize sound engineering judgment and experience when applying any engineering methodology to a particular project.

1.2.2 Limitations of Liability

The Engineer performing stormwater analyses and preparing drainage reports for projects in Pinal County must assume the final responsibility for the appropriateness of their analysis and correctness of their results. This Manual is not intended to provide "lookup" answers to drainage questions or "one size fits all" methods. Proper and sound engineering judgment is required in all cases. The inappropriate use of and adherence to this Manual does not relieve the Engineer from the professional responsibility to provide an appropriate design.

Adherence to the provisions of this Manual and use of any method contained herein does not relieve any owner, Engineer, or designer of any present or future liability related to the design of works covered by this Manual. Pinal County is not liable for direct or consequential damages resulting from the construction of works covered by this Manual, whether the provisions of this Manual were followed or not.

1.2.3 Floodplain Regulations and Drainage Policies

The County is mandated to adopt and enforce regulations designed to protect health, safety, and general welfare of the citizens within the jurisdiction area of Pinal County and to minimize public and private losses due to flood conditions in specific areas.

The County is also mandated by the Federal Emergency Management Agency (FEMA) to regulate areas of special flood hazards. FEMA supplies the District with Floodway Maps and Flood Insurance Rate Maps which provide flood risk information and other technical data to be used in administering both floodplain management and insurance aspects of the National Flood Insurance Program (NFIP).

Requirements from both agencies have led to the adoption of the Pinal County Drainage Ordinance with potential subsequent revision(s). The Ordinance requires that Pinal County regulate all activities within and along all watercourses within its jurisdiction.

1.2.4 Updates

Pinal County may choose to modify and update this Manual at any time, and anyone needing to perform design or construct works covered by this Manual must be sure that they are using the most current version. Check with the Pinal County Department of Public Works or the Pinal County Flood Control District for the current version.

1.3 UNIFORM POLICY REQUIREMENTS

The following policies have been adopted by Pinal County and shall apply to all projects under County jurisdiction.

1.3.1 Design Procedures

A widely accepted software program may be used in lieu of any design procedures set forth in this manual, with the prior approval from the County Engineer.

Numerous computer software programs have been developed for flood routing through detention/retention facilities. Use of a particular computer program should be approved by the appropriate governing agency prior to its application on a particular project.

1.3.2 Standard Specifications

All hydraulic structures are to be constructed according to the Maricopa Association of Government (MAG) Uniform Standard Specification for Public Works (latest revision) unless specifically superceded by local governing agency Standards and Specifications.

1.3.3 Waters of the US

If the proposed project will impact Waters of the U.S., the engineer or designer shall take into account the requirements of the Clean Water Act, Section 404.

1.4 DESIGN REQUIREMENTS

1.4.1 Warning Signs

Signs should be provided at all designated entry ways of detention and retention facilities. They should also be provided at intervals (~100 feet) around the perimeter of the facility to inform visitors who might gain access at other than designated entrances.

In addition to entry and perimeter signs, signs should be installed within the facility. These signs should restate the potential flood hazard and should provide directions for appropriate routes out of the basin area should flooding occur.

1.4.2 Drainageway Entrance and Exit Points

All drainageway entrance and exit points in the proposed development must remain in the original location and, as near as possible, in the original condition.

Unless special exception is made by the governing agency, all artificial channels must begin and end where historic runoff has flowed.

1.4.3 Floodplain Encroachments

Encroachments into the floodplain of a natural water course are to be analyzed according to the FEMA requirements.

At no time should an encroachment adversely affect the stability of a water course or adversely alter flooding conditions on adjacent properties. When encroachment is proposed within the floodplain of a major watercourse, the regulating entity may, at its discretion, request that a detailed study be performed to determine if a reduction in overbank flood storage will significantly affect downstream flood peaks.

1.4.4 Landscape/Grading

Walls, fences, decorative borders, berms and other similar structures or features, less than one foot in height above finished grade, are permitted without first obtaining a Drainage Clearance provided it does not:

1. Have an adverse effect on adjacent land
2. Obstruct, retard or divert any offsite runoff drainage way
3. Impact other drainage design feature.

This does not relieve any person from liability if that person's actions cause flood damage to any other person or property.

1.4.5 Pesticides

Extensive use of herbicides in basins where the primary or secondary purpose is groundwater recharge is not acceptable.

1.4.6 Stormwater

1.4.6.1 Floodplain Development

That portion of a development that is within a designated special flood hazard area shall comply with Pinal County Floodplain Management Ordinance No. 81582. If a developer desires to re-delineate a floodplain, he shall prepare a Floodplain Analysis following the submittal requirements in Chapter 2 of this Volume and submit the necessary data to the Floodplain Administrator. Development within a delineated floodplain is not exempt from drainage and grading requirements of the Drainage Ordinance.

1.4.6.2 Intent of Drainage Systems

The entire drainage detention, retention, and runoff conveyance system shall be designed to eliminate or minimize storm water runoff effects as well as convey the runoff through the development with minimum detrimental effects to the development or to any other property. No system shall be approved if the effect:

1. Causes an increase in the peak discharge or velocity of runoff
2. Changes the point of entry of drainage onto other property during any runoff event.
3. Impedes runoff from adjoining upstream properties

1.4.7 Revising Flood Insurance Rate Maps

In 1968 the U.S. Congress established the National Flood Insurance Program (NFIP) which enables property owners within participating communities to purchase flood insurance at reasonable rates. The flood-hazard areas identified as Special Flood Hazard Areas (SFHA) have been delineated on Flood Insurance Rate Maps (FIRMs) which are available from Pinal County. These maps depict 100-year flood boundaries, flood insurance rate zones, and regulatory flood elevations, most of which are the result of detailed engineering analyses performed as part of a Flood Insurance Study (FIS).

FIRMs are used by the County for establishing flood-insurance rates for affected structures and for floodplain management by the County. All new development within regulatory floodplains must be reviewed and approved by Pinal County. During the review process County staff may require a more detailed analysis than was presented in the FIS. For smaller developments the FIRMs may be used to establish minimum finished floor elevations, or other site grade elevations.

1.4.7.1 Map Amendments and Revisions

Occasionally, because of limitations of the scale at which a NFIP map was prepared, the floodplain boundaries are not delineated in sufficient detail to reflect individual structures that are elevated on relatively high ground, or show small parcels of land that have been filled. Similarly, floodplain information is subject to change, such as after the construction of drainage improvements or development of more accurate hydrology methods. Since FIRMs are subject to change because of a variety of reasons, FEMA has developed a map modification process designed to keep maps updated with current information.

Information depicted on effective NFIP maps may be changed by a physical revision of the map, by a Letter of Map Revision (LOMR), or by a Letter of Map Amendment (LOMA). New map panels may be printed; or, if the revisions are relatively small, a LOMR/LOMA may be issued that describes the modifications. Changes to effective FIRMs resulting from the exclusion of individual structures and undeveloped parcels are described in a LOMA; whereas communities having updated data, or having constructed new flood-control improvements, may request a LOMR.

The general requirements for technical and scientific data needed to substantiate a LOMR or LOMA are similar. However, there are procedural differences that determine the amount of data required, and how the data is to be submitted. General descriptions of the FIRM modification process are presented within the FEMA publication entitled *Appeals, Revisions, and Amendments to Flood Insurance Rate Maps: A Guide for Community Officials*. More technical information is included within the FEMA publication entitled *Flood Insurance Study Guidelines and Specifications for Study Contractors*. FEMA also publishes standard forms for presenting technical data for LOMAs and LOMRs. In addition, all map amendments and revisions proposed within Pinal County are required to be reviewed and approved by the Arizona Department of Water Resources (ADWR). The engineer engaged in the process of performing map revisions within Pinal County should contact ADWR to obtain current standards and criteria for performing such flood studies.

If construction is proposed on land within a SFHA, a Conditional LOMA or LOMR can be obtained, provided that the proposed structural information meets the established criteria for a standard LOMA or LOMR. After construction is completed, certified "as-built" information must be provided to FEMA for the purpose of obtaining a LOMA or LOMR. The information required for a Conditional LOMA or LOMR is basically the same information that is required for either a LOMA or LOMR. Property owners and developers should note that a Conditional LOMA or LOMR only provides a comment on the proposed plan, and does not amend the map or waive requirements to purchase flood insurance.

FEMA typically charges fees for the review of requests for the various types of map amendments and revisions. These fees can range from a few hundred to a few thousand dollars, depending on the complexity of the request. Since these fees are modified periodically, those engaged in preparing such requests should contact FEMA to obtain a current fee schedule.

1.4.7.2 Construction Within a Designated Floodway

The following criteria are intended to provide guidance to qualified professional engineers when analyzing and certifying proposed encroachments within an adopted regulatory floodway.

The Pinal County Drainage Ordinance and the requirements of the National Flood Insurance Program (NFIP) prohibit encroachments within a regulatory floodway; including fill, new construction, substantial improvements and other development. Encroachments may be permitted if certification by a registered professional engineer is provided demonstrating that the encroachment does not result in any increase in flood levels. This requirement is outlined in Section 302.4 of the Drainage Ordinance and is included in paragraph 60.3(d)(3) of the NFIP regulations.

Because floodway development is contradictory to the tenets of sound floodplain management, such development is discouraged by the Federal Emergency Management Agency (FEMA). Therefore, these certification requirements assume that all practical alternatives to floodway development have been investigated thoroughly and have been deemed not feasible.

1.5 ACRONYMS AND ABBREVIATIONS

The following acronyms and abbreviations are used within the contents of this Manual:

A	Area
ADEQ	Arizona Department of Environmental Quality
ADOT	Arizona Department of Transportation
ARS	Arizona Revised Statutes
BFE	Base Flood Elevation
BMP	Best Management Practices
CAP	Corrugated Aluminum Pipe
CAPA	Corrugated Aluminum Pipe Arch
CLOMA	Conditional Letter of Map Amendment
CLOMR	Conditional Letter of Map Revision
CMP	Corrugated Metal Pipe
CMPA	Corrugated Metal Pipe Arch
CN	Curve Number
CSP	Corrugated Steel Pipe
CSPA	Corrugated Steel Pipe Arch
EGL	Energy Grade Line
FAA	Federal Aviation Administration
ft	feet, foot
FEMA	Federal Emergency Management Agency
FIRM	Flood Insurance Rate Map
fps	feet per second
HDS	Hydraulic Design Series
HEC	Hydraulic Engineering Circular
HERCP	Horizontal Elliptical Reinforced Concrete Pipe
HGL	Hydraulic Grade Line
HOA	Home Owners Association
hr	hour(s)
in	inch(es)
LOMA	Letter of Map Amendment
LOMR	Letter of Map Revision
mi	mile(s)
min	minute(s)
NAVD 88	North American Vertical Datum of 1988
NFIP	National Flood Insurance Program
NGVD 29	National Geodetic Vertical Datum of 1929
NOAA	National Oceanic and Atmospheric Administration
NPDES	National Pollutant Discharge Elimination System
NRCS	National Resources Conservation Service (formerly SCS, Soil Conservation Service)
NURP	Nationwide Urban Runoff Program
NWS	National Weather Service

PE	Professional Engineer (Licensed by the State of Arizona)
PMF	Probable Maximum Flood
PMR	Physical Map Revision
RCBC	Reinforced Concrete Box Culvert
RCP	Reinforced Concrete Pipe
ROW	Right-of-Way
SFHA	Special Flood Hazard Area
SPP	Structural Plate Pipe
SPPA	Structural Plate Pipe Arch
sq mi	square mile(s)
SWPPP	Storm Water Pollution Prevention Plan
t_c	time of concentration
t_i	initial inlet or overland flow time
t_p	time-to-peak
t_t	travel time
USACE	United States Army Corps of Engineers
USBR	United States Bureau of Reclamation
USEPA	United States Environmental Protection Agency
USGS	United States Geological Survey

VOLUME 1 – DESIGN CRITERIA

Chapter 2: Drainage Report and Design Submittal

2.1 INTRODUCTION

As per Pinal County Ordinance No. 100798-DO, An Ordinance to Regulate Drainage in Pinal County, drainage analysis and reports are required for all mobile home parks, recreational vehicle parks, industrial parks and residential subdivisions. The drainage analyses and report will vary with the focus of the project and its size. Based on the conclusion of the report, the County Engineer may require that reasonable modifications be made to the development plans.

The following sections provide the format for drainage analyses and reports depending on the type of development. The appendices, at the end of the chapter, provide checklists and outlines for the drainage reports.

2.2 DRAINAGE REPORT CLEARENCE

Drainage Reports are not required for developments including, but not limited to, single family residence, building additions, utility sheds, and similar uses on a single parcel of land and within low flow areas or adequately protected by up-slope drainage control structures. However, the final determination of whether a Drainage Report is required lies with the County Building Official. If, in the opinion of the County Engineer the conditions require more complete analysis and reporting a Drainage Report of the appropriate type shall be submitted. Refer to Volume 1, Chapter 2 for description of Drainage Report requirements.

In cases where a Drainage Report is not required, a Parcel Drainage Report meeting the requirements in Chapter 2 of this Volume shall be submitted.

If after reviewing the submittal for a Drainage Clearance the County Engineer or staff determines that a more thorough analysis and reporting is required the County Engineer shall notify the applicant that a Drainage Report in accordance with Chapter 2 of this Volume will be required, as well as designate which type of Drainage Report is required.

2.3 ENCROACHMENT OF FLOODPLAIN

2.3.1 Floodplain Analysis

Flood Insurance Rate Maps show delineated floodplain limits and water surface elevations. A floodplain analysis is needed for projects that will encroach within the floodplain, to demonstrate that the policies regulating 100-year floodplains have not been violated. Floodplain analysis is also needed for proposed revisions to Flood Insurance Rate Maps. This section describes the submittal needed to document this work. The methods are covered in Volume 2.

2.3.1.1 Submittal Requirements

Prior to any development occurring in a floodway area, the proposer must demonstrate that a proposed encroachment within the floodway area will not result in any increase in the Base Flood Elevation (BFE). The items identified in Appendix C are deemed necessary (at a minimum) to document and demonstrate compliance with this “no-rise” criterion for simple floodway encroachments. The analysis must be submitted to the County for review and approval along with the Site Drainage Plan or Development Drainage Plan, as appropriate.

2.4 PLANNED DEVELOPMENT

2.4.1 Master Drainage Plan

Reports pertaining to a specific planned development consisting of more than one lot or parcel of land are generally undertaken by the owner or developer of that land. The development may span the boundaries of more than one drainage basin or sub basin, and may include parts of a basin only. The purpose of these analyses is to quantify drainage flows and size facilities to manage stormwater within the County requirements.

A Master Drainage Plan is required for any proposed development that may be undertaken in more than one phase. The Master Drainage Plan must cover all proposed phases of development under the same ownership. Should a developer undertake development of lands adjacent to but not part of a previous Master Drainage Plan the developer will be required to immediately undertake a Master Drainage Plan if one has not been done, or to expand a previously approved Master Drainage Plan to include the new property(s).

2.4.1.1 Report Requirements

The Master Drainage Plan must follow the outline in Appendix B to this Chapter. The checklist in Appendix A is available in an electronic format that can be filled in either on-screen or manually by the user then printed at any time. The checklist is also available in a document format.

2.4.2 Drainage Studies and Plans

Reports pertaining to a specific planned development consisting of more than one lot or parcel of land are generally undertaken by the owner or developer of that land. The development may span the boundaries of more than one drainage basin or sub basin, and may include parts of a basin only. The purpose of these analyses is to quantify drainage flows and size facilities to manage stormwater within the County requirements.

A Development Drainage Study or Plan is required for any development that will be undertaken in a single phase. A Development Drainage Plan is also required for any phase of a project covered under an approved Master Drainage Plan. The difference is in the level of detail and specificity between a Master Drainage Plan and a Development Drainage Plan. A Development Drainage Plan may modify the specifics contained in a previously-approved Master Drainage Plan for the property.

2.4.2.1 Report Requirements

The Development Drainage Study must follow the outline in Appendix B to this Chapter. The checklist in Appendix A is available in an electronic format that can be filled in either on-screen or manually by the user then printed at any time. The checklist is also available in a document format.

2.5 SINGLE PARCEL DEVELOPMENT

2.5.1 Drainage Studies and Plans

Site drainage reports pertaining to a single parcel are generally undertaken by the owner or developer of that parcel or lot. Drainage analyses for single parcels generally need little in the way of detailed studies and analyses. The general drainage principles and goals of the County dictate that the owner of a single parcel shall not change the routing of drainage patterns that existed before development and may not change the point of inflow to or outflow from the parcel or lot**.**

A site drainage report is a short letter type report, which addresses existing and proposed drainage conditions from sites which generally have minor impact on the overall local and regional drainage facilities. The Site Drainage Report documents the existing drainage conditions of the property and presents the details of the proposed drainage system. The Site Drainage Report generally provides sufficient information to obtain a Drainage Clearance.

The Site Drainage Report shall contain a brief narrative letter, the checklist in Appendix A, any supporting material, and a calculation appendix (if required).

2.5.1.1 Report Requirements

The Site Drainage Report shall be submitted accompanied by the checklist found in Appendix A. The checklist is available in an electronic format that can be filled in either on-screen or manually by the user then printed at any time. The checklist is also available in a document format.

2.6 ROADWAY DEVELOPMENT

2.6.1 Drainage Studies and Plans

Reports and analyses pertaining to a road or similar project are generally undertaken by the County (or on its behalf). These projects are not focused on a basin and may cross basin boundaries. These analyses are usually done to size specific drainage facilities associated with the project.

2.6.1.1 Report Requirements

The drainage report shall be completed using the format set forth by Appendix B and submitted accompanied by the checklist found in the Appendix A. The checklist is available in an electronic format that can be filled in either on-screen or manually by the user then printed at any time. The checklist is also available in a document format.

2.7 REVIEW PROCESS

To be provided at a later date

2.8 APPENDICES

2.8.1 Appendix A: Drainage Report/Plan Submittal Checklist

✓				Report outline to be followed
☐	**I**			**Introduction**
☐		A		Project
☐			*1*	*Project Name, Type of Study, Study Date*
☐			*2*	*General Location Map (8½“ x 11” is suggested)*
☐			*3*	*Assessor’s parcel numbers*
☐			*4*	*Township, Range, Section, ¼ Section*
☐		B		Contact info
☐			*1*	*Owner’s name and contact info*
☐			*2*	*Developer’s name and contact info*
☐			*3*	*Engineer’s firm, name, contact info (Submittal must be sealed)*
☐		C		Description of Project
☐			*1*	*Project description*
☐			*2*	*Project size, ac*
☐			*3*	*Existing and proposed land use*
☐			*4*	*Local streets within and adjacent to the project*
☐			*5*	*Rights-of-way and easements within and adjacent to the project*
☐			*6*	*Existing irrigation facilities within and adjacent to the project*
☐			*7*	*Names of surrounding development*
☐			*8*	*Surrounding land uses and zoning*

D Existing Site Conditions

1 *Zoning and land use*

2 *Irrigation facilities within the basin*

3 *Soils classification maps*

4 *Ground cover; type and area of trees, shrubs, vegetation*

5 *Topography and ground slopes*

6 *Existing detention facilities*

II Background

A References to all drainage planning studies associated with or near project

1 *Flood hazard delineation*

2 *Flood insurance rate maps (FIRM)*

3 *Drainage planning reports*

4 *Basin studies*

III Existing and proposed hydrology and hydraulics

A Discuss existing and proposed drainage basin boundaries

B Hydrology

1 *Design rainfall and recurrence intervals*

2 *Runoff calculation method*

3 *Other criteria or methods used*

4 *Discuss existing drainage patterns and areas of inundation (if applicable)*

C Hydraulics

1 *Methods used in performing hydraulic calculations*

2 *Other criteria or methods used*

3 *Present existing and proposed minor and major storm flow calculations (if required)*

IV Proposed drainage facilities

A Discuss routing of flow in and/or around site and location of drainage facilities

B Discuss mitigation measures (if applicable)

C Discuss floodplain modifications (if applicable)

D Present calculations for proposed facilities and typical sections for stormwater conveyance, if applicable

V Conclusions

A Compliance with Manual

B Ability to provide emergency all weather access

C Compliance with Federal Emergency Management Agency (FEMA) (if applicable)

D Discuss effect of development on adjacent properties

1 *Flow rates*

2 *Discharge location*

3 *Discharge velocity*

4 *Inundation limits*

5 *List of facilities required*

VI Exhibits

A Drainage plan

B Watershed maps

☐		C	Cross section location maps
☐	**VII**		**Calculations appendix (if required)**
☐		A	Runoff calculations (existing and proposed)
☐		B	Street and drainage facility capacity calculations, existing and proposed flood limit calculations
☐		C	Detention calculations (if applicable)
☐	**VIII**		**Drainage plan.** **An 8½" x 11" or larger, legible drainage plan that covers the development area bound with the Conceptual Drainage Study. The plan shall contain, as a minimum, the following:**
☐		A	Locate and label development boundary
☐		B	Locate and label adjacent streets
☐		C	Locate and label known 100-year floodplains
☐		D	Locate and label existing and/or planned Pinal County facilities
☐		E	Locate and label existing and/or planned local flood control facilities
☐		F	Show flow paths
☐		G	Identify design inflow points and design outflow points and corresponding minor and major storm flow rates

Note: The drainage plan stated above is preferred; however, multiple exhibits containing the same information may be submitted

2.8.2 Appendix B: Drainage Report format and design submittals

I. GENERAL LOCATION AND DESCRIPTION
   A. Contacts and responsible parties
      1. Owner name and contact info
      2. Developer name and contact info
      3. Engineer's name and contact info
   B. Location
      1. Assessor's Parcel Number(s)
      2. Township, range, section, ¼ section
   C. Description of Property
      1. Project description
      2. Area of the property in acres
      3. Existing and proposed land use
      4. Local streets within and adjacent to the subdivision with right-of-way width shown
      5. Location of major drainageways, drainage facilities, and drainage easements
      6. Location and name of irrigation facilities adjacent to or on the property
      7. Names of surrounding development, land uses, and identification of present zoning

II. DRAINAGE BASINS AND SUB-BASINS
   A. Major Basin Description
      1. References to all drainage planning studies, such as flood hazard delineation reports, drainage planning reports, and flood insurance rate maps
      2. Discussion of existing drainage studies prepared for adjacent projects
      3. Major basin drainage characteristics, existing and planned land uses
      4. Identification of all irrigation facilities within the basin which will influence or be influenced by the local drainage
      5. Soils classification map
      6. Ground cover (type and area of trees, shrubs, vegetation, general soil conditions, topography, and slope)
      7. Identification of all detention facilities
   B. Sub-Basin Description
      1. Discussion of historic drainage patterns for the property in question
      2. Discussion of off-site drainage flow patterns and the impact on development under existing and fully developed basin conditions, using development conditions as defined by the Planning Department

III. DRAINAGE DESIGN CRITERIA

A. Regulations: Discussion of the optional provisions selected or the deviation from this Manual, if any, and its justification
B. Development Criteria and Constraints
   1. Discussion of previous drainage studies (i.e. project master plans) for the site in question that influence or are influenced by the drainage design and how the plan will affect drainage design for the site
   2. Discussion of the drainage impact of site constraints, such as streets, utilities, light rail rapid transit, existing structures and developments
C. Hydrologic Criteria and Results
   1. Identify design rainfall
   2. Identify runoff calculation method
   3. Identify detention discharge/volumes and storage calculation method
   4. Identify design storm recurrence intervals
   5. Discussion and justification of other criteria or calculation methods used that are not presented in or referenced by this Manual
   6. Summary table of pre- and post-development watershed areas and peak discharges for the 2, 10, 25 and 100-year return periods
D. Hydraulic Criteria
   1. Identify references/methodologies used in performing hydraulic analysis
   2. Discussion of other drainage facility design criteria used that are not presented within this Manual
E. Variances from this Manual
   1. Identify provisions by section number for which a variance is requested
   2. Provide justification for each variance requested

IV. DRAINAGE FACILITY DESIGN
A. General Concept
   1. Discussion of existing drainage patterns
   2. Discussion of off-site runoff considerations and compliance with applicable criteria
   3. Discussion of the content of tables, charts, figures, plates or drawings presented in the report
   4. Discussion of anticipated and proposed drainage patterns and/or improvements
   5. Discussion of the stormwater runoff quality aspects of the drainage design including those activities necessary to control erosion and sedimentation during construction
B. Specific Details
   1. Discussion of drainage problems encountered and solutions at specific design points
   2. Discussion of detention storage and outlet design
   3. Discussion of maintenance access and aspects of the design

4. Discussion of easements and tracts for drainage purposes

V. CONCLUSIONS

A. Compliance with Standards

1. Discussion of compliance with Pinal County drainage criteria

B. Drainage Plan

1. Discussion of influence of proposed development on existing drainage conditions
2. Discussion of effectiveness of the drainage design to control damage from storm runoff

VI. REFERENCES

Reference all criteria and technical information used

VII. APPENDICES

A. Hydrologic Computations

1. Land-use assumptions regarding adjacent properties
2. Initial and major storm runoff at specific design points
3. Historic and fully developed (pre- and post-) runoff computations at specific design points
4. Hydrographs at critical design points
5. Time of concentration and runoff coefficients

B. Hydraulic Computations

1. Culvert capacities
2. Storm drain capacities
3. Gutter capacities
4. Storm inlet capacity, including inlet control rating at connection to storm drain
5. Open channel design
6. Roadside ditch capacities
7. Check dam and/or channel drop design
8. Detention area/volume capacity and outlet design, details, and all supporting calculations
9. Depths of detention basins
10. Downstream/outfall system capacity to the major drainageway system

2.8.3 Appendix C: Floodplain Analysis Report Requirements

The floodplain analysis report must contain:

1. Hydraulic backwater models for the 100-year flood and floodway water-surface profiles using the Corps of Engineers HEC-2 or HEC-RAS software. As a minimum, the proposer must submit model runs covering the following cases:

    a) A calibration or test run that duplicates the effective Flood Insurance Study (FIS) model. This calibration run shall include the natural cross-sections and shall extend upstream and downstream far enough to fully evaluate the impact of the proposed development.

    b) An existing conditions model modified to include a minimum of at least three additional field surveyed cross-sections through the proposed project site. These added cross-sections must reflect existing site conditions prior to construction of the project.

    c) A post-project conditions model which includes cross-sections through the proposed project site reflecting floodplain conditions after construction of the proposed project. Refer to Volume 2 for demonstrations needed and methods involved.

    d) If needed for a project, other models needed to describe possible impacts of the proposed development or project or as may be required by the County Engineer.

    e) A copy of the appropriate NFIP maps showing the existing floodway and indicating the project or study area.

    f) Topographic mapping of the entire project area indicating the location of all cross sections used in the modified hydraulic model and a plan view of all project elements. The map elevations shall be tied to the appropriate elevation reference mark(s) shown on the NFIP maps. The plan shall be to a scale of no more than 100 feet per inch (1:1200), and shall display contours with a contour interval of two feet or less.

2. If the floodplain analysis is done for a project or development, the following must also be submitted.

    a) Construction and foundation plans, certified by a registered professional engineer, for all project elements including those measures employed to provide additional effective conveyance.

b) Scour Analysis using at least the criteria given in Chapter 3.

c) Lateral Loading Analysis using at least the criteria given in Chapter 3.

d) Impact Loading Analysis using at least the criteria given in Chapter 3.

e) An executed copy of a Certification Statement signed and sealed by a Arizona Registered Professional Engineer. A blank copy of the Certification Statement is included at the end of this Chapter.

f) Additional analysis may be appropriate and/or required by the County Engineer on a case-by-case basis.

NOTE: The proposer or engineer should discuss proposed encroachments with County Engineer prior to submitting the analysis.

2.8.4 Appendix D: Drainage Drawing Contents

All drawings shall be 24” x 36” in size

A. General Location Map:

Provide a map with sufficient detail to identify drainage flows entering and leaving the development and general drainage patterns. The map should be at a scale of 1” = 1000” to 1” = 2000” and show the path of all drainage from the upper end of any offsite basin to the defined major drainageways. The map should identify any major facilities that can affect drainage (e.g. development, irrigation ditches, existing detention facilities, storm drains) along the entire path of drainage. Identify basins and divides and include topographic contours. USGS Quadrangle maps (7.5-minute) are acceptable.

B. Floodplain Information:

Use the appropriate FEMA FIRM and Floodway Map, if available, to plot the location of the parcel and provide a copy in the report.

C. Drainage Plan:

Include map(s) of the proposed development at a scale of 1” = 20” to 1” = 200” on 24” x 36” sheets. The plan(s) should show the following:

1. Existing (dashed lines) and, if available, proposed (solid line) contours at 2-foot maximum intervals. In terrain where the slope exceeds 15%, the maximum interval can be 10-feet. Show the contours extending a minimum of 100 feet beyond the property lines. Include a benchmark and relate topography to the USGS survey datum or other local floodplain survey datum if applicable. NOTE: USGS Quadrangle maps are not acceptable for this purpose.

2. Property lines and easements, indicating the type of easement.

3. Streets, including right-of-way width, flow-line width, sidewalk, and other pertinent dimensions.

4. Existing and proposed drainage facilities and structures, including irrigation ditches, roadside ditches, drainageways, gutter flow directions and culverts. All pertinent information, such as material, size, shape, slope and location shall also be included.

5. Overall drainage area boundary and drainage sub-area boundaries, both off-site and on-site.

6. Proposed type of street flow (i.e. vertical curb or combination curb and gutter), roadside ditch, gutter slope and flow direction, and valley gutters.

7. Proposed storm drains and open drainageways, including inlets, outlets, manholes, culverts, other appurtenances, and channel protection.

8. Proposed outfall point for runoff from the developed area and drainage facilities to convey flows to the final outfall point without damage to downstream properties.

9. Routing and accumulation of flows at various critical points for the initial storm runoff listed on the drawing.

10. Routing and accumulation of flows at various critical points for the major storm runoff listed on the drawing using the format shown in Table C-1.

11. Volumes, release rates, and locations for detention storage facilities and information on outlet works. This shall include design drawings, consisting of plan views, cross-sections, and details of the basin and the outlet and inlet works.

12. Identify all flood hazard areas (pre and post development, if applicable); detailed delineations (drainage basins greater than 160 acres), approximate delineations (drainage basins between 40 and 160 acres), and drainage paths (drainage basins between 10 and 40 acres.).

13. Location and elevation of all floodplains affecting the property, including detailed delineations.

14. Location and elevation of all existing and proposed utilities affected by or affecting the drainage design.

15. Identification of drainage patterns through the development.

16. Definition of flow path leaving the development through the downstream properties ending at a major drainageway.

17. Legend to define map symbols.

18. Title block in lower right hand corner.

19. Location of stormwater pollution prevention activities and identify methods of controlling erosion and sedimentation during grading and construction phase(s).

Table C- 1: Data layout for drainage data

<table>
<tr><th rowspan="3">Design Point</th><th colspan="5">Pre-Development</th><th colspan="5">Post-Development</th></tr>
<tr><th rowspan="2">Drainage Area</th><th colspan="4">Peak Flow, cfs</th><th rowspan="2">Drainage Area</th><th colspan="4">Peak Flow, cfs</th></tr>
<tr><th>2-yr</th><th>10-yr</th><th>25-yr</th><th>100-yr</th><th>2-yr</th><th>10-yr</th><th>25-yr</th><th>100-yr</th></tr>
<tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
</table>

VOLUME 1 – DEISGN CRITERIA

Chapter 3: Design Criteria

3.1 INTRODUCTION

The following sections in this chapter are arranged to correlate with the chapters in Volume II of this manual. For instance, Chapter 2 in Volume II covers the same material as Section 3.2 of Volume I, Chapter 3 in Volume II covers the same material as Section 3.3 of Volume I, etc. Section 9 and 10 cover information that is considered pertinent to drainage design but does not require a complete chapter to describe design procedure or application of the design.

3.1.1 Conventions Used

3.1.1.1 Slope Ratios

Slope ratios used in this Manual are shown in H:V format and are to be interpreted as the ratio of horizontal to vertical unless otherwise specified.

3.2 HYDROLOGY

3.2.1 Storm Frequency

The rainfall event, based upon the 100-year duration, which generates the peak discharge for the area contributing runoff to the development, shall be used in designing the overall development drainage system. This may vary by watershed, but is often the 100-year, 6 hour event.

3.2.2 Floodplain Delineation

The 100-year runoff, using the duration that will result in the largest discharge, will be used to delineate a floodplain for major channels with discharges of more than 500 cfs and will be processed through the local government, ADWR, and FEMA.

3.3 STREET DRAINAGE

3.3.1 General

To facilitate drainage, the design of roadways and subdivision streets that may be in or are proposed for future inclusion into the Pinal County Highway Maintenance System shall conform to Pinal County standards for roadways.

Adequate drainage ways shall be constructed to convey the street design flow if that flow is designed to leave the public right-of-way. Such drainage ways shall be platted as drainage easements or as separate tracts with maintenance provisions designated.

A Manning's "n" value of 0.015 (normal asphalt) or 0.016 (rough asphalt) shall be used for street flow unless special conditions exist. Such special conditions must be clearly documented in the Drainage Design Report.

The County Engineer may require construction of a culvert or bridge where a road crosses a natural drainage way. The size of a culvert or bridge shall conform to the requirements set forth in this chapter.

If roads are designed to convey runoff, the amount conveyed shall not exceed design standards stated below. Excess flow shall be conveyed in drainage ways if the design standards would be exceeded.

To prevent back and head cutting, dip sections and culvert crossings of rights-of-way shall have adequate cutoff walls or aprons constructed of non-erodible material.

Each site shall have one all-weather road access with a maximum flow of eight inches in depth crossing over the roadway at the culvert or overflow section during the 25-year peak flow event, with no adverse backwater effect during a 100-year peak flow event.

The County Engineer in evaluating various backwater conditions, may allow some roads to be constructed which result in substantial overtopping of the roadway surface provided that the velocity of flow or hydraulic features prevent roadway erosion and meet the requirements in this Volume.

3.3.2 Streets and Gutters

3.3.2.1 Gutter Spread

Drainage flowing along streets may not encroach more than the width of a lane from either side.

3.3.2.2 Longitudinal Street Grades

The desirable minimum longitudinal street grade is 0.4% to ensure that the gutters will function properly, and all streets should be designed to meet or exceed this grade. In situation where this minimum grade is not attainable, for instance with projects involving existing streets, the absolute minimum longitudinal street grade shall be 0.2% or greater.

The maximum street grades should not exceed 10%.

Additionally, when planning street grades, emergency vehicle access needs to be considered in the design.

Any slope less than 0.4% or greater than 10% needs specific review and approval by County staff.

3.3.3 Conveyance

3.3.3.1 Roadway Conveyance

When designing stormwater conveyance within roadways the following criteria shall be met:

1) The 10-year runoff shall be carried within the curbs

2) The 100-year runoff may be carried within the right-of-way provided that flow depths do not exceed six inches above the center line

Flows which would cause those limits to be exceeded shall be diverted to drainageways specifically designed for that purpose.

3.3.3.2 Valley Gutters

When local streets intersect arterial or collector streets, the grades of the arterial or collector should be continued uninterrupted. When collector and arterial streets intersect, the grade of the more major street should be maintained as much as possible.

No form of valley gutter for drainage purposes should be constructed across an arterial street. Occasionally, with agency approval, valley gutters may be considered on collector streets.

Conventional valley gutters may be used to transport runoff across local streets when a storm drain system is not required and when approved by the governmental agency. The valley gutter should be sufficient to transport the runoff across the intersection with encroachment equivalent to that allowed on the street.

3.3.3.3 Roadside Ditches

Roadside ditches are commonly used in rural areas to convey runoff from the highway pavement, and from areas that drain toward the highway. Where practicable, the flow from major areas draining toward curbed highway pavements should be intercepted by ditches. The following criteria pertain to the design of open channels along roadsides. For additional criteria for open channels, see the section further on in this chapter.

1) Roadside ditches adjacent to public streets are discouraged in urban areas and require approval from the County Engineer. When they are allowed, adhere to the criteria outlined in this section.

2) Depth of flow in roadside ditches for the design storm shall be limited to preclude saturation of the adjacent roadway subgrade.

3) Where curbs exist and roadside ditches are used in lieu of storm drains, catch basins or scuppers should be provided as needed to drain the pavement into the drainage ditch.

4) Geometric considerations in the design of channel cross sections should incorporate hydraulic requirements for the design discharge, safety, minimizing right-of-way acquisition, economy in construction and maintenance, and good appearance. Channel side slopes should be as mild as practical and should be no steeper than 4:1 where terrain and right-of-way permit. The advantages of mild slopes are that the potential for erosion and slides is lessened, the cost of maintenance is reduced, and the safety of errant vehicles is enhanced.

5) Trapezoidal channel bottoms should be a minimum of 4 feet wide for maintenance purposes. V-shaped channels may also be used when approved by the County Engineer.

6) Local soil conditions, flow depths, and velocities within the channel are usually the primary hydraulic considerations in channel geometric design; however, terrain and safety considerations have considerable influence. Steeper side slopes of rigid, lined channels may be more economical and will improve the hydraulic flow characteristics. The use of steeper slopes is normally limited to areas with limited right-of-way where the hazard to traffic can be minimized through the use of guardrails or parapets.

3.3.3.4 Rural Crown Ditch (Interceptor Ditch)

In mountainous terrain where large cuts are required, crown ditches constructed on top of the cut embankment will intercept runoff preventing it from eroding the face of the cut slope. Drainage captured by crown ditches must be conveyed to natural drainage or to the drainage system associated with the street.

3.4 STORM DRAINS AND CATCH BASINS

In this manual, a storm drain system refers to a coordinated group of inlets, underground conduits, manholes, and various other appurtenances that are designed to collect stormwater runoff from the design storm and convey to a point of discharge into a major or regional drain outfall. The size of a storm drain system is based on a designated design storm.

Storm drains should generally only be considered for minor watercourses. Storm drains typically are not economical for the flows conveyed within larger watercourses. Typically, the storm drain system will collect and convey runoff to a point where storm drains become too large to be economical and will then discharge into a major or regional watercourse outfall consisting of a man-made channel, or natural watercourse.

3.4.1 Storm Drain Materials

All materials used for a storm drain system must be approved by the County Engineer prior to use.

3.4.2 Hydraulic Grade Line

Storm drain systems shall be designed so that the hydraulic grade line is at least six inches below the inlet elevation.

3.4.3 Inlet and Outlet Design

3.4.3.1 Inlets

In general, the interception of flow from a natural watercourse directly into a storm drain system should be avoided. If avoiding this situation is not possible, then an inlet structure should be provided. Strong consideration should be given to the use of a debris or sediment basin upstream of the inlet structure. The inlet structure should generally consist of a headwall, wingwalls to protect the adjacent banks from erosion, and a paved inlet apron. Wall heights should conform to the height of the water upstream of the inlet, and should be adequate to protect both the fill over the drain and the embankments. Headwall and wingwall fencing, an access barrier, and a trash rack should be considered to promote public safety.

See Chapter 3.9.11 for more information on safety and fencing.

3.4.3.2 Outlets

When a storm drain outlets into a natural channel, an outlet structure must be provided that prevents erosion and property damage. Velocity of flow at the outlet should agree as closely as possible with the existing channel velocity.

1) When the discharge velocity is low or subcritical, the outlet structure should consist of a concrete headwall, wingwalls, and an apron. See velocity tolerances for unlined and grass lined channels in this chapter.

2) When the discharge velocity is high or supercritical, the designer should also consider adding bank protection in the vicinity of the outlet and an energy dissipater structure.

See Volume 2, Chapters 5 and 7 for additional information concerning conduit outlet structures.

Where practical the outlet of the storm drain should be positioned in the outfall channel so that it is pointed in a downstream direction. This will reduce excessive flow disturbance and the potential for excessive erosion. If the outfall structures cannot be oriented in a downstream direction, the potential for outlet scour must be considered. For example, where a storm drain outfall discharges perpendicular to the direction of flow of the receiving channel consideration should be given to the possibility of erosion on the opposite channel bank. If erosion potential exists, a channel bank lining of riprap or other suitable material should be installed in the bank. Alternatively an energy dissipater structure could be used at the storm drain outlet.

3.4.3.3 Trash Racks and Access Barriers

An access barrier is a device for preventing people and animals from entering storm drain pipes. Protection barriers may consist of large, heavy breakaway gates, single horizontal bars across catch basin openings, or fencing around an exposed inlet or outlet.

Chapter 3.9.11 overviews safety related considerations for drainage structures including storm drains. In some areas, there may be a high potential for debris to enter a storm drain that could block it. In these situations, a trash rack on the open inlet end of a storm drain pipe may be helpful.

3.4.4 Pipes

3.4.4.1 Junction Considerations

A lateral pipe entering a main line pipe storm drain generally should be connected radially (spring line to spring line).

A lateral pipe entering a main line box structure should conform to the following:

1) A lateral pipe 24 inches or less in diameter should enter no more than 5 feet above the invert.

2) A lateral pipe 27 inches or larger in diameter should enter no more than 18 inches above the invert, with the exception that a catch basin connector pipe less than 50 feet in length may be no more than 5 feet above the invert.

Exceptions to the above requirements may be permitted where it can be shown that the cost of bringing laterals into a main line box conduit in conformance with the above requirements would be excessive.

3.4.4.2 Transition from Large to Small Conduit

As a general rule, storm drains are designed with sizes increasing in the downstream direction. However, when studies indicate it may be advisable to decrease the size of a downstream section, the conduit may be decreased in size in accordance with the following limitations:

1) For slopes of 0.0025 ft/ft (0.25 percent) or less, only conduits 78 inches and greater may be decreased in size a maximum of 6 inches.

2) For slopes of more than 0.0025 ft/ft, only conduits 36 inches and greater may be decreased in size. Each reduction should be limited to a maximum of 6 inches for pipe larger than 48 inches in diameter. Reductions exceeding the above criteria should be approved by the County Engineer.

The pipe size reductions should include approved transition, result in a more economical system and not cause any adverse impacts.

3.4.5 Manhole Design

A manhole is generally placed in a storm drain system at locations where the pipe size, slope or horizontal alignment changes, at pipe intersections, and at other periodic locations to provide access to the system for maintenance.

Headloss through manholes must be taken into account in the hydraulic design. This can be especially important if the conduit can operate as a pressure conduit.

In order to maintain hydraulic efficency at a manhole, pipes of different sizes entering and exiting a manhole should be positioned vertically so that their crowns are aligned.

A horizontal offset in the inlet and outlet pipe alignments is allowable provided the projected area of the smaller pipe falls within that of the larger.

If two lateral pipes are aligned opposite each other such that the outflows impinge directly upon each other installation of a deflector can result in significantly reduced losses.

Where possible, lateral pipes entering a manhole should be located vertically higher than the main pipes entering and exiting the manhole

3.5 CULVERTS, BRIDGES AND AT-GRADE CROSSINGS

Culverts and bridges are structures that convey stormwater under roads. Their purpose is to prevent water from the more frequent storm events from overtopping and crossing the road as such conditions inhibit safe passage of vehicles.

3.5.1 Culverts

3.5.1.1 Safety

Culverts shall be designed to conform to the safety protocols identified in Volume I Chapter 1.

3.5.1.2 Sizing

Minimum culvert sizing shall be in accordance with the appropriate jurisdictional standards.

For Pinal County, minimum culvert sizes are as follows:

Table 3- 1: Pinal County Minimum Culvert Sizes

Culvert Type	Minimum Size
Cross Drain	18”
Median Drain	15” *
Side Drain	15” *
Box Culvert (Precast)	3’ x 3’
Box Culvert (Cast in Place)	4’ x 4’
Drains from inlets on high fills (e.g., gutter drains)	15” **

* Some locations require 18” minimum. Verify project specific requirements with the District Drainage Engineer.
** When debris control is not provided by grates, use 18” minimum.

For culverts requiring more than two parallel pipes, other alternatives shall be investigated.

Culverts for collector and arterial streets are to be designed to convey at least the 50-year peak discharge with no flow crossing over the roadway.

Culverts for collector and arterial streets are to be designed to convey at least the 50-year peak discharge with no flow crossing over the roadway.

3.5.1.3 Materials

The selection of a culvert material may depend upon structural strength, hydraulic roughness, durability, and corrosion and abrasion resistance. The culvert materials that should be considered are concrete (reinforced and nonreinforced), corrugated aluminum, corrugated steel, and PVC. Culverts may also be lined with other materials to inhibit corrosion and abrasion. Linings are not recommended to reduce hydraulic resistance because culvert linings have a short life span and are seldom reapplied as part of normal culvert maintenance. When linings are applied, the culvert sizing should neglect the reduced roughness from the lining material.

Manning's Roughness Coefficients

Standard values for Manning's roughness coefficient are as follows:

Table 3- 2: Culvert Pipe Roughness Factors

Pipe Type	Roughness Factor (n)
Concrete Box Culverts	0.012
Concrete Pipes	0.012
Metal Pipes*:*	
Pipe and Pipe Arch - Helical Fabrication	
*Re-corrugated Ends - All Flow Conditions**	
12" – 24"	0.020
30" – 54"	0.022
60" and larger	0.024
Pipe and Pipe Arch - Spiral Rib Fabrication	
*Re-corrugated Ends - All Flow Conditions**	
All Sizes	0.012
Structural Plate - Pipe and Pipe Arch	
*Annular Fabrication - All Flow Conditions**	
All - 6 x 2	0.033
All - 9 x 2-1/2	0.034
Plastic Pipes:	
Polyvinyl Chloride-PVC (external rib/smooth internal)	
All Sizes	0.012
Polyethylene	
Single Wall	0.024
Double Wall (Smooth)	0.012

3.5.1.4 Velocity

Minimum Velocity

Culverts should be designed to provide adequate velocity to self-clean during partial depth flow events. Debo and Reese (1995) suggest a minimum velocity of 2.5 feet per second for partial flow depths. Greater velocities are recommended for installations where sediment loads are heavy. Alternatively, a sediment trap can be utilized where culvert velocities are lower or excessive sediment deposition is expected.

Maximum Velocity

As a practical limit, outlet velocities should be kept below 15 feet per second unless special conditions exist. The maximum velocity should be consistent with channel stability requirements at the culvert outlet. If culvert outlet velocities exceed permissible velocities for the outlet channel lining material, suitable outlet protection must be provided. Outlet velocities may exceed permissible downstream channel velocities by up to 10 percent without providing outlet protection if the culvert tailwater depth is greater than the culvert critical depth of flow under design flow conditions. Table 3-4 in Chapter 3.6.3.2 outlines the permissible velocities for several channel lining materials.

3.5.1.5 Minimum Cover

Minimum cover of fill over culverts must be provided to maintain the structural integrity of the structure under anticipated loading conditions. Culvert manufacturers provide minimum cover requirements for prefabricated pipe. A general rule of thumb for estimating minimum cover requirements is to provide a minimum of 1 foot.

The top of culverts should not extend into the roadway subgrade. Minimum cover should be measured from the top of subgrade, which is the bottom of the pavement structural section.

3.5.1.6 Depth for Road Crossing

.The flow depth over the roadway shall be limited to 0.8 feet for the 100-year peak discharge. Regardless of the size of the culvert, street crossings shall be designed to convey the 100-year storm runoff under and/or over the road to an area downstream of the crossing to which the flow would have gone in the absence of the street crossing. Flows up to and including the 100-year frequency event should not cause increased flooding to adjacent property or buildings, unless a drainage easement is acquired for those areas.

The ponded headwater elevation should be delineated on a contour map, or by other surveying and mapping methods to identify the area inundated by the ponded water.

In general, dip sections are not recommended. However, for flows crossing broad shallow washes where the construction of a culvert is not practical, the road may be dipped to allow the entire flow to cross the road. Use of dip sections for specific, individual cases must be approved by the County Engineer. The pavement through the dip section should be concrete and should have a one way slope in the direction of flow with curbs and medians flush with the pavement. Upstream and downstream cutoff walls and aprons should be provided to minimize the effects of headcutting and erosion.

3.5.1.7 Scour and Sedimentation

Possible aggradation or degradation at culvert crossings must be examined during the design of culverts. An ideal culvert design should pass drainage water through it without upsetting the delicate balance between hydraulics and sediment transport.

An effective culvert design should minimize scour and deposition. To minimize sedimentation problems, inlets should not be depressed below the natural channel flowline.

In addition, multi-barrel culvert installations tend to reduce the channel velocity, particularly in low flow situations. Where multi-barrel installations are necessary, provisions should be made to handle sedimentation with minimal maintenance.

3.5.1.8 Skewed Channels

A good culvert design is one that limits the hydraulic and environmental stress placed on an existing natural watercourse. This stress can be minimized by designing a culvert that closely conforms to the natural stream in alignment and grade. Often the culvert barrel must be skewed with respect to the roadway centerline to accomplish this goal. The alignment of a culvert barrel with respect to a line perpendicular to the roadway centerline at the point of crossing is referred to as the barrel skew angle. A culvert aligned normal to the road-way centerline has a zero skew angle. Directions (right or left) must accompany the barrel skew angle.

Some advantages of following a natural stream alignment include: reduction of entrance losses, equal depths of scour at the footings, less sedimentation, and less excavation for installation. The angle from the culvert face to a line normal to the culvert barrel is referred to as the inlet skew angle.

The structural integrity of circular sections is compromised when the inlet is skewed due to the loss of a portion of the full circular section where the culvert barrel extends beyond the full section. Although concrete headwalls help stabilize the pipe section, structural considerations should not be overlooked in the design of skewed inlets.

Culverts which have a barrel skew angle often have an inlet skew angle as well. This is because headwalls are generally constructed parallel with a roadway centerline to avoid warping of the embankment fill. In cases where the culvert barrel cannot be aligned with the channel flowline, such as when runoff is directed along a roadway embankment to a suitable crossing location, the flow enters the culvert barrel at an angle. The approach angle should be limited to a maximum of 90 degrees.

When high velocities exist, inlet losses resulting from turning the flow into the culvert should be considered. The loss should be added to the other inlet losses in the culvert design computation, if they aren't included in the appropriate nomographs.

In cases where the culvert barrel cannot be aligned with the channel flowline, such as when runoff is directed along a roadway embankment to a suitable crossing location, the flow enters the culvert barrel at an angle. The approach angle should be limited to a maximum of 90 degrees.

3.5.1.9 Bends

A straight culvert alignment is desirable to avoid clogging, to reduce construction costs, and to increase hydraulic efficiency. However, site conditions may require either a horizontal or vertical change of alignment. Particular attention should be given to erosion, sedimentation, and debris control when considering a nonlinear culvert alignment.

Vertical bends are permitted when they transition from a flatter to a steeper slope, but should not transition from steeper to flatter slopes because of the potential for sediment deposition in the flatter reach.

The energy losses due to bends in the culvert must be considered when designing a nonlinear culvert. If the culvert operates in inlet control, no increase in headwater occurs unless the bend losses cause the culvert to flow under outlet control. An increase in energy losses and headwater will result due to the bend losses if the culvert operates in outlet control. To minimize these losses, the culvert should be curved or have bends not exceeding 15 degrees at intervals of not less than 50 feet. Bend losses can be ignored under these conditions. Analysis of bend losses is required if these conditions cannot be met.

3.5.1.10 Junctions

Flow from two or more separate culverts or stormdrains may be combined at a junction into a single culvert barrel. For example, a tributary and a main stream intersecting at a roadway crossing can be accommodated by a culvert junction.

Loss of head may be important in the hydraulic design of a culvert containing a junction. Attention should be given to streamlining the junction to minimize turbulence and head loss. Also, timing of peak flows from the two branches should be considered in analyzing flow conditions and control. When possible, the tributary flow should be released downstream of the culvert barrel.

3.5.1.11 Inlet and Outlet Design

Inlets

The hydraulic performance of culverts operating in inlet control can be improved by changing the inlet geometry of the headwall. Improvements include bevel-edged, side-tapered, and slope-tapered inlets. The advantage of these improvements is to convert an inlet control culvert closer to outlet control by using more of the barrel capacity. A beveled-edge provides a decrease in flow contraction losses at the inlet and the entrance loss coefficient is normally improved and can increase the culvert capacity by as much as 20 percent.

Outlets

The receiving channel at culvert outlets must be protected from high culvert outlet velocities caused by the flow constriction that is inherent in culvert operation. If the culvert outlet velocity is greater than the allowable velocity for the receiving channel, protective measures must be provided. Projecting culvert outlets are not permitted unless approved by the County Engineer. The minimum requirement is to provide a preformed metal or concrete end section, or a headwall (with or without a wingwall configuration) with a cutoff wall provided at the end of the apron.

3.5.1.12 Trash Racks and Access Barriers

When any of the following conditions are met, trash racks will be required on the entrances.

- When a conduit outfalls into a channel with side slopes steeper than 4:1 for concrete, grass and earth linings, and 3:1 for riprap linings.
- Conduits smaller than 7 feet in diameter, longer than 100 feet in length, or without 12 inches of freeboard at the design flow rate.
- Conduits with energy dissipaters at the end.
- Conduits being used as outlets from multiple use detention facilities
- Conduits with sufficient bend that the opposite end cannot be clearly seen.

It is not necessary to include a head loss for the trashrack with approach velocities less than 3 feet per second. Such computations are required for velocities greater than 3 feet per second.

A plugging factor of 50 percent shall be used for all trashrack analysis. For maximum headloss, 1/2 of the net area between the bars shall be considered blocked. This will result in twice the velocity through the trashrack. For detention basin and dam outlet works analysis, trashrack headloss shall be calculated for the plugged condition as well as the unplugged condition.

The trashrack/access barrier assembly shall be hinged or removable to allow access to the outlet construction. The screen shall be fabricated of a minimum of ½ inch x 2 inch flat steel bars or larger designed to withstand the hydrostatic load resulting from the 100-year design ponding with screen openings blocked. Attachment points shall be cast in the headwall concrete and anchored by substantial anchor bolts. Shear pins shall be in 1/8 inch, 3/16 inch or 1/4 inch 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 antivortex device should be included with the trashrack design if vortices are anticipated which could affect hydraulic efficiency and cause erosion of adjacent earth slopes.

3.5.1.13 Flotation and Anchorage

Flotation is the term used to describe the failure of a culvert due to the uplift forces caused by buoyancy. The buoyant force is produced from a combination of high head on the outside of the inlet and the large region of low pressure on the inside of the inlet caused by flow separation. As a result, a large bending moment is exerted on the end of the culvert. This problem has been noted in the case of culverts under high head, with shallow cover, on steep slopes, and with projecting inlets. The phenomenon can also be caused by debris blocking the culvert end or by damage to the inlet. The resulting uplift may cause the inlet ends of the barrel to rise and bend. Occasionally, the uplift force is great enough to dislodge the embankment. Generally, flexible barrel materials are more vulnerable to failure of this type because of their lightweight and lack of resistance to longitudinal bending. Large, projecting, or mitered corrugated metal culverts are the most susceptible. A number of precautions can be taken by the designer to guard against flotation. Steep slopes (1 to 1 or steeper) of adequate height, which are protected against erosion by slope paving or headwalls, help inlet and outlet stability. When embankment fill heights are less than 1.5 times the pipe diameter or fill slopes are flatter than 1 to 1, the designer may consider other applications such as concrete encasement, concrete headwalls, and tie bars to guard against failures caused by flotation. Limiting headwater buildup also helps prevent flotation. It is desirable to limit design headwater depths to 1.5 times the culvert height.

3.5.1.14 Interaction with Other Systems

Closed conduit inlets and outlets provide transitions from a ponded or channelized condition upstream into the closed conduit and then back to a natural or channelized condition down-stream. Additional channel bank protection may be required in the vicinity of the inlet or outlet to complete the transition to the design velocity and flow depth of the receiving channel. The design of inlets and outlets should take into account all conditions in the upstream and downstream direction to the location where the inlet, outlet, and closed conduit have no effect on pre-design flow conditions.

When an open channel or stormwater storage basin drains into a stormdrain system, culvert type inlets are frequently used. The stormdrain hydraulic grade line must be considered when estimating the inlet capacity for culvert type inlets. The stormdrain hydraulic grade line at the inlet, with the appropriate entrance loss added, should be substituted for the outlet control headwater elevation normally used for outlet control computations. To determine the controlling headwater, the computed outlet control headwater elevation should be compared with the inlet control headwater elevation obtained from the standard inlet control nomograph.

3.5.1.15 Special Criteria for Closed Conduits Bank Protection

Roadway embankments with culverts passing through them should be protected from potential damage caused by roadway overtopping during a runoff event in excess of the culvert design capacity. When a planned flow over the road has damage potential, such as when the 100-year discharge causes flow over the roadway, the embankment for both upstream and downstream sides may need to be protected by use of paving, grouted riprap, or other means of permanent stabilization.

3.5.2 Inverted Siphons

Inverted siphons are rarely used in urban drainage and should be avoided where possible. Due to the flat topography and a large number of canals in Pinal County, however, there are possibilities that the designer may have to consider using an inverted siphon. Inverted siphons are used to convey water by gravity under canals, roads, railroads, other structures, and depressions.

An inverted siphon is a closed conduit designed to run full and under pressure. When flowing at design capacity, the structure should operate without excess head.

For canal structures, inverted siphons are economical, easily designed and built, and have proven to be a reliable means of water conveyance. However, because of sediment and debris present in stormwater, maintenance can be a significant negative factor. In addition, canals run more or less continually and can be drained between periods of use, but inverted siphons for stormwater do not operate on a regular cycle. If water is left to stand, significant health hazards could result. Inverted siphons shall be considered only when absolutely necessary, and permitted by the jurisdictional agency. The use of siphons should adhere to the following:

1. All pipes should be designed for watertight joints. Velocity in the conduit should be a minimum of 5.0 ft/sec to prevent sedimentation.

2. The cover over the conduit should exceed the minimum cover necessary to meet its loading classification. Inlet and outlet structures are required, and the facility shall meet the requirements for safety described in Chapter 1.

3. Pipe collars and blow-off structures may be required as determined by the jurisdictional agency. Air vents should be used unless the agency agrees with eliminating the vents.

3.5.3 Bridges

Bridges shall be designed using ADOT Bridge Criteria. Under ADOT's criteria, any battery of culverts with an aggregate width greater than 20 feet is considered a bridge and must be designed using bridge criteria. Quoting from the ADOT Bridge Design Guidelines:

> "A 'bridge' is defined as a structure including supports erected over a depression or an obstruction, as water, highway or railway and having a track or passageway for carrying traffic or other moving loads and having an opening measured along the center of the roadway of more than 20 feet between undercopings of abutments or springlines of arches or extreme ends of openings for multiple boxes; it may include multiple pipes, where the clear distance between openings is less than half of the smaller contiguous opening."

In general, bridges should be designed to have as little effect as possible upon the flow passing beneath them. If possible, bridges over natural or man-made channels should be designed so that there is no disturbance to the flow whatsoever.

A new or replacement bridge should not be permitted to create a rise in the existing water surface elevation, to cause an increase in lateral extent of the floodplain, or to otherwise worsen existing conditions for discharges up to and including the 100-year discharge, unless appropriate measures are taken to mitigate the effects of such increases.

3.5.3.1 Freeboard

Bridges should be designed to have a minimum freeboard of two feet for the 100-year event. The structural design of the bridge should take into account the possibility of debris and/or flows impacting the bridge. In general, variances to the minimum freeboard requirement will be evaluated on a case-by-case basis by the jurisdictional agency.

3.5.3.2 Supercritical Flow

For the special condition of supercritical flow within a lined channel, the bridge structure should not affect the flow at all. That is, there should be no projections, piers, etc, in the channel area. The bridge opening should be clear and permit the flow to pass unimpeded and unchanged in cross section.

3.5.3.3 Piers and Scour

Whenever piers are used, they need to be oriented parallel with flow. Impacts upon channels and floodplains created by bridges usually take the form of increased flow velocities through and downstream of the bridges, increased scour and upstream ponding due to backwater effects. These impacts can cause flood damage to the channel, to adjacent property and to the bridge structure itself.

3.6 OPEN CHANNELS

An open channel is a conveyance system in which water flows with a free surface at the water atmosphere interface. The channel may be either a natural watercourse or an artificial, “engineered” conveyance. Natural streams typically consist of a main flow channel, often termed the thalweg, and adjacent floodplains. Artificial channels are used for a wide variety of applications varying in scale from modest roadside ditches to large conveyance facilities that can be up to several hundred feet wide. Design guides are provided for the analysis of both natural and engineered channel.

3.6.1 Flow Condition

The state of open channel flow is governed by the effects of viscosity and gravity relative to the inertial forces of the flow. The effect of gravity on the state of flow is represented by a ratio of inertial forces to gravity forces called the Froude number (Fr). Refer to Chapter 6 in Volume II for equation to calculate Froude number.

3.6.1.1 Critical Flow

When Fr is equal to 1, the flow is in the critical state. This flow condition is unstable and flow depths at or near critical depth should be avoided. If Fr falls between 0.86 and 1.13 the situation could easily change so that flow will be stable, so that range must be avoided by design.

3.6.1.2 Sub-critical Flow

If Fr is less than 1, the flow is subcritical and gravity forces dominate. Subcritical flows have the following general characteristics relative to critical depth:

- Slower velocities
- Greater depths
- Lower hydraulic losses
- Less erosive power
- Less sediment carrying capacity
- Behavior easily described by relatively simple mathematical equations
- Surface waves propagate upstream.

Subcritical flow can generally be handled in channels without linings because erosive velocities are avoided.

3.6.1.3 Supercritical Flow

If Fr is greater than 1, the flow is supercritical and inertial forces predominate. Supercritical flows have the following general characteristics relative to critical depth:

- Higher velocities
- Shallower depths
- Higher hydraulic losses
- More erosive power
- More sediment carrying capacity
- With few exceptions, behavior can't be easily predicted mathematically
- Surface waves propagate downstream only

Supercritical flow in an open channel in an urbanized area creates certain hazards that the designer must take into consideration. From a practical standpoint it is generally unwise to have any curvature in a supercritical channel. Careful attention must be taken to prevent or control excessive oscillatory waves that may extend the entire length of the channel from only minor obstructions upstream. For channels carrying supercritical flow, there shall be no reduction in cross sectional area at bridges or culverts and no obstructions in the flow path. Imperfections at joints in a channel lining may rapidly cause a deterioration of the joints, in which case a complete failure of the channel can readily occur. High velocity flow can enter cracks or joints and create uplift forces by the conversion of velocity head to pressure head causing damage to the channel lining. When designing a lined channel with supercritical flow the designer must use utmost care and consider all relevant factors.

Supercritical flow requires a smaller channel cross section to carry the same flow rate.

3.6.2 Roughness Coefficients

Roughness coefficients (Manning's n-values) vary considerably according to depth of flow, and type and quality of the surface material. Estimates of n-values should include consideration that roughness may vary with flood stage, depending on such factors as the width-depth ratio of the watercourse; presence of vegetation in the main channel; the types of materials making up the channel bed; and the degree of meandering. Additional information concerning Manning's roughness coefficients can be found in Phillips and Ingersoll (1998), Thomsen and Hjalmarson (1991), Davidian (1984), Aldridge and Garret (1973) and Barnes (1967).

Typical values of roughness coefficients are given in Table 3-3. For each material and/or construction method listed, three possible values of n are given. These values should be interpreted as follows:

- Minimum = new construction
- Normal = good maintenance
- Maximum = deteriorated and/or poor maintenance.

Channels designed using the maximum n-value result in a greater flow depth. Using the minimum n-value results in a greater velocity of flow in the channel.

Hydraulic capacity of a channel should be based upon the maximum n-value anticipated during the life of the structure.

Maximum expected channel velocity should be a consideration when analyzing supercritical flow, hydraulic jumps, and forces on structures, among others. Flows that are sufficient to damage vegetation also reduce resistance to flow. Using the minimum n-value results in higher computed velocities.

Both maximum and minimum n-values should be considered in the design of channels to check for sufficient hydraulic capacity and stability of channel linings, respectively.

Table 3- 3: Manning's Roughness Coefficients

Channel Material	**Roughness Coefficient (n)**		
	Minimum	**Normal**	**Maximum**
Concrete:			
Trowel finish	0.011	0.013	0.015
Float finish	0.013	0.015	0.016
Unfinished	0.014	0.017	0.020
Shotcrete, good section	0.016	0.019	0.023
Shotcrete, wavy section	0.018	0.022	0.025
Soil cement	0.018	0.020	0.025
Constructed channels with earthen bed			
Clean earth; straight	0.018	0.022	0.025
Earth with grass and forbs	0.020	0.025	0.030
Earth with sparse trees and shrubs	0.024	0.032	0.040
Shotcrete	0.018	0.022	0.025
Soil cement	0.022	0.025	0.028
Concrete	0.017	0.020	0.024
Riprap	0.023	0.032	0.036

3.6.3 Design Velocities

Design velocities for all linings should not fall below 2 fps so that sediment deposition is minimized.

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

3.6.3.1 Minimum Velocity

Very low velocities encourage sedimentation and undesirable plant growth, which decreases channel carrying capacity and promotes nuisance ponding. Channels must be designed with respect to sedimentation issues elaborated in Chapter 7 in Volume II.

3.6.3.2 Maximum Velocity

For earthen or grass lined channels, maximum permissible velocities should be governed by Table 3-4 and Table 3-5, respectively. If the natural channel slope would cause excessive velocity, employ drop structures, checks, riprap (USDOT, FHWA HEC-11), or other suitable velocity control design features .

Table 3- 4: Max Permissable Velocities for Roadside Drainage Channels with Erodible Linings

Soil types in lining (No vegetation; plain earth)	Permissable velocity, ft/sec
Fine sand (noncolloidal)	2.5
Sandy loam (noncolloidal)	2.5
Silt loam (noncolloidal)	3.0
Ordinary fine 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 soils (noncolloidal)	3.5
Alluvial soils (colloidal)	5.0
Coarse gravel (noncolloidal)	6.0
Cobbles and shingles	5.5
Shales and hardpans	6.0

Table 3- 5: Max Permissable Velocities for Roadside Channels with Uniform Stand of Grass Cover and Well-maintained

Cover	Permissible velocity, ft/sec
Bermuda grass	6.0
Desert salt grass	5.0
Vine mesquite	
Lehman lovegrass	3.5
Big galleta	
Purple threeawn	
Sand dropseed	

3.6.4 Freeboard

Freeboard is the distance available above the water surface and the top of the channel. Freeboard provides capacity for events larger than the design event as well as capacity to handle other anomalies such as channel blockage. Freeboard is considered above any increases in water surface due to superelevation or channel curvature.

Pinal County requires freeboard for all channels consistent with the following criteria:

The minimum freeboard value for rigid channels shall be the larger of 1 foot for subcritical and 2 feet for supercritical flows or the freeboard computed using the formula in Volume 2, Chapter 6. Less freeboard requires specific approval of the County Engineer.

Freeboard for levees must meet the FEMA freeboard requirements (3.0, 3.5, or 4.0 feet minimum depending on location relative to the end of levee and to other structures).

3.6.5 Curvature

The minimum recommended centerline radius for a curved channel carrying subcritical flow is as calculated by the following formula:

$$r_c \geq 3T \tag{3.1}$$

Where T is the width of the water surface.

The minimum recommended centerline radius for a curved channel carrying supercritical flow is as computed using the following formula:

$$r_c \geq \frac{4V^2T}{gy} \tag{3.2}$$

Where y is the depth of flow.

Channel curves should be avoided where the flow is supercritical because the velocity distribution could become disturbed and the channel may not operate as intended. Refer to Chapter 6 in Volume II of this manual.

3.6.6 Bank Protection; Channel Linings

Channel embankments can be eroded or undermined by the action of flowing water if adequate protection is not provided. There are generally two methods of protecting the toe of embankments in erodible channels:

- Extend protection along the embankment toe to a depth that guards against the maximum extent of scour. This usually requires a rigid cutoff wall.

- Provide protection that adjusts to the scour as it occurs. Riprap, grouted rock, or rock baskets or mattresses placed against the toe can accomplish this.

Channel linings constructed with placed, graded riprap or rock baskets or mattresses to control channel erosion have been found to be cost effective where channel reaches are relatively short and where a nearby source of quality rock is available. Situations where riprap or basket or mattress linings may be appropriate are:

- Major flows are found to produce channel velocities in excess of allowable non-eroding values

- Channel side slopes no steeper than 3:1 for riprap and 2:1 for gabion mattresses

- Rapid changes in channel geometry occur, such as channel bends and transitions

3.6.6.1 Cutoff Walls

An embankment may be protected against scour by constructing a concrete cutoff wall to a depth below the expected scour. A cutoff wall does not, however, protect the bank against erosive forces from regular flow and other than scour.

The depth of cutoff walls designed to protect embankments must be as designed using the methods in Volume II Chapter 6, or other method approved by the County Engineer. Concrete cutoff walls shall be reinforced to at least meet the requirements of ACI 318 for temperature and shrinkage. The engineer shall also consider potential forces that may act on the cutoff wall that require greater or specialize reinforcing. The minimum thickness for cutoff walls shall be 8 inches, and the minimum concrete compressive strength shall be 3,000 psi at 28 days.

3.6.6.2 Riprap

Common riprap can be an effective channel lining or slope protection material if properly designed and constructed. The choice of riprap usually depends on the availability of graded rock with suitable material properties and at a cost that is competitive with alternative lining systems.

Riprap design involves the evaluation of five performance areas. These areas include the evaluation of:

- Riprap quality
- Riprap layer characteristics
- Hydraulic requirements
- Site conditions
- River conditions

Riprap rock quality

Riprap quality determination refers to the physical characteristics of the rock particles that make up the bank protection. Qualities determined to be most important include density, durability, and shape. Requirements for each of these properties are summarized in this section.

All stones composing the riprap should have a specific gravity of at least 2.4, following the standard test ASTM C127.

Durability addresses the in-place performance of the individual rock particles, and also the transportation of riprap to the construction site. In-place deterioration of rock particles can occur due to cycles of freezing and thawing, or can occur during transportation to the site. The rock particles must have sufficient strength to withstand abrasive action without reducing the gradation below specified limits. Qualitatively, a stone that is hard, dense, and resistant to weathering and water action should be used. Rocks derived from igneous and metamorphic sources provide the most durable riprap. Laboratory tests should be conducted to document the quality of the rock. Specified tests that should be used to determine durability include: the durability index test and absorption test (see ASTM C127). Based on these tests, the durability absorption ratio (DAR) is computed as follows:

$$DAR = \frac{DurabilityIndex}{PercentAbsorption + 1} \tag{3.3}$$

The following specifications are used to accept or reject material:

1. DAR greater than 23, material is accepted
2. DAR less than 10, material is rejected
3. DAR 10 through 23 and Durability index 52 or greater, material is accepted
4. DAR 10 through 23 and Durability index 51 or less, material is rejected.

There are two basic shape criteria. First, the stones should be angular. Angular stones with relatively flat faces will form a mass having an angle of internal friction greater than rounded stones, and therefore will be less susceptible to slope failures. Second, not more than 25 percent of the stones should have a length more than 2.5 times the breadth. The shape of the riprap stone should be cubical, rather than elongated. Cubical stones nest together, and are more resistant to movement. The length is the longest axis through the stone, and the breadth is the shortest axis perpendicular to the length. Angularity is a qualitative parameter that is assessed by visual inspection. No standard tests are used to evaluate this specification. If the engineer is faced with a supply of rounded river rock without a crusher to create angular rock, stone size should be increased 25% and side slopes decreased (USACE, 1995).

Riprap layer characteristics

The major characteristics of the riprap layer include: characteristic size, gradation, thickness, and filter-blanket requirements.

Characteristic Size - The characteristic size in a riprap gradation is the d_{50}. This size represents the average diameter of a rock particle for which 50 percent of the gradation is finer, by weight.

Gradation - To form an interlocked mass of stones, a range of stone sizes must be specified. The object is to obtain a dense, uniform mass of durable, angular stones with no apparent voids or pockets. The recommended maximum stone size is 2 times the d_{50} and the recommended minimum size is one-third of the d_{50}. The gradation coefficient, G, should equal 1.5.

$$G = 0.5\left(\frac{d_{84}}{d_{50}} + \frac{d_{50}}{d_{16}}\right) = 1.5 \quad (3.4)$$

Table 3-6 provides design gradations for riprap. As a practical matter, the designer should check with local quarries and suppliers regarding the classes and quality of riprap available near the site.

Table 3- 6: Riprap Gradation Limits

Stone Size Range (ft x d_{50})	Stone Weight Range, (lb x /W_{50})	Percent of Gradation Smaller Than
1.5 to 1.7	3.0 to 5.0	100
1.2 to 1.4	2.0 to 2.75	85
1.0 to 1.15	1.0 to 1.5	50
0.4 to 0.6	0.1 to 0.2	15

Thickness - The riprap-layer thickness shall be the greater of 1.0 times the d_{100} value, or 1.5 times the d_{50} value. But the thickness need not exceed twice the d_{100} value. The thickness is measured perpendicular to the slope upon which the riprap is placed.

3.6.6.3 Grouted Rock

Grouted rock is a structural lining comprised of a blanket of rock that is interlocked and bound together by means of concrete grout injected into the void spaces to form a monolithic revetment. The grout must extend the full thickness of the rock blanket, with the face rocks exposed for a maximum of one-fourth to one-third of their depth. This lining type is often suggested as a substitute for adequately sized riprap. It is not an equivalent product because it is neither rigid nor flexible. Any movement or settlement of the subgrade immediately results in cracks in the matrix that, in turn, allow water to enter behind the lining and greatly accelerate the lining's destruction. Because of these concerns the rock size in grouted rock armoring must be designed using the guidelines for ungrouted riprap.

Some reasons for using grouted rock instead of plain riprap are to provide a special aesthetic, to minimize vandalism, to inhibit the growth of volunteer vegetation, and to aid in maintenance.

Filter material must be placed under grouted rock armoring using the same design guidelines as for plain riprap.

A further concern in using grouted rock armoring is the possibility of uplift or seepage under the structure, since captured groundwater cannot easily surface as through an ungrouted rock face. Weeps must be included in grouted rock armoring to provide relief from water captured behind the structures. Grouted rock armoring more than five feet high must have two rows of weeps. The analysis for seepage and uplift is given in Volume 2, Chapter 6.

3.6.6.4 Rock Baskets and Mattresses

Rock baskets and rock mattresses refer to rocks that are confined by a wire basket so that they act as a single unit. The wire mesh enclosed rock units are also known as gabion baskets or gabion mattresses. This type of armoring provides an alternative where available rock sizes are too small for common riprap. Another advantage is that the regular geometric shapes can be fashioned into almost any shape that can be formed with concrete.

Mattresses are wire-enclosed rock units where the thickness (depth) is considerably less than the width or length. Baskets are thicker (deeper) than mattresses.

The durability of wire-enclosed rock is generally limited by the service life of the galvanized binding wire, considered to be about 35 years under normal conditions here in the arid southwest. Where the baskets or mattresses are subjected to frequent wet conditions, the life span diminishes to about 15 years (Myers, 2000). Water carrying silt, sand or gravel can reduce the service life of the wire. Also, water that rolls or otherwise moves cobbles and large stones breaks the wire with a hammer and anvil action and considerably shortens the life of the wire. The wire has been found to be susceptible to corrosion by various chemical agents and is particularly affected by high sulfate soils. If corrosive agents are known to be in the water or soil, a plastic coated wire should be specified. The designer should verify site-specific conditions and coordinate with a qualified manufacturer to properly specify the wire. See ASTM A-974 and ASTM A-975.

Baskets and mattresses are not maintenance free and must be periodically inspected to determine whether the wire is sound. If breaks are found while they are still relatively small, they may be patched by weaving new strands of wire into the wire cage. Wire-enclosed rock installations have been found to attract vandalism. Flat mattress surfaces seem to be particularly susceptible to having wires cut and stones removed. It is recommended that, where possible, mattress surfaces be buried, where they are less prone to vandalism.

Wire enclosed rock installations should be inspected at least once a year under the best circumstances and may require inspection every three months in vandalism prone areas in conjunction with a regular maintenance program. They should also be inspected after high flow events. Under high flow velocity conditions, mattresses on sloping surfaces must be securely anchored to the surface of the soil.

Materials

Rock filler for the wire baskets should meet the rock property requirements for common riprap. Rock sizes and basket characteristics should meet ASTM A-974 and ASTM A-975. The minimum rock size should be equal to the size of the basket mesh opening. The maximum rock size d_{100} should be less than the basket or mattress thickness.

Design Considerations

Twisted wire mesh has been found to be more tolerant to settlement than welded wire mesh (See ASTM A-975).

The long side of the basket or mattress should be aligned parallel with the channel for applications on banks steeper than 2:1. Channel linings should be tied to the channel banks with basket or mattress counterforts (thickened sections that extend into the channel bank) at the upstream edge of the lining. Counterfort spacing shall be per manufacturer's recommendations.

Mattresses and baskets on channel side slopes must to be tied to the banks using metal stakes no less than 4 feet in length (sandy soils warrant longer lengths). These should be located at the inside corners of basket or mattress diaphragms along an upslope (highest) basket wall, so that the metal stakes are an integral part of the basket. The exact spacing of the stakes depends upon the configuration of the baskets, however the following is the suggested minimum spacing: stake every 6 feet along and down the slope for 2:1 slopes or steeper.

For most applications, mattresses should be a minimum of 9 inches thick.

3.6.6.5 Concrete Linings

Reinforced cast in place concrete and pneumatically placed concrete are alternative lining materials for channels with limited right of way, high velocity flow, and supercritical flow conditions. The most common problems of concrete lined channels are due to bedding and liner failures. Typical failures are:

- Liner cracking due to settlement of the sub-grade
- Liner cracking due to the removal of bed and bank material by seepage force
- Liner cracking and floating due to hydrostatic back pressure from high groundwater.

The minimum slope for concrete and shotcrete channels is 0.0015 ft/ft due to limitations of construction.

Lack of maintenance can result in vegetation growth through the concrete lining and sediment deposition in the channel that will increase the flow resistance.

All concrete lined channels must have continuous reinforcement extending both longitudinally and laterally. For channels carrying supercritical flow, there shall be no reduction in cross sectional area at bridges or culverts, or any obstructions in the flow path.

Bridges or other structures crossing the channel must be anchored satisfactorily to withstand the full dynamic load that might be imposed upon the structure in the event of major debris blockage. Tributary storm drain pipes must not protrude into the channel flow area.

Safety and structural requirements become a primary concern if side slopes are steeper than 2:1. Design of the lining should also include consideration of anticipated vehicular loading from maintenance equipment. Joints in the lining should be designed in accordance with standard structural analysis procedures with consideration of the size of the channel, thickness of the lining and anticipated construction techniques. The concrete lining must be keyed into the adjacent over-banks. The lining must also be keyed into the bed at points where transitions are made between flatter and steeper slopes if the difference in gradient is substantial. And, the liner must be keyed into the bed upstream and downstream from zones of supercritical flow.

Shotcrete vs. Cast-in-Place Concrete

The shotcrete process has become an important and widely used technique for channel linings. Shotcrete is mortar or concrete pneumatically projected at high velocities onto a surface. In the past, the term 'gunite' was commonly used to designate dry-mix mortar shotcrete. The term is currently outdated and ‘shotcrete’ has become the trade name for all pneumatically applied dry-mix or wet-mix concrete or mortar.

ACI 506R (1985) discusses the properties, applications, materials, reinforcement, equipment, shotcrete crews, proportioning, batching, placement, and quality control of the shotcrete process.

As a channel lining, shotcrete is an acceptable method of applying concrete with a general improvement in density, bonding, and decreased permeability. The same design considerations discussed for cast-in-place concrete channels apply in the design of shotcrete channels. Shotcrete linings are to be designed to the same thickness and reinforcement as required for concrete linings.

Roughness coefficients

The roughness coefficient for a concrete lining can vary from 0.011 for a troweled finish to 0.020 for a very rough or unfinished surface. For pneumatically place concrete, roughness coefficients can vary from 0.016 to 0.025. The accumulation of sediment and debris may modify the roughness coefficient.

Bedding

Long-term stability of concrete lined channels depends in part on proper bedding. Undisturbed soils often are satisfactory for a foundation for lining without further treatment. Expansive clays are usually an extreme hazard to concrete lining and should be avoided or replaced. A filter underneath the lining is recommended to protect fine material from creeping along the lining. A well-graded gravel filter should be placed over the channel bed prior to lining the channel with concrete.

Transitions

Transitions between earth-lined and concrete lined channel can allow water to enter the soils under the concrete lined section either by seepage or by scour. Such transitions must be designed to prevent undermining of the lining and to reduce turbulence. Cutoff walls should be incorporated at both the upstream and downstream end of the concrete lined channel segment to reduce seepage forces and to prevent lining failure due to scour, undermining, and piping. The depth of cutoff walls should extend below the expected scour depth. Determination of expected total scour depth requires analyses as discussed in Volume 2, Chapter 6.

Underdrains

Underdrains can greatly reduce the probability of damaging the concrete lining due to hydrostatic backpressure and subgrade erosion.

One type of underdrain consists of 4- or 6-inch diameter perforated pipelines placed in gravel-filled trenches along one or both toes of the inside slopes. These longitudinal drains are either connected to transverse cross drains which discharge the water below the channel or to pump pits, or extend through the lining and connect to outlet boxes on the floor of the channel. The outlet boxes are equipped with one-way flap valves that prevent backflow and relieve any external pressure that is greater than the water pressure on the upper surface of the channel bottom.

A second type consists of a permeable gravel blanket of selected material or sand and gravel pockets, drained into the channel at frequent intervals (10 to 20 feet) by flap valves in the channel invert. Both the tile and pipe system and the unconnected flap valve type must be encased in a filter that will prevent piping of subgrade material into the pipe or through the valve. For detailed information on underdrains refer to Lining for Irrigation Canals (USBR, undated).

Weep holes spaced at appropriate intervals may be used where a lesser degree of seepage control is warranted. Weep holes may be equipped with flap valves or other measures that allow seepage relief but prevent backflow or introduction of surface water behind the lining.

3.6.6.6 Soil-Cement Linings; Roller-Compacted Concrete

Soil cement (a combination of native soil, Portland cement and water) has been shown to be an effective and economical method for slope protection and channel lining in the Pinal County area.

Materials

A wide variety of soils can be used to make durable soil cement. For maximum economy and most efficient construction, it is recommended that:

- The soil contains no material retained on a 3-inch (75 mm) sieve
- Between 40 percent and 80 percent pass the No. 4 (4.75 mm) sieve
- Between 2 percent and 10 percent pass the No. 200 (0.074 mm) sieve
- The Plasticity Index (PI) of the fines should not exceed 10

If the onsite material does not meet these guidelines, the addition of imported material may be necessary. Standard laboratory tests are available to determine the required proportions of cement and moisture to produce durable soil cement. The design of most soil cement for water control projects is based on the cement content indicated by ASTM testing procedures and increased by a suitable factor to account for direct exposure, erosion or abrasion forces.

The Portland cement should comply with one of the following specifications: ASTM C150, CSA A5, or AASHTO M85 for Portland cement of the type specified; or ASTM C595 or AASHTO M240 for Portland blast-furnace slag or Portland pozzolan cement, excluding slag cements Types S and SA.

It is important that testing to establish required cement content be done with the specific cement type, soil, and water that will be used in the project.

Typically, soil cement linings are constructed by the central-plant method, where selected onsite soil materials, or soils borrowed from nearby areas, are mixed with Portland cement and water and transported to the site for placement and compaction.

Soil Cement Linings on Slopes

On side slopes, the soil cement is often constructed by placing and compacting the material in horizontal layers stair-stepped up the slope. The rounded step facing results from ordinary placement and compaction methods. The sideslope can vary from 1:1 to 3:1 depending on the soil type and natural angle of repose. Side slopes steeper than 2:1 are not recommended, due to safety issues, but may be allowed when right-of-way is a problem. Soil cement may be placed on slopes 3:1 or flatter at a minimum thickness of eight to twelve inches, depending upon the mixing technique. This would be done without the stair-step layer approach, where a lesser level of protection is permissible.

Transitions

The soil cement facing must be tied into non-erodible sections or abutments. The upstream and downstream ends of the facing should terminate smoothly into the natural channel banks. A buried cutoff wall normal to the slope or other measures may be necessary to prevent undermining of the soil cement facing by flood flows.

The top of the lining should be keyed into the ground to protect against erosion of the backside of the soil cement layer by lateral inflows.

Seepage and Uplift; Scour

Seepage and related uplift forces must be considered and appropriate counter-measures such as weep holes or subdrains provided if required.

The lining should be designed to extend to the anticipated depth of total scour or some other suitable means of scour protection provided.

Pipe Penetrations

The junction where pipes penetrate the soil-cement lining can be protected by placing and compacting the soil cement by hand, using small power tools, or by using a lean mix concrete.

Further design information may be found in ACI 230.1, State Of The Art Report On Soil Cement. Additional information on design and construction is available from the Portland Cement Association, Skokie, IL.

3.6.6.7 Synthetic Stabilization Materials

Many new materials have been developed for slope stabilization and erosion protection and have been tested and found useful for channel armoring. Use of these materials shall be approved by the County Engineer.

3.6.6.8 Granular Filter Blankets

Granular filter blankets underlying riprap and rock mattresses and baskets protect the underlying soil from washing out and provide a base on which the armoring will rest. The need for a filter blanket is a function of particle-size ratios between the armoring and the underlying soil that comprise the channel bank. The inequalities that must be satisfied are as follows:

$$\frac{(d_{15})\,filter}{(d_{85})base} < 5 < \frac{(d_{15})\,filter}{(d_{15})base} < 40 \qquad \text{AND} \qquad \frac{(d_{50})\,filter}{(d_{50})base} < 40 \qquad (3.5)$$

In these relationships, "filter" refers to the overlying material and "base" refers to the underlying material. The relationships must hold between the filter blanket and base material and between the armoring and filter blanket (USDOT, 1988 and 1989).

If the inequalities are satisfied by the riprap itself, then no filter blanket is required. If the difference between the base material and the riprap gradations are very large, then multiple filter layers may be necessary. To simplify the use of a gravel filter layer, Table 3-7 outlines recommended standard gradations. The Type-I and Type-II bedding specifications shown in Table 3-7 were developed using the criteria given in the equations above, considering that very fine grained, silty, non-cohesive soils can be protected with the same bedding gradation developed for a mean grain size of 0.045 mm. The Type-I bedding in Table 3-7 is designed to be the lower layer in a two-layer filter for protecting fine-grained soils. When the channel is excavated in coarse sand and gravel (i.e., 50 percent or more by weight retained on the #40 sieve), only the Type-II filter is required. Otherwise, two bedding layers (Type-I topped by Type-II) are required. For the required bedding thickness, see Table 3-8.

Table 3- 7: Gradation for Gravel Bedding

Sieve Size	Type I (% passing)	Type II (% passing)
3 inches (76 mm)	-	-
1-1/2 inches (38 mm)	-	60 to 70
3/4 inch (19 mm)	-	20 to 90
3/8 inch (9.5 mm)	100	-
#4 (4.75 mm)	95 to 100	0 to 20
#6 (1.18 mm)	45 to 80	-
#50 (0.30 mm)	10 to 30	-
3/4 inch (19 mm)	-	-
#100 (0.15 mm)	2 to 10	-
#200 (0.075 mm)	0 to 2	0 to 3

Table 3- 8: Thickness Requirements for Gravel Bedding

Riprap Size, (in)	Minimum Bedding Thickness, (in)		
	Fine Grain Native Soils		Coarse Grain Native Soils
	Type I	***Type II***	***Type III***
6, 8	4	4	6
12	4	4	6
18	4	6	8
24	4	6	8
30	4	8	10
36	4	8	10

3.6.6.9 Fabric Filters

Filter fabric is not a complete substitute for granular bedding. Filter fabric provides filtering action only perpendicular to the fabric and has only a single equivalent pore opening between the channel bed and the riprap. Filter fabric has a relatively smooth surface that provides less resistance to stone movement. Tears in the fabric greatly reduce its effectiveness so that direct dumping of riprap on the filter fabric is not allowed and due care must be exercised during construction. The site conditions and specific application and installation procedures must be carefully considered in evaluating filter fabric as a replacement for granular bedding material. Filter fabric can provide adequate bedding for channel linings along uniform mild sloping channels where leaching forces are primarily perpendicular to the fabric. Numerous failures have occurred because of the improper installation of filter fabric. Therefore, when using filter fabric it is critical that the manufacture's guidelines for installing it be followed.

The design criteria for filter fabric are a function of the permeability of the fabric and the effective opening size. The permeability of the fabric must exceed the permeability of the underlying soil, and the apparent opening size (AOS) must be small enough to retain the soil.

The criteria for apparent opening size (AOS) are as follows:

- For soil with less than 50 percent of the particles, by weight, passing a No. 200 sieve, the AOS should be less than 0.6 mm (a No. 30 sieve).
- For soil with more than 50 percent of the particles, by weight, passing a No. 200 sieve, the AOS should be less than 0.3 mm (a No. 50 sieve).

3.6.7 Drop Structures

Drop structures having loose riprap on a sloping face are not permitted due to a high failure rate and excessive maintenance costs.

Analysis of drop structures should be conducted for a range of flows because flow characteristics at the drop can vary with discharge.

The maximum vertical drop height from crest to basin for a vertical hard basin drop is limited to 3 feet for safety considerations.

Maximum drop depth for a vertical riprap basin is limited to 3 feet due to safety considerations and the practicality of obtaining large basin riprap for higher drops.

Refer to Chapter 8 of Volume II of this manual.

3.6.8 Safety

Hydraulic structures constructed in Pinal County will usually be subject to public access. Designs for hydraulic structures must address the issue of safety. Appropriate measures must be designed to keep the public away from hazardous locations. For example, vertical drop structures should not exceed 2.5 feet in height with 6-foot horizontal aprons, and adequate fencing or railings must be provided along all other walls, such as wing walls or training walls.

3.6.8.1 Fencing

Except as subsequently provided, fencing will be required for all new concrete, shotcrete, and soil-cement lined channels with side slopes steeper than 4:1. Subcritical channels lined with concrete, shotcrete, and soil-cement lined channels having depths and bottom widths less than 3 feet and 5 feet, respectively, will not require fencing. Reviewing authorities may require fencing regardless of the conditions listed in this manual.

3.6.8.2 Signs

Designs for hydraulic structures must address the issue of safety. Signage must be provided to identify the potential hazard of flooding or dangerous flow conditions to the public. Appropriate measures must be included to keep the public away from hazardous locations.

3.7 EROSION AND SEDIMENTATION

Sedimentation and the fluvial processes associated with sediment transport play an important role in the long-term conveyance capacity of a drainage system as well as the on-going cost of maintenance. Sedimentation is a very complex subject and not all drainage designs need to consider it as a primary design criterion. However, because sedimentation may have significant impacts the designer of stormwater facilities must at least evaluate whether the design may affect erosion and sedimentation.

3.7.1 Watercourse Stabilization

Any watercourse modifications affecting the flow direction, depth, velocity or duration of discharge may result in changes in erosion and sedimentation. The following is a partial list of watercourse modifications and potential impacts:

- Channel straightening will generally increase channel gradient and flow velocity, and may initiate channel erosion.
- Channel constriction increases flow velocities and often flow depth, thus increasing sediment transport capacity and may initiate channel erosion.
- Lowering the bed elevation of a watercourse may prompt degradation in the main stem of the watercourse and its tributaries.
- Raising the bed elevation or reducing the slope of the energy grade line may result in sediment deposition upstream due to reduced transport capacity.
- Bank lining may increase flow velocity and increase erosion and/or bank attack where banks are left unprotected.

3.7.2 Stormwater Storage

Stormwater storage facilities must be planned and analyzed for their impact on sediment transport in watercourses. Those impacts can result from the following:

- Sediment deposition in the impoundment and upstream backwater of the impoundment may result in decreased upstream conveyance capacity, the potential for breakout flows due to sediment deposits, and maintenance requirements in regard to sediment deposits.
- Release of “clear water” downstream of the impoundment may result in local scour and/or degradation of the downstream watercourse including the bed becoming more “cobbly” and loss of riparian vegetation.

- Although the peak discharges are usually reduced downstream of the storage facility, the duration of high flows often increases. This may increase the opportunity for scour and the flushing of finer sediments through the system.

3.7.3 Sand and Gravel Mining

Sand and gravel mining located within floodplains may also represent a source of large amounts of sediment, but mine excavations may also act as a sediment sink depending upon the geometry of the mine pit and quantity of flow. The fluvial processes in and around sand and gravel mining facilities during times of flood are complex and should be taken into consideration.

3.7.4 Water Quality

Sediment in water is often viewed as an undesired element for municipal and industrial water but can be an asset for irrigated agriculture and for certain riparian habitats. The undesirable characteristics are related to quantity, sediment's abrasive nature and the fact that compounds can become attached to sediment particles and thereby be transported and stored along with sediment.

3.8 HYDRAULIC STRUCTURES

3.8.1 Hydrodynamic Forces

Elevated foundations must be designed to withstand both hydrodynamic forces caused by velocity of waters and hydrostatic forces caused by standing water. They must also meet the requirements of the Planning and Building Department. The lateral loading on structures shall allow for at least the following amounts of debris accumulation:

1. Piers or stem walls - 1 foot either side of pier or stem wall.

2. Lowest structural member - The following minimal lateral load shall be considered:

   - Elevation of lowest structural member above floodway elevation

   - Equivalent depth of penetration into water surface

The engineer shall evaluate the amount and nature of debris available in the drainage basin to determine if a greater debris accumulation provision is justified for any or all calculations.

The flow velocities used for the loading analysis shall be obtained from the HEC-2/HEC-RAS run with the encroachment in place. A flow distribution showing velocities for each increment between elevation station, or “GR”, points at the location of the structure shall be included with the analysis. If applicable, it may be necessary to run the HEC-2/HEC-RAS model as “supercritical” to obtain the appropriate design data.

3.8.1.1 Impact forces on foundations

Normal impact loads are those that relate to isolated occurrences of typically sized ice blocks, logs, or floating objects striking the structure. For design purposes, this can be considered a concentrated load acting horizontally at the maximum water elevation, or any point below it, equal to the impact force created by a 1000 pound mass traveling at the velocity of the flood water, acting on a one-square-foot surface of the structure.

Special impact loads are those that relate to large conglomerates of floating debris, either striking or resting against a structure or its parts. In an area where special impact loads may occur, the load considered for design purposes is the impact created by a 100 pound load times the width of the building, acting horizontally over a one foot wide horizontal strip at the maximum water elevation or at any level below it. Where natural or artificial barriers exist which would effectively prevent these special impact loads from occurring, these loads may be ignored in the design. The equivalent depth of penetration into the water surface given for the lateral loading analysis shall be applied to the special impact loading.

3.8.2 Scour

Foundation footings shall be located at least three feet below the maximum estimated scour elevation.

A scour analysis should consider debris accumulation on piers or structures as required for the HEC-2/HEC-RAS computer modeling. Soil tests shall be presented to verify the nature of the in-place soils to the full depth of the projected scour. The method of scour analysis shall be at the engineer’s discretion, but shall be appropriate for the type of encroachment (piers, stem walls, solid foundation or fill). Acceptable methods for scour analysis can be found in Volume 2.

3.8.3 Operation and Maintenance

Hydraulic structures should be designed so they can be maintained. As with other drainage facilities, maintenance operations will consist of scheduled and unscheduled operations. Scheduled operations include mowing, debris removal, graffiti removal, and rock replacement. Unscheduled operations are those which follow a storm event and include debris removal, rock replacement, erosion repair, fence or railing repair and other activities for which the frequency and scope cannot be predicted. Some maintenance considerations appropriate for hydraulic structures are presented below. Access to key areas (i.e. crest area, stilling basin area) for maintenance equipment and personnel is the primary consideration common to all structure types.

Slopes of 4:1 or flatter are recommended for mowing equipment on landscaped or grass bank and transition slopes. The County Engineer should be consulted regarding special circumstances for specific site constraints where a steeper slope may be necessary.

Transition areas upstream and downstream of the structures should be designed to drain completely. This applies particularly to stilling basins.

Selection and placement of rock for a stilling basin or upstream of a drop crest should consider a size range not easily displaced by flow and not easily moved by vandalism. Grouted boulders are a suitable alternative.

Open channels are recommended in lieu of pipes for conveyance of low flows through the drop structure area. Pipes may plug or frequently overtop, leading to additional maintenance problems. Riprap should be provided at likely scour areas that are relatively expensive to access and repair later.

3.8.4 Structure Aesthetics

3.8.4.1 General

Aesthetics, safety, recreation, and overall integration with nearby land uses are important aspects in the design, of hydraulic structures. The design, planning, construction, and maintenance of hydraulic structures and natural drainageways in an urban setting offer opportunities for promoting aesthetic design and habitat features. Maximizing functional uses while improving visual quality requires good planning from the onset of the project, and the coordinated efforts of the owner/client, engineer, landscape architect, and planner. The significance of providing an aesthetic and visually appealing project depends on the number, type, and frequency of viewer; as well as the viewing angle; project location; and the overall environment of the project area. Aesthetic considerations are site and project specific.

The combination and diversity of forms, lines, colors, and textures create the visual experience. Material selection and landscape design can provide visual character and create interesting spaces in and around hydraulic structures.

3.8.4.2 Open Spaces and Parks

Creative planning concepts in urban and urbanizing areas, particularly in residential areas, emphasize multiple uses of flood control, recreation, and open spaces. Cluster housing and good subdivision planning may be coordinated to offer opportunities to maintain the natural habitat characteristics of the drainageway while fulfilling open space and recreation requirements.

Multiple use of flood control structures and open space parks has proven to be an effective and aesthetic land use combination. Athletic fields and stormwater storage areas which remain dry most of the time have been used in many communities. The design of overflow structures and crest controls can be combined with concrete pathways to blend with a park lined environment.

3.8.4.3 Materials

A variety of materials and finishes are available for use in hydraulic structures. Concrete color additions, exposed aggregates and form liners can be used to create visual interest to otherwise stark walls. The location of expansion and control joints in combination with reveals can be used to create effective design detailing of headwalls and abutments. Rock and vegetation can be used for bank stability and erosion protection around structures to provide visual contrast and diversity, and spatial character.

3.9 STRUCTURES NEAR FLOODPLAIN

3.9.1 Structure Finished Floor

All structures built within Pinal County should be positioned to prevent property damage from regular flooding occurrences. Flooding may occur in association with defined channels and detention or retention basins as well as washes and alluvial fans. No structure may be built in the 100-year floodplain without prior approval of the County Engineer.

3.9.1.1 Elevation Requirements

The finished floor of all structures shall be elevated a minimum of one foot above the 100-year floodplain, the 100-year water surface of a detention or retention facility and the emergency outfall of any basin. For development in areas subject to sheet flow (such as alluvial fans), the requirements of Arizona State Standard SSA 4-95 Identification of and Development Within Sheet Flow Areas (page 16) must be met and receive approval from the County Engineer.

A finished floor elevation may be other than the minimum permitted, provided it is determined by technical data certified by an Arizona Registered Civil Engineer to be a minimum of one foot above the 100-year water surface elevation in adjacent streets and drainage ways, the minimum necessary to be safe from inundation by the 100 year peak runoff event. Finished floor elevations shall be referenced to a known benchmark. Aerial photographs of the 1983 and 1993 floods may be used, with the approval of the County Engineer, to supplement FEMA flood zone water surface elevations.

3.9.1.2 Lateral Requirements

Lateral migration, or shifting, often occurs in watercourses located in the southwest, where water flow tends to be brief but swift and banks and beds are easily eroded. A shifting watercourse presents difficulty when establishing a floodplain as the delineation of the floodplain changes as the watercourse path changes. For this reason, setback requirements must be enforced to ensure that a structure is placed well away from not only the present floodplain but also any floodplain that might occur from a shifting watercourse. For horizontal setback requirements for structures the requirements of Arizona State Standard SSA 5-69 Watercourse System Sediment Balance, Guideline 1 must be met and receive approval from the County Engineer.

3.10 DETENTION AND RETENTION

3.10.1 Event Size

All detention/retention facilities incorporated within new developments shall be designed to accommodate the peak flow and volume of runoff from the 100-year, 2-hour duration storm event in order to meet the peak discharge requirement.

The requirement for a development to provide storage of excess runoff by detention or retention facilities shall not be waived unless determined otherwise by the jurisdictional agency on a case-by-case basis.

3.10.2 Limitations Storage/Conveyance Facilities

The requirement for a development to provide stormwater storage facilities will not be waived unless determined otherwise by the County Engineer on a case by case basis. The County Engineer may reduce the requirements for on-site retention/ detention where the storm water runoff discharges directly to a regional drain, provided that any reduction in on-site retention/detention does not increase peak flow within the watercourse for the 100-year event.

The use of detention instead of retention will also be reviewed on a case by case basis. Retention is the preferred stormwater storage method in Pinal County.

If runoff is to be conveyed by an underground system, complete detailed plans shall be submitted to the County Engineer.

On-site drainage shall be either to the street or to a designated drainage easement with adequate outfall. Off-site flows may not be routed through a retention facility unless specifically approved by the County Engineer.

Retention of runoff emanating from industrial developments and infiltration of runoff to the subsurface will be handled on a case-by-case basis by the appropriate reviewing agency subject to water quality concerns.

3.10.2.1 Multiple Lot Residential Developments

Whenever possible, the facilities shall be designed for multiple uses, such as parks or other recreational facilities, to offset the cost of open space and to encourage improved maintenance. Residential developments (recorded subdivisions) shall not provide for nor rely on single-lot, on-site stormwater storage, and the design of common facilities shall not assume any individual lot on-site storage, unless approved by the County Engineer. Individual lot retention may be permitted in residential subdivisions with a minimum lot size of one acre with the approval of the County Engineer.

Developments with Homeowner’s Associations will locate its facilities in private drainage tracts or in public sites dedicated by the developer, in accordance with the jurisdictional agency’s requirements. The Homeowner’s Association will maintain the private facilities, and the jurisdictional agency will usually maintain the public tracts. Common storage facilities for single family developments without a Homeowner's Association and with public streets will have maintenance provisions determined by the County Engineer. In any landscaping and maintenance agreement, provisions shall be made for an annual maintenance certification. The number and location of storage facilities within a development are to be approved by the County Engineer. Dedication to the public may require the inclusion of recreational facilities or other features deemed necessary by the County

3.10.2.2 Single Lot Non-Residential Development

Single lot, non-residential developments that are not served by a public stormwater storage facility will provide the required storage on the lot itself and outside the right-of-way area, regardless of lot size. Maintenance shall be provided for by the property owner.

3.10.2.3 Single Lot Residential Development

Single lot, residential parcels that are not a part of a recorded subdivision, such as lots created by parcel splits and minor land divisions, will also provide the required storage on the lot itself and outside the right-of-way area, if it is demonstrated that a common basin with adjacent parcels is not practical. Each jurisdictional agency may establish lot size requirements governing the application of this requirement, but in all cases the residential lots smaller than 1 acre in size shall provide the required storage.

3.10.2.4 Regional Stormwater Storage Facilities

Regional detention/retention facilities are large storage facilities located at strategic sites within a watershed to provide control of runoff. The advantage of this type of facility is that the siting and design of regional storage facilities is normally incorporated as part of an overall drainage master plan. Thus, alternative siting combinations and their respective hydraulic routing effects can be investigated. Storage alternatives can be evaluated with other factors (that is, conveyance system, land and maintenance costs), to arrive at an optimal solution to alleviate flooding problems within the drainage basin.

3.10.3 Basin Configuration

3.10.3.1 Location

In general, storage facilities are to be located so they can intercept the flow from the entire development area. If portions of the area cannot drain to a single facility, then additional facilities may be added to provide control of those areas as approved by the County Engineer. The objective is to provide storage of excess runoff with a minimum number of detention/retention facilities located at optimum points within a development area. Whenever possible, the facilities shall be designed for multiple uses, such as parks or other recreational facilities, to offset the cost of open space and to encourage improved maintenance.

The detention/retention 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 shoreline.

Retention/detention basins shall not be located within 25 feet of septic system facilities.

Utility lines and structures shall not be located within drainage facilities unless approved by the utility company and the County Engineer.

If reasonable alternatives are not available detention in the County right-of-way may be acceptable provided the County Engineer approves the design.

A right-of-way or public utility easement shall not be designated for drainage or retention without prior written approval of the appropriate agency or affected utility.

3.10.3.2 Sitting

With respect to siting, stormwater storage facilities which utilize a method of subsurface disposal shall be located such that the infiltration surface will be a specific distance, both horizontal and vertical, from any functioning water well. The County Engineer should be contacted regarding regulations governing the siting of such facilities near wells or near the static groundwater table.

3.10.3.3 Shape

As a general rule, curvilinear, irregularly shaped facilities will have the most natural character. A wide range of shapes can be considered and utilized to integrate the stormwater storage facility with the surrounding site development. Smooth curves should be used in the plan layout of the grading for the facility. On-site retention/detention facilities may include natural depressions or man-made basins.

3.10.3.4 Side Slopes

Where grass is intended to be established, side slopes shall not be steeper than 4 horizontal to 1 vertical. Where other protection measures are intended, such as shrub planting, rock riprap or other structural measures, slopes shall not exceed 3 horizontal to 1 vertical unless approved by the appropriate jurisdictional agency. Where slopes abut the street right-of-way, the minimum slope shall be 4 horizontal to 1 vertical regardless of surface treatment. Some jurisdictions may require a flatter slope. The designer should verify the slope requirement prior to commencing design.

Transitions from slopes to level ground at the top and bottom of a facility shall be smooth curves. In all cases, slopes must be designed to allow for safe operation of maintenance equipment. Refer to Section 8.5.1 for maintenance access provisions. Side slope design should be done with the visual character of the completed facility in mind. A more natural appearance can be achieved by varying side slopes within a stormwater storage area.

3.10.3.5 Depth and Bottom Configuration

Maximum ponding depth shall be 3 feet and minimum freeboard shall be 1 foot unless authorized by the County Engineer. With respect to grading, deep facilities should be avoided, if possible. The bottom shall be designed to drain to a low flow channel for a detention facility.

3.10.3.6 Basin Inlet

The detention or retention basin inlet must be designed to avoid erosion within the basin or headcutting or deposition in the inlet channel.

The inlet must be designed so that sediment carried into the basin will not deposit near the inlet and block inflows to the basin.

3.10.3.7 Basin Outlet

The minimum allowable pipe size for primary outlet structures is 12 inches in diameter.

Flows from basins shall not exceed pre-development flows for the 2, 10 and 100 year runoff event and shall be in the location and direction of the historic flows.

Trash Racks shall be provided for pipe and orifice outlets.

The County Engineer may require attenuation of a single frequency storm or a number of frequencies for a given detention facility. Refer to the specific requirements of the jurisdiction where the design is being prepared; however, two-stage and multi-stage control structures are becoming more widely used.

If the flow capacity of an outlet pipe must be further reduced, an orifice plate may be attached. The orifice plate must be constructed of heavy, galvanized steel and attached by tamper-proof bolts. Other outlet configurations may be allowed provided they meet the requirements of the permitted release rates at the required volume and include proper provisions for maintenance and reliability.

Primary outlet structures, particularly those controlling multiple storm events, are often special design structures unique to specific site applications. Furthermore, consideration must be given to structural adequacy and flotation under hydrostatic loads.

Low Flow/Low Level Outlets

For health and safety reasons, stormwater storage facilities must drain within 36 hours for the design storm runoff volume. For stormwater facilities in a series, the cumulative post storm drain time is 36 hours. In addition, the peak discharge from a low flow out-let shall be significantly less than the existing watershed peak discharge for retention facilities. These guidelines form the basis for design. Compliance with NPDES requirements often dictates a third criterion for low flow/low level outlets. Here, the outlet is often designed to retain the first flush and/or the floating hydrocarbon pollutants. In this situation, undershot weirs or inverted siphons may be used.

Energy Dissipation at Outlet

Adequate energy dissipation measures shall be provided at the downstream end of primary outlet structures. Such measures shall be designed to control local scour at the pipe outlet and to reduce velocities to pre-development conditions prior to exiting onto the downstream property.

3.10.3.8 Trashracks

Trashracks shall be provided to inlets of pipe and orifice outlet structures. See Volume II Chapter 7 for hydraulic analysis guidelines and Chapter 1 for safety considerations.

3.10.3.9 Lining/Surface Treatment

In keeping with the goal of stormwater storage facilities as amenities that incorporate multiple use concepts where possible, grass and/or landscape plantings are preferred surface treatments. As a general rule, grass and plant species used for landscape development and revegetation should be native to Pinal County. A registered landscape architect should prepare the landscape design with consideration toward use of plant species appropriate for the level and frequency of inundation of the facility. Permanent irrigation systems are required for grass areas and most types of basin revegetation and landscaping. However, use of native and drought tolerant species (including seeding) may only require a temporary system to obtain effective germination and establishment. Whether permanent or temporary, that portion of the irrigation system within the flood zone must be designed to tolerate inundation and silt accumulations.

The use of inert materials is appropriate for stabilization and erosion control where steep slopes are unavoidable, including along channels, at inflow points, at the outlet control structure and any other location where flowing water may threaten stability. Use of these materials should be properly engineered (refer to Chapter 6) and should respond to aesthetic considerations. Inert materials for erosion control include:

- Loose rock riprap with a specific, engineered gradation
- Loose or grouted boulders (minimum dimension 18 inches and larger)
- River stone
- Gabions
- Soil cement and concrete

Designs that combine landscape planting with the use of inert materials are recommended. Voids can be designed within the inert material to allow installation of plants. The result is a durable and attractive method of protection.

3.10.3.10 Emergency Spillways

Emergency spillways are normally surface overflow weirs, channels, or combinations thereof, provided for the safe overflow and routing of floodwaters under unusual circumstances. Such situations include the blockage or malfunction of the primary outlet structure or the occurrence of a storm event larger than that for which the facility was designed. Consideration must be given to the layout and configuration of the emergency spillway so that excess flow is safely released and conveyed without increasing flood hazards to adjacent properties and in the same manner and direction as would have occurred under pre-development or historic conditions. Emergency spillways must be designed to convey the unattenuated 100-year peak discharge at non-erosive velocities.

3.10.3.11 Permanent Pools

Certain jurisdictions permit the design of a stormwater storage facility that incorporates a permanent pool for aesthetic purposes. The engineer should contact the County Engineer for specific criteria and regulations regarding such facilities. General considerations for facilities incorporating permanent pools are listed below:

- Flood storage volume shall be maintained above the level of the permanent pool. Provision for draining the full depth of the pond shall be included at the outlet structure.

- Maintenance of a minimum water level should be provided either by the inflow from the watershed, and/or by augmentation from other sources during prolonged dry periods and by the capability of the bottom of the facility to retain water. Seepage and evaporation losses shall be considered.

- Maintain water quality and minimize algae growth by designing for sufficient minimum depth and incorporating use of recirculation and aeration measures.

- Consider public safety as primary in the design of all features related to the permanent pool.

- Geometric characteristics of the pond include:

    - Choose bottom lining material suitable for retention of water and with consideration toward maintenance (that is, ease of sediment removal, etc.). Provisions for completely draining the pond should be made.

    - Create aesthetic yet maintainable edges. Edge design also should consider the effect of drawdown of the water surface. That is, a drop in water surface elevation should not create a wide expanse of unsightly shoreline. Similarly, the area surrounding the permanent pool should be designed for periodic inundation. The area should drain completely and return to a stable surface following a flood event.

    - Provision of stable side slopes above and below the permanent water surface.

    - 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 shoreline.

    - Resolve permanent pool water depth issues versus safety needs; a 3-foot depth at shoreline required to limit pond edge vegetation growth exceeds the recommended pond edge depth (1.5 to 2.0 feet). Therefore, other safety measures must be considered.

- The design should consider measures to minimize sediment inflow to the pond. Once sediment has entered the permanent pond, then removal can be expensive and may require draining the pond. Erosion should ideally be controlled at the source or by mitigation measures along the incoming channel. However, if such measures are not feasible, a sediment trap should be designed at the pond inflow location to intercept the majority of the incoming sediment and to facilitate removal.

- If the stormwater storage facility and permanent pool are created by a retaining structure, such as an earth embankment, then the design guidelines for embankments shall be followed, with particular emphasis on seepage control and embankment stability

- Potential impacts downstream shall be considered. The designer should be aware that an impoundment may improve, worsen or maintain existing downstream flow characteristics, and that any changes, even apparent improvements, may be viewed as infringements of downstream riparian rights.

- Since a permanent pool is most often desired for creation of a focal amenity for a development, it is appropriate that a registered landscape architect work in conjunction with the engineer to achieve an aesthetic design with consideration of costs of construction and maintenance.

3.10.3.12 Low Flow Channels

A low flow channel is required in the bottom of a detention facility to provide positive routing of drainage to the primary outlet structure. The engineer will provide design of the reinforcement of the channel. The channel shall have a 0.5 percent maximum longitudinal slope. Alternative low flow channel designs may be considered at the discretion of the County Engineer; however, use of loose rock or other movable materials can only be made after careful consideration. See Volume II Chapter 6 for additional discussion relating to channels.

3.10.3.13 Drain Time

The design of all stormwater storage facilities shall be such that the stored runoff is completely discharged from the facility within 36 hours after the runoff event has ended. The draining of stormwater storage facilities may be accomplished via in-situ percolation, bleeding off (low flow) outlets, drywells, pump station, or a combination thereof.

3.10.4 Subsurface Disposal

The primary methods of underground disposal of stormwater runoff at retention facilities are engineered basin floors and drywells. Infiltration rates of basin floors or drywells shall not be used in determining outflow rates in flood-routing procedures.

3.10.4.1 Engineered Basin Floors

Analysis and design of the bottom of a retention facility intended for subsurface disposal is detailed in Underground Disposal of Stormwater Runoff Design Guide-lines Manual (USDOT, 1980); refer to that publication for specific design criteria.

3.10.4.2 Drywells

Drywells may be used for subsurface disposal of stormwater, if approved by the County Engineer, and if criteria such as subsurface strata permeability, groundwater levels and maintenance can be satisfactorily addressed. The main cause of drywell failure is clogging of the transmission media (gravel) by silt and debris. Failure can be avoided by utilizing proper design and installation guidelines, and by following recommended maintenance procedures. All drywells must be registered with the Arizona Department of Environmental Quality (ADEQ).

3.10.4.3 Requirements and Criteria

The following list of general requirements and criteria shall be used in the design and construction of engineered basins and drywells (or other methods of subsurface disposal of stormwater). In addition, the engineer is referred to specific policies of the applicable jurisdictional agency.

- The feasibility of subsurface disposal of stormwater at a site by an engineered basin must be documented by field investigations and testing in accordance with ASTM Standard D-3385, Double-Ring Infiltrometer. The field investigation shall include borings at least 10-feet deep to assure that the soils underlying the basin will not impede flow, thereby resulting in water mounding. One test boring shall be provided for each 2500 square feet of basin bottom. The test results shall be de-rated by a factor of 10 to allow for future working conditions in the completed retention facility. This compensation is necessary because actual functional rates will diminish over time due to basin soil compaction, progressive silt loading, and actual stormwater constituents that clog soil pores. A report shall be prepared by a geotechnical or civil engineer, licensed to practice in Arizona, which documents the field investigations, testing, adjusted design rates and the final proposed design. Shallow-pit percolation tests for obtaining permeability rates are not recommended. Refer to the jurisdiction's Policy and Standards Manual for specific requirements in this regard. If shallow-pit percolation tests are used in-lieu of the double-ring infiltrometer test, a de-rating factor of 20 shall be applied.

- The accepted design disposal rate for a drywell is not to exceed 0.1 cfs per well unless a greater rate can be supported by a constant-head percolation test on a completed well at the site. Should this test reflect a higher value, the results shall be de-rated based on the in-situ soil conditions. A de-rating factor of 2 shall be applied for coarse-grained soils (cobbles, gravels and sands). A de-rating factor of 3 shall be applied for fine grained soils (silts and loams). A de-rating factor of 5 shall be applied for clay soils. The maximum allowable rate shall not exceed 0.5 cfs per drywell in any case for design purposes. In accordance with ADEQ requirements, the installation of any subsurface drainage structure must be located at least 10-feet above saturated soils and 100-feet away from any water supply well.

- A test well shall be installed for any retention facility utilizing drywells for stormwater disposal. Upon approval of performance, adjusted as presented above, this test well may then be used as one of the functioning drywells within the retention facility.

- The design of a drywell must include provisions for trapping sediment within a settling chamber. The system shall use a floating absorbent blanket or pillow to enhance the removal of petroleum-based organics floating on the water. A hydrophobic petrochemical absorbent with a minimum capacity of 100 ounces per chamber is recommended. This measure will significantly increase both the efficiency and useful life of the well. Once a year, at a minimum, the settling chamber should be inspected, and it should also be inspected after any major inflow to the drywell. Sediment shall be removed from the chamber at such time that approximately 15 to 20% of the original volume of the chamber is filled. All sediment removed from a settling chamber shall be disposed of either at an authorized sanitary landfill or at any other suitable location approved by the governing jurisdiction.

- Infiltration rates of drywells shall not be used in determining outflow rates in flood-routing procedures. Any retention facility which relies solely upon infiltration as its method of drainage shall be sized to contain the maximum storage volume that would be required without considering an outflow rate.

- Disposal methods using infiltration shall not be permitted for stormwater runoff which carries significant concentrations of sediment. This includes stormwater runoff flowing through sand bed channels, as well as stormwater runoff emanating from a predominantly natural watershed.

- During site development, all drywells shall be securely covered with filter cloth or other material to prevent the introduction of excessive sediment into the settling chamber.

- Retention of runoff emanating from industrial developments and infiltration of runoff to the subsurface will be handled on a case-by-case basis by the appropriate reviewing agency subject to water quality concerns.

- Runoff stored in a retention facility shall be completely drained from the facility within a maximum time period of 36 hours after the runoff event has ended. Drywells that cease to drain a facility within the 36-hour period shall be replaced by the owner with new ones, unless an alternate method of drainage is available.

3.10.5 Criteria for Special Stormwater Storage Methods

Methods of stormwater storage include underground storage, conveyance storage, roadway embankment storage, and storage in parking lots, pedestrian plazas, courtyards and common areas.The use of rooftops as storage areas for runoff is not permitted. Furthermore, basins established in the bottoms of channels are generally not permitted since these are prone to on-going sedimentation problems.

Application of the special measures discussed below is regulated according to specific jurisdictions. Contact the County Engineer before beginning to design using any of these methods.

Since the following methods often result in facilities near buildings, it should be emphasized that the finished floor elevation of a structure shall be a minimum of 1 foot above the 100-year water surface of the stormwater storage facility. The finished floor elevation shall also be above the emergency outfall for the basin.

3.10.5.1 Underground Storage

This type of storage involves the construction of underground tanks, pipes, or vaults, which accept stormwater runoff by means of inlets and storm drain pipes. Due to the high cost of this type of installation, it is generally limited to high-density developments, where surface storage is not feasible due either to the scarcity or high cost of land, or both.

Underground storage facilities must be provided with some method of outfall (that is, gravity drains, pumps, or infiltration). In all cases, manholes (or some other means of access to the underground storage facilities) must be provided for maintenance purposes.

3.10.5.2 Conveyance Storage

During the period that channels and floodplains are filling with runoff, the stormwater is being stored in transient form. This type of storage is known as conveyance storage. Construction of slow velocity channels with large cross sectional areas assist in the accomplishment of such storage. Conveyance storage systems are usually feasible only on large projects, and require detailed hydrologic modeling for analysis.

3.10.5.3 Roadway Embankment Storage

When feasible, use of roadway fill slopes as an embankment for a stormwater storage basin provides an economical means of stormwater storage. Special considerations must be given both to the stability of the embankment and to the protection of the embankment from erosion. Additionally, State of Arizona dam safety requirements may need to be addressed if the embankment height and/or the potential storage volume exceeds certain limitsas set by the State of Arizona.

3.10.5.4 Parking Lot Storage

Using parking lots for stormwater storage is a special case of surface storage. It is an economical option for meeting stormwater storage requirements in high density commercial and industrial developments. Planning of areas within a parking lot, which will accept ponding, should be such that pedestrians are inconvenienced as little as possible.

The maximum depth of ponded water within any parking lot location shall be 1 foot, unless separate approval is obtained from the County Engineer. Deeper ponding, if approved by the County Engineer, should be confined to remote areas of parking lots, whenever possible. Drainage of parking lots can be accomplished by means of drywells (if permitted), curb openings, weirs, storm drains, orifices in walls, or gated outlets.

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.

3.10.5.5 Storage in Plazas, Courtyards and Common Areas

Landscaped common areas, pedestrian plazas and courtyards, which are typically provided in conjunction with high density residential, commercial and office developments, provide opportunities for multiple use as stormwater storage facilities. Such facilities should be designed to minimize public inconvenience, especially during frequent storm events. Public safety issues are also very important with this type of facility. Positive drainage to the outlet structures and trash/debris control must be provided so that the facility drains completely and efficiently.