ARIZONA DEPARTMENT OF TRANSPORTATION

Highway Drainage Design Manual

Volume 3
Hydraulics

ADOT
Intermodal Transportation
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</tr>
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</tr>
</tbody>
</table>
Chapter 1
INTRODUCTION

This chapter contains the following details:

- The organization and use of the ADOT Highway Drainage Design Manual
- Important considerations for successful drainage designs

1.1 INTRODUCTION

1.1.1 Background

The Arizona Department of Transportation (ADOT) Highway Drainage Design Manual (DDM) consists of the following 3 volumes:

- Volume 1 - Policy & Guidelines
- Volume 2 - Hydrology, 2nd Edition
- Volume 3 - Hydraulics, 2nd Edition

Volume 1 of the DDM, contains policies and guidelines for drainage design on ADOT projects. Volume 2 of the DDM, is the 2nd Edition of the Hydrology Manual which contains guidance and procedures for determining design discharges and volumes for drainage design. This 2nd Edition of the Hydraulics Manual is DDM Volume 3 – Hydraulics and provides guidance on procedures and methods for hydraulic design for ADOT drainage facilities.

This DDM Volume 3 - Hydraulics is an update to the first edition which was published in January 2007. The purpose of the update is to incorporate advances by various federal agencies, institutions, and research entities in the field of hydraulics as well as update and revise procedures to match revisions in various source documents, particularly with ADOT design guidelines and construction standard details.

1.1.2 Design Resources from Federal Agencies

The Federal Highway Administration (FHWA) has developed an extensive resource library in support of drainage analysis and design on highway projects. These resources include, among others, the Hydraulic Design Series (HDS) and Hydraulic Engineering Circulars (HEC). Many of the procedures presented in this manual are based on these key source documents. References to specific computer programs, FHWA circulars, guidelines, manuals, and regulations will be noted within the manual. It is expected that the designer will be knowledgeable in the use of the
referenced items. The designer should remain up-to-date on the latest versions of referenced documents by checking the FHWA website or contacting the ADOT Drainage Section.

A comprehensive general drainage reference is FHWA Hydraulic Design Series No. 4 – Introduction to Highway Hydraulics, (HDS 4, 2008) which should be particularly useful for designers and engineers lacking extensive drainage training or experience. More detailed information on each of the topics discussed is provided by other Hydraulic Design Series and Hydraulic Engineering Circulars.

1.2 DESIGN CONSIDERATIONS

During the concept development phase, a drainage plan should be developed that adequately addresses the project goals. The drainage plan should specifically identify the criteria of the many elements to be considered in achieving a design that meets or exceeds the required and/or desired performance.

For the areas under consideration the following questions must be addressed:

- What is the impact of the facility on the existing condition?
- What is the impact of the existing condition on the facility?

In addition to the drainage design itself, a good roadway design will include collaboration between the roadway designer, the drainage designer, and others so that the elements from each discipline are well integrated. The roadway alignment, profile and typical section will impact the design and performance of the cross drainage structure in many ways. The design process should include this collaboration early in the project development before the roadway design is set.

Drainage considerations to be addressed during design are discussed in the following sections.

1.2.1 Stormwater Quantity

Determination of stormwater quantity is necessary for evaluating the impact of a project. Guidance on selection of appropriate methods for determining stormwater quantity is contained in DDM Volume 1 – Policy & Guidelines. The application of methods is discussed in the DDM Volume 2 - Hydrology.

1.2.2 Flood Hazards

Flood flow characteristics at a highway stream crossing should be carefully analyzed to determine their effect upon the highway as well as evaluate the effects of the highway upon the flood flow. Evaluation of flood hazards should include effects to private property both upstream and downstream (i.e., changes to flooding such as overtopping floodwaters diverted onto previously unaffected property or increased ponding that backs up outside of highway right of way) as well as considerations of flow regime and sediment transport characteristics. The primary goal in dealing with off-project flows is to preserve the natural drainage conditions.
1.2.3 Floodplain Encroachment Considerations

A primary drainage consideration for facility sizing of a stream crossing is the evaluation of the impact of floodplain encroachments. This includes both FEMA Regulatory floodplains as well as unmapped floodplains. Hydraulic and environmental considerations of highway river crossings and encroachments are presented in the FHWA Hydraulic Design Series No. 6 - River Engineering for Highway Encroachments, Highways in the River Environment (HDS 6, 2001). HDS 6 provides examples of typical river environments and identifies possible local, upstream and downstream effects of highway encroachments.

The principal factors to be considered when designing a stream crossing that involves encroachment within a floodplain are:

- River type (straight or meandering)
- River characteristics (stable or unstable)
- River geometry and alignment
- Hydrology
- Hydraulics
- Floodplain flow
- Economic (land use and land ownership) considerations

A detailed evaluation of these factors is part of the hydraulics study. Specific crossing components to be determined include:

- The geometry and length of the approaches to the crossing.
- The location of the longitudinal encroachment in the floodplain.
- The amount of allowable longitudinal encroachment into the main channel.
- The type and size of structure, bridge or culvert, and the means to ensure the stability of the structure against flood flows.
- The required river training works to ensure that river flows approach the crossing or the encroachment in a complementary way.

1.2.4 Environmental & Water Quality Coordination

Environmental and water quality considerations are an important component for successful drainage design. ADOT’s Environmental Planning Group (EPG) is responsible for obtaining permits through Section 404/401 of the Clean Water Act. As part of the requirements under ADOT’s Arizona Pollutant Discharge Elimination System (AZPDES) Statewide Stormwater Permit, the Statewide Stormwater Management Plan (SSWMP, 2010) establishes the comprehensive statewide stormwater management program for all ADOT activities and functional units. In addition to the guidance provided in the Highway Drainage Design Manual, guidance developed by the Office of Environmental Services (OES) and the EPG must also be incorporated into the design process. DDM Volume 1 - Policy & Guidelines contains the policies and criteria for drainage design and references the various procedure documents for implementation. As part of the
requirements of the SSWMP, the Water Quality Group within OES has developed a Stormwater Library of guidance manuals for various activities. The OES updates the documents as necessary.

Of particular interest to designers of ADOT stormwater facilities is the *Post-Construction Best Management Practices Manual for Water Quality* (BMP Manual, 2013). The manual is intended to serve as a general guide to assist ADOT staff and consultants in understanding when and where to consider post-construction (i.e., permanent) best management practices (BMPs) for water quality in new construction and redevelopment projects. The manual is most applicable and useful during the planning and preliminary design phases of a highway project; incorporating design standards and maintenance requirements for post-construction BMPs. As such, designers should consult this manual early in the project to identify BMPs that should be incorporated into the design. It is the designer’s responsibility to coordinate with both ADOT Environmental and Drainage Groups.

Another manual within the Stormwater Library is the *Erosion and Pollution Control Manual* (ADOT, 2012). The manual provides guidance to contractors, design professionals, field inspectors, maintenance personnel, ADOT staff and local public officials or staff. The purpose of the manual is to:

- Provide an overview of water quality regulations and permits.
- Outline ADOT’s procedures for complying with water quality regulations and permits.
- Provide guidance for the selection of construction (temporary) BMPs on ADOT construction projects.
- Provide a “tool box” of construction BMPs.

### 1.2.5 Permits

Specific Federal and State drainage permits that will be needed for a highway project must be identified early in the planning stages. Prior to initiating design work, the designer must review the environmental document to identify regulatory commitments, constraints and any required permits. The permits usually required are:

- Stormwater Discharge Permits (NPDES/AZPDES)
- Dredge and Fill Permits (COE 404)

ADOT has prepared the *Clean Water Act 404/401 Guidance Manual* (404/401, 2013) to provide general guidance on the preparation of jurisdictional delineations, Nationwide Permit Pre-Construction Notification submittals, and Individual COE 404 Permit applications.

### 1.2.6 Construction Considerations

Consideration of constructability of drainage features should be a part of the design process. Items that impact constructability include:

- Local availability of materials such as riprap of the size and type required.
• Presence of rock impacting construction of outlet protection and cross-drainage structures.
• Excavation and construction in areas of saturated soils such as high groundwater.
• Phasing of construction when traffic is to be maintained during construction.
• Maintenance of stream base flows or storm runoff occurring during construction.

1.2.7 Maintenance Considerations

Consideration of maintenance of drainage structures should be a part of the design process. Items that impact maintenance include:

• Providing access and adequate room for vehicle maneuvering for maintenance activities.
• Providing access for visual inspection during a storm event.
• Designing to minimize sediment deposition or scour by maintaining natural flow velocities.
• Incorporating details into design to preclude or minimize rill erosion from uncontrolled sheet flows over embankment slopes.
• Utilizing standard and readily available materials for ease of replacement and repair.

1.3 MANUAL UPDATES

1.3.1 Comments

Users of this manual are invited to submit comments, suggestions, or findings of errors. This information should be addressed to:

Chief Drainage Engineer
Arizona Department of Transportation
205 S. 17th Ave. MD 634E
Phoenix, AZ 85007

1.3.2 Updates

Due to the ongoing technical and administrative changes in the field of stormwater management, revisions to this manual will be required from time to time. Such revisions will take place on an ongoing, as-needed, basis and will be posted on ADOT’s website.

1.4 REFERENCES


Chapter 2
DOCUMENTATION

This chapter contains the following details:

- Requirements for drainage reports
- Requirements for calculations and computer results

2.1 OVERVIEW

The ADOT Intermodal Transportation Division administers the Project Development Process which includes the location, design, and construction of new highways and related facilities as well as reconstruction or improvement of the existing system. The Intermodal Transportation Division implements projects from the Five-Year Highway Construction Program, as adopted by the State Transportation Board. The design phase and pre-construction activities include the five stages of design, which represent significant project review milestones. Documentation of hydrologic and hydraulic analysis and design is tied to this 5-stage process. The following review stages are identified in the Project Development Process Manual (ADOT, 2004):

- Stage 1- 15% Review
- Stage 2- 30% Review
- Stage 3- 60% Review
- Stage 4- 95% Review
- Stage 5- Final Plans, Specifications, and Estimates

Documentation of design is the compilation and preservation of the design and related information on which the design is based. This includes drainage area maps, field survey information, source references, photographs, engineering calculations and analyses, computer model output, measured and other data and flood history including narratives from newspapers and individuals such as highway maintenance personnel and local residents who witnessed or had knowledge of an unusual event.

The level of detail required for documentation of drainage varies according to the size and scope of the project. For example, a pavement preservation project will likely require less documentation than a new roadway or major reconstruction. When an existing roadway is being widened or reconstructed, it is important to carefully document the performance of existing drainage features before considering modifications or replacement. Typical project types where drainage analysis and design are required are listed below. The level of detail required for
documentation should be established during project scoping and will depend on whether the project is:

- New construction
- Major reconstruction
- Roadway widening by addition of driving lanes or shoulders
- Pavement preservation
- Drainage improvements

### 2.2 DRAINAGE REPORTS

The ADOT Project Development Process typically requires a separate Drainage Report to document the drainage design development for a project. It is important that the Final Drainage Report be complete and comprehensive. The drainage design development is tied to the design development of the entire project according to the five-stage process described in the previous section. Progress submittals of the Drainage Report include:

- Initial Drainage Report with the Stage 2 review
- Draft Final Drainage Report with the Stage 3 review
- The Final sealed Drainage Report with the Stage 4 review

Following are general requirements related to documentation of hydrologic and hydraulic designs and analyses.

- The amount of detail of documentation for each design or analysis shall be commensurate with the scope and size of the project.
- Documentation shall be organized to be as concise and complete as practicable so that knowledgeable designers can understand years hence what was done.
- Documentation shall include all data and information used for project development.
- Documentation shall be organized into reports that logically lead the reader from past history through the problem background, into the findings and through the design process.
- The report may require an executive summary at the beginning to assist users in finding detailed information.

The drainage report should include all related information and data, criteria, assumptions and judgments, identification of methods and computer programs, calculations, analyses, and results used in developing conclusions and recommendations related to drainage requirements. Discussions shall address inputs, design approach, results and conclusions. Identification of published data, reports, memos, letters and interviews used. If circumstances are such that the drainage facility is sized by other than normal procedures or if the size of the facility is governed by factors other than hydrologic or hydraulic factors, a narrative summary detailing the design analysis shall be included. Additionally, the drainage report should include items not listed herein useful to understanding the analysis, design, findings, and final recommendations.
2.2.1 Initial Drainage Report

The Initial Drainage Report is to be submitted with the Stage 2 submittal. According to the Project Development Process Manual, the purpose of the Initial Drainage Report is:

- To document the methodology and results of the hydrologic analysis and the rationale used in developing the roadway drainage system.
- To define the type, size, and location of cross drainage structures and channels and to determine flood level elevations.
- To determine the initial type, size, and location of the onsite roadway drainage system and to determine outfall location(s).
- To determine preliminary project costs.
- To identify environmental and regulatory floodplain requirements.

2.2.2 Final Drainage Report

The draft Final Drainage Report is to be submitted with the Stage 3 submittal with the final report being submitted with Stage 4. According to the Project Development Process Manual, the purpose of the Final Drainage Report is:

- To finalize the selected drainage system including an analysis of changes to existing flow patterns; design of channels, culverts, and other drainage structures; and location and design of storm sewer systems for on-site and off-site drainage.
- To have plans, profiles, and hydraulic data available for review in the field.

The following are to be included in the Final Drainage Report:

- Final hydraulic design data, analysis, and results for all drainage structures and systems.
- Summaries for all drainage structures including culverts, channels, and storm sewers.
- Design water surface elevation information for culverts and bridges.
- Scour, aggradation and degradation calculations for foundations of major structures.
- Bank protection requirements for drainage structures.

A design/review checklist to assist in report compilation is provided in Appendix 2A. A list of items to be included for the design of various hydraulic structures is provided in Appendix 2B. A sample outline for a drainage report is provided in Appendix 2C.

2.3 REFERENCES

APPENDIX 2A
EXAMPLE CHECKLIST
ADOT ROADWAY DRAINAGE
DRAINAGE REPORT
DESIGN AND REVIEW CHECKLIST

Project Name: ___________________________  Designed/Reviewed By: ___________________________

Type of Project: ___________________________
☐ New Construction  ☐ Major Reconstruction  ☐ Widening for Shoulders or Driving Lane  ☐ Pavement Preservation  ☐ Drainage Repair  ☐ Other

Submittal (Initial, Draft Final, or Final?): ___________________________

Route: __________  Functional Classification __________  Design Storm Frequency __________

Please Check All Items

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<td>1.</td>
<td></td>
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<tr>
<td>Drainage report sealed and dated by Arizona Licensed Professional Engineer.</td>
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<td>2.</td>
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<tr>
<td>A complete set of roadway plans with drainage details is provided.</td>
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<td>3.</td>
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<tr>
<td>Location map/vicinity map showing project begin and end mile posts is included.</td>
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<td>All prior review comments are addressed.</td>
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<td>A USGS Quad map is provided with scale.</td>
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<td>2.</td>
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<tr>
<td>Floodplain delineation map is provided showing FEMA flood hazard zones.</td>
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<td>3.</td>
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<tr>
<td>Land use map is provided (if applicable) for existing and future conditions.</td>
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<tr>
<td>4.</td>
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<tr>
<td>A soils map is provided showing the NRCS soils within the project site.</td>
<td></td>
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<tr>
<td>5.</td>
<td></td>
<td></td>
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<tr>
<td>The correct coordinate system is used for all shapefiles and maps. For horizontal State Plane coordinate system – NAD 83 is used.</td>
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### CURRENT SITE DATA

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<tr>
<td>1.</td>
<td>Topographic survey for the site is provided.</td>
<td></td>
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<tr>
<td>2.</td>
<td>As-built plans for the project area are provided.</td>
<td></td>
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<tr>
<td>3.</td>
<td>Field reconnaissance accounts and photos are provided.</td>
<td></td>
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<tr>
<td>4.</td>
<td>Copies of bridge inspection reports are included for the structures.</td>
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### FLOOD HISTORY

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<tr>
<td>1.</td>
<td>Eye witness accounts of previous flooding.</td>
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<td></td>
</tr>
<tr>
<td>2.</td>
<td>Maintenance records addressing flooding issues and photos.</td>
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### EXTERNAL AGENCIES

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<td>FEMA floodplain (FIS) studies in the project area are included.</td>
<td></td>
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<tr>
<td>2.</td>
<td>Area Drainage Master Plan (ADMP) or Area Drainage Master Study (ADMS) for project area.</td>
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### DESIGN CRITERIA

#### HYDROLOGIC CRITERIA

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<th>YES</th>
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<tbody>
<tr>
<td>1.</td>
<td>Cross drainage design criteria are correctly addressed and specified according to ADOT Drainage Policy and Guidelines.</td>
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<tr>
<td>2.</td>
<td>Is NOAA 14 data the source for precipitation data?</td>
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<tr>
<td>3.</td>
<td>Pavement drainage design criteria are correctly specified.</td>
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<tr>
<td>4.</td>
<td>Channel and ditch design criteria are correctly specified.</td>
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<tr>
<td>5.</td>
<td>Detention/retention basin design criteria are specified.</td>
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<tr>
<td>6.</td>
<td>Pump station design criteria is specified according to ADOT guidelines.</td>
<td></td>
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<tr>
<td>7.</td>
<td>Are FEMA/local agency’s discharges governing for the design?</td>
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</table>
**HYDRAULIC CRITERIA**

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<th>NA</th>
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<tbody>
<tr>
<td>1.</td>
<td>Allowable spread is specified and met.</td>
<td></td>
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<tr>
<td>2.</td>
<td>For culverts, the allowable headwater and overtopping depth are specified.</td>
<td></td>
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<tr>
<td>3.</td>
<td>The criteria for minimum and maximum velocity where applicable per hydraulics manual are specified.</td>
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**HYDROLOGY**

<table>
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<tr>
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<tbody>
<tr>
<td>1.</td>
<td>Drainage map is provided with basins and sub-basins delineated.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.</td>
<td>Basin model, meteorological model and control specifications are all set up and entered correctly.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.</td>
<td>Selection of Green and Ampt parameters correspond to ADOT Hydrology Manual methodology. (Initial storage, conductivity, etc.)</td>
<td></td>
<td></td>
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<tr>
<td>4.</td>
<td>Transform method used is the Clark Unit Hydrograph. Storage and time of concentration are computed correctly.</td>
<td></td>
<td></td>
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<tr>
<td>5.</td>
<td>Appropriate routing methodology per ADOT Hydrology Manual is used (Muskingum Cunge).</td>
<td></td>
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<tr>
<td>6.</td>
<td>Are diversions recognized and modeled correctly?</td>
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<td>7.</td>
<td>For storage routing, ensure the outlet structures are represented and storage methods are input correctly.</td>
<td></td>
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<tr>
<td>8.</td>
<td>Are the peak discharges reasonable for the watershed characteristics (Size, roughness, etc.)?</td>
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<tr>
<td>9.</td>
<td>Are the input data and output results clearly shown in tables?</td>
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**HYDRAULIC CRITERIA**

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**HYDROLOGY**

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<tr>
<td>8.</td>
<td>Are the peak discharges reasonable for the watershed characteristics (Size, roughness, etc.)?</td>
<td></td>
<td></td>
</tr>
<tr>
<td>9.</td>
<td>Are the input data and output results clearly shown in tables?</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**HYDRAULIC CRITERIA**

<table>
<thead>
<tr>
<th></th>
<th>NA</th>
<th>YES</th>
<th>NO</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Allowable spread is specified and met.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.</td>
<td>For culverts, the allowable headwater and overtopping depth are specified.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.</td>
<td>The criteria for minimum and maximum velocity where applicable per hydraulics manual are specified.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**HYDROLOGY**

<table>
<thead>
<tr>
<th></th>
<th>NA</th>
<th>YES</th>
<th>NO</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Drainage map is provided with basins and sub-basins delineated.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.</td>
<td>Basin model, meteorological model and control specifications are all set up and entered correctly.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.</td>
<td>Selection of Green and Ampt parameters correspond to ADOT Hydrology Manual methodology. (Initial storage, conductivity, etc.)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.</td>
<td>Transform method used is the Clark Unit Hydrograph. Storage and time of concentration are computed correctly.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.</td>
<td>Appropriate routing methodology per ADOT Hydrology Manual is used (Muskingum Cunge).</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6.</td>
<td>Are diversions recognized and modeled correctly?</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7.</td>
<td>For storage routing, ensure the outlet structures are represented and storage methods are input correctly.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8.</td>
<td>Are the peak discharges reasonable for the watershed characteristics (Size, roughness, etc.)?</td>
<td></td>
<td></td>
</tr>
<tr>
<td>9.</td>
<td>Are the input data and output results clearly shown in tables?</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
3. Check that the minimum time of concentration used for Rational Method is 10 minutes and maximum is 60 minutes.

4. The C-value is obtained correctly from the ADOT Hydrology Manual.

5. The $K_b$ value is selected appropriately and in accordance with ADOT hydrology manual methodology.

<table>
<thead>
<tr>
<th>HYDROLOGY – FLOOD FREQUENCY ANALYSIS</th>
<th>NA</th>
<th>YES</th>
<th>NO</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Is gage data available for the site and is it considered?</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2. Is flood frequency analysis completed according to ADOT Hydrology Manual?</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>HYDROLOGY – REGRESSION ANALYSIS</th>
<th>NA</th>
<th>YES</th>
<th>NO</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Are USGS regional regression equations used with approval of ADOT?</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>HYDRAULICS</th>
<th>STORM DRAINS AND PAVEMENT DRAINAGE</th>
<th>NA</th>
<th>YES</th>
<th>NO</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Check HGL and ensure is 6&quot; below grate or manhole.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2. Is minimum pipe size used per ADOT Drainage Guidelines?</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3. Ensure the correct clogging factors/capture ratios are applied.</td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>4. Ensure all catch basins are analyzed correctly with proper cross-slope, gutter slope, longitudinal slope, local depression and Manning’s ‘n’.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5. Is outlet protection shown for the outfall?</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6. If StormCAD is used, check the conduit and structure tables and ensure all the results are addressed in the table.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7. For StormCAD, ensure that an engineering profile is included that displays the HGL and EGL.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8. Inlet spacing is completed according to design criteria.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9. Are flanking inlets designed properly at the sag locations?</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10. Do pipe profiles make sense? E.g., no 30’ deep catch basin, etc.?</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
11. Does the pipe layout make sense or is better layout possible? 

<table>
<thead>
<tr>
<th>RETENTION AND DETENTION BASINS</th>
<th>NA</th>
<th>YES</th>
<th>NO</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Is first flush containment required and addressed?</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2. Ensure side slopes and depths of the basin are per ADOT Guidelines.</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>3. If retention basin, does the basin evacuate within 36 hours?</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4. Is the basin accessible for maintenance?</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5. Retention basin volume requirement is met.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6. Is emergency spillway provided at the proper location to discharge to historic flow path?</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7. Are safety mitigations required (railing, etc.)?</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8. For detention, the proper outlet structure is provided and modeled correctly.</td>
<td></td>
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<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>CULVERTS</th>
<th>NA</th>
<th>YES</th>
<th>NO</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Is culvert information clearly shown (length, diameter, slope and material)?</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2. Is ADOT Pipe Selection Guidelines used as the basis for design? i.e. pH value, fill height, thickness and class of pipe.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3. End section or headwall provided according to ADOT requirement.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4. Are clear zone limitations considered and incorporated?</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>5. Pipe profile information used in the analysis matches the plans.</td>
<td></td>
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</tr>
<tr>
<td>6. Review if the culvert is analyzed correctly in HY-8.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7. For culvert extension, is the condition of the existing culvert evaluated for continued service (e.g., 100 years for concrete, 75 years for plastic, and for CMP service life depends on gage and coating)?</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8. Is design headwater elevation for $Q_{\text{design}} \leq 3''$ below edge of pavement elevation or at the elevation at which flow diverts?</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9. Ensure that the HW/D ratio $\leq 1.5$.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Question</td>
<td>NA</td>
<td>YES</td>
</tr>
<tr>
<td>---</td>
<td>-------------------------------------------------------------------------</td>
<td>----</td>
<td>-----</td>
</tr>
<tr>
<td>10</td>
<td>Is the culvert evaluated for overtopping to determine overtopping flow, frequency, overtopping elevation?</td>
<td></td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>Ensure that overtopping frequency is calculated.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>Is headwater elevation for Q\textsubscript{100} evaluated for backwater impacts to adjacent properties?</td>
<td></td>
<td></td>
</tr>
<tr>
<td>13</td>
<td>Is an easement recommended based on impacts?</td>
<td></td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>The basis for tailwater elevation is reasonable.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>Is outlet protection necessary and designed according to ADOT guidelines?</td>
<td></td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>Are the appropriate Manning’s ‘n’ values used per ADOT Hydraulics Manual and Pipe Selection Guidelines?</td>
<td></td>
<td></td>
</tr>
<tr>
<td>17</td>
<td>If total span is greater than 20’, a structure number requested from Bridge Group and is assigned?</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td><strong>CHANNELS / DITCHES</strong></td>
<td>NA</td>
<td>YES</td>
</tr>
<tr>
<td>1</td>
<td>Do side slopes and longitudinal slope meet ADOT Guidelines per Table 9.1 of drainage policy manual?</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Is channel lining needed?</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Is channel bottom width selection coordinated with Maintenance District personnel?</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Channel has a bottom cross-slope to one side to concentrate flow.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Are access points provided within reasonable distances and coordinated with Maintenance District personnel?</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>Freeboard requirements are stipulated and are met.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>Are channel transitions accounted for and addressed in the analysis?</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>Is the channel protected from concentrated flows causing erosion behind the slopes?</td>
<td></td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>Are cut and fill transitions protected and addressed in the report?</td>
<td></td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>Are the appropriate Manning’s ‘n’ values used?</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### 10. Is a check dam used along the ditch for erosion protection?

### PLANS

<table>
<thead>
<tr>
<th></th>
<th>- PLANS/ PROFILES/ PIPE SUMMARY -</th>
<th>NA</th>
<th>YES</th>
<th>NO</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Check consistency in labels and callouts between plans, profiles and pipe summary sheets.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.</td>
<td>Are detail drawings of all necessary drainage components provided and are they accurate?</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.</td>
<td>For culvert details, profiles are cut along the stream thalweg; not just straight through the culvert.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.</td>
<td>Are storm drain conflicts with utilities identified and addressed.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.</td>
<td>All rim and grate elevations on pipe profiles match with pipe summary sheet information and hydraulic design computations.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6.</td>
<td>Does the pipe profile information match hydraulic design?</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7.</td>
<td>Do the culvert detail sheets show $Q_{\text{design}}$, ($Q_{100}$, or $Q_{\text{overtopping}}$, whichever is lower) and corresponding headwater elevations.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8.</td>
<td>Flow direction is shown on ditches and channels with arrows.</td>
<td></td>
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<td></td>
</tr>
</tbody>
</table>

If used for QA/QC, by signing below I certify that the QC procedure and checklist were followed.

<table>
<thead>
<tr>
<th>Checker</th>
<th>Designer</th>
</tr>
</thead>
<tbody>
<tr>
<td>Signature: ______________________</td>
<td>Signature: ______________________</td>
</tr>
<tr>
<td>Date: ______________________</td>
<td>Date: ______________________</td>
</tr>
</tbody>
</table>

By signing below, I certify that the QA procedure was followed.

<table>
<thead>
<tr>
<th>QA Manager</th>
<th>Signature</th>
<th>Date</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Signature</td>
<td>Date</td>
</tr>
</tbody>
</table>
APPENDIX 2B
DESIGN PROCESS DOCUMENTATION
DESIGN PROCESS DOCUMENTATION

Introduction

The following design inputs and results shall be included in the documentation file. The intent is not to limit the data to only those items listed, but rather identify minimum requirements, as appropriate, consistent with the hydraulic design procedures as outlined in this manual. Inclusion of calculations and computer data should be organized into appendices with a summary of results included in the body of the report.

Circumstances may warrant or require special solutions not addressed by routine forms or formulas. In such cases, the report should reference the formula used and the source. If necessary, a typical calculation may be shown in detail to clarify the application of the formula and logic of the solution. The results of the calculations are to be presented in tabular summary form in the body of the report with backup calculations in the appendix. Summary forms should provide space for each of the critical variables used in the calculations.

Hydrology

- Methodology
- Contributing watershed area size and identification of source (map name, etc.)
- Design frequencies used
- Identification of design values, how they are determined, and discussion of any unusual variance from normal usage
- Soil characterization
- Development characterization (land use)
- Rainfall amount and distribution
- Hydrograph parameters (time of concentration, storage coefficient)
- Discharges for the design frequencies

Bridges

- Roughness coefficient (n-value) assignments
- Observed highwater, dates, and discharges
- Potential flood hazards to adjacent properties
- Existing roadway geometry (plan and profile)
- Proposed roadway geometry (plan and profile)
- Cross-section(s) used
- Identification of the method used for computation of water surface elevations
- Stage-discharge curve for existing and proposed conditions for the design frequencies
- Through-bridge and channel velocity estimates for the design frequencies
- Calculated backwater, velocity, and scour for the design frequencies
- Magnitude and frequency of overtopping flood, if applicable
- FEMA floodplain impacts
- Bridge scour elevation
- Copies of all computer analyses
- Economic analysis of design and alternatives

**Culverts**
- Allowable headwater elevation and basis for its selection
- Roughness coefficient assignments (n-values)
- Observed highwater, dates and discharges
- Potential flood hazard to adjacent properties
- Existing roadway geometry (plan and profile)
- Proposed roadway geometry (plan and profile)
- Cross-section(s) used for the downstream channel tailwater elevations
- Culvert and entrance type
- Stage discharge information for existing and proposed conditions for the design frequencies
- Outlet velocity predictions for the design frequencies
- Predicted scour for the design frequencies
- Culvert outlet appurtenances and energy dissipation calculations and designs
- Copies of all computer analyses

**Open Channels**
- Observed highwater, dates, and discharges
- Cross-section(s) used in the design water surface determinations and their locations
- Roughness coefficient assignments (n-values), existing and proposed conditions
- Identification of the method used for computation of water surface elevations
- Channel velocity and locations determinations
- Stage discharge curves for the design frequencies
- Water surface profiles through the reach for the design frequencies
- Design or analysis of materials proposed for the channel bed and banks for the design frequencies
- Energy dissipation calculations and designs for the design frequencies
- Copies of all computer analyses

**Storm Drains**
- Complete drainage area map
- Design frequency
- Information concerning outfalls and existing storm drains
- Information concerning utilities
- Schematic of storm drain system layout
- Computations for inlets and pipes, including hydraulic grade lines
Pump Stations

- Maximum allowable headwater elevations
- Inflow design hydrograph from drainage area to pump
- Sump dimensions
- Available storage volumes
- Pump sizes and operations
- Starting sequence and elevations
- Pump calculations and design report
- Dedicated storage capacity

Detention/Retention Basins

- Maximum allowable headwater elevations
- Inflow design hydrograph
- Basin dimensions
- Stage-storage curve
- Stage-discharge curve
- Outflow hydrograph
APPENDIX 2C

DRAINAGE REPORT OUTLINE
DRAINAGE REPORT OUTLINE

1. EXECUTIVE SUMMARY

2. INTRODUCTION
   2.1. General
   2.2. Study Area
   2.3. Existing Data and Reports

3. DESIGN CRITERIA
   3.1. Applicable Standards
   3.2. Rainfall
   3.3. Roadway Inundation
   3.4. Catch Basins
   3.5. Storm Drains
   3.6. Open Channels
   3.7. Detention/Retention Basins
   3.8. Culverts
   3.9. Bridges
   3.10. Pump Stations

4. HYDROLOGY
   4.1. Methodology
      4.1.1. Basis of Design Discharges
      4.1.2. Rational Method Hydrology
      4.1.3. HEC-HMS Hydrology
      4.1.4. Two-Dimensional Flow Modeling
   4.2. Watershed Characteristics
   4.3. Design Assumptions
   4.4. Results

5. HYDRAULICS
   5.1. Culverts & Outlet Protection
      5.1.1. Methodology
      5.1.2. Assumptions
      5.1.3. Post Construction Best Management Practices
      5.1.4. Results and Recommendations
         5.1.4.1. Existing Conditions
         5.1.4.2. With Project Conditions
         5.1.4.3. Outlet Protection
   5.2. Bridges (Reference to separate Bridge Hydraulics Report(s))
5.2.1. Methodology
5.2.2. Assumptions
5.2.3. Post Construction Best Management Practices
5.2.4. Results

5.3. Channels & Bank Protection
5.3.1. Methodology
5.3.2. Assumptions
5.3.3. Post Construction Best Management Practices
5.3.4. Results and Recommendations

5.4. Roadway Drainage
5.4.1. Methodology
5.4.2. Assumptions
5.4.3. Post Construction Best Management Practices
5.4.4. Results and Recommendations

5.5. Floodplain Analysis
5.5.1. Regulatory Floodplains
5.5.2. Non-Regulatory Floodplains

5.6. Hydraulic Structures
5.7. Pump Stations

6. REFERENCES
7. APPENDICES
Chapter 3

CHANNELS AND BANK PROTECTION

This chapter contains details on the following:

- Purpose, analysis, and design of channels associated with transportation projects
- Types, constraints, and design of channel bank linings and bank protection

3.1 INTRODUCTION

Channels are used within transportation projects to convey stormwater runoff originating outside of the project right-of-way (“off-site” runoff) and runoff originating from within the project right-of-way (“on-site” runoff) to a suitable outfall in a safe and efficient manner.

This chapter presents guidance and reference to technical resources for design of open channels. Elements in this process include principles of fluid mechanics, hydraulic analysis as well as assessment and design of linings or other features to prevent erosion of channel banks and scour of the channel bed.

3.1.1 Definitions

Open channels in this document are classified as natural or constructed channels. Natural channels are existing features formed over time by natural geomorphic processes. Constructed channels are man-made channels that may or may not follow a natural or historic flow path.

Natural Channels

Natural channels may also be referred to as stream channels or washes. They may contain vegetation and may change dramatically in physical characteristics along their lengths.

Constructed Channels

Constructed channels are also referred to as man-made, engineered, or artificial channels. These channels range from drainage channels adjacent to urban freeways to rural roadside ditches and swales. Constructed channels may be lined or unlined depending upon project needs and design constraints.

The terms “roadside channel” or “ditch” are used for those elements that collect and convey surface or sheet flow of stormwater runoff from the highway and adjacent lands. Usually the alignment and profile of roadside channels and ditches is governed by the highway cross-section. Roadside channels may convey off-site runoff, on-site runoff or both.
A stable channel is one in which lateral and vertical movement of the channel bed and banks are constrained. Methods used to create a stable channel are defined by the following categories.

**Lining**

“Channel bank lining” refers to the practice of armoring channel banks to prevent erosion while leaving the channel bed unlined. The channel bank lining is also referred to as “revetment”.

“Continuous channel lining” refers to the practice of continuously armoring the total cross-section of channel bed and banks to prevent erosion.

**Bank Protection**

Bank protection describes the use of training structures, such as spurs and guide banks (spur dikes) to re-orient flow away from channel banks or highway embankments to prevent erosion.

**3.1.2 Design Resources**

Hydraulic design of channels and bank protection is described in a number of Federal Highway Administration documents. The primary FHWA resources used in this chapter are the following:

- **River Engineering for Highway Encroachments, Hydraulic Design Series No. 6** (HDS 6, 2001) has proven to be a singularly authoritative document for the design of highway associated hydraulic structures in natural channels with moveable boundaries. Hydraulic problems at stream crossings are described in detail and the hydraulic principles of rigid and moveable boundary channels are discussed.

- **Design of Riprap Revetments, Hydraulic Engineering Circular No. 11** (HEC 11, 1989) provides design guidance for rock riprap, rubble riprap, gabions, preformed blocks, grouted rock, and paved linings. Design guidance is also presented for wire-enclosed rock (gabions), precast concrete blocks, and concrete paved linings. Although HEC 23 provides more current guidance on many applications contained in HEC 11, the guidance on grouted riprap and stacked gabions from HEC 11 is still referenced in this chapter.

- **Hydraulic Design of Energy Dissipators for Culverts and Channels, Hydraulic Engineering Circular No. 14** (HEC 14, 2006) provides design information for analyzing and mitigating energy dissipation problems at culvert outlets and in open channels. HEC 14 guidance utilized in this chapter is from Chapter 6, Hydraulic Jump and Chapter 11, Drop Structures.

- **Design of Roadside Channels with Flexible Linings, Hydraulic Engineering Circular No. 15** (HEC 15, 2005) provides procedures for design of flexible channel linings in constructed roadside channels, swales, and ditches with flow depths up to five feet.

- **Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance, Hydraulic Engineering Circular No. 23** (HEC 23, 2009) identifies and provides design guidelines for bridge scour and stream instability countermeasures based on the results of recently completed National Cooperative Highway Research Program (NCHRP) projects and include: riprap design criteria, specifications, and quality control. Volume 2 contains four detailed design guidelines for countermeasures for streambank...
and roadway embankment protection used for this chapter. They include Riprap Revetment (Guideline 4), Wire Enclosed Riprap Mattress (Guideline 6), Soil-cement (Guideline 7), and Gabion Mattresses (Guideline 10).

Primary guidance documents produced by ADOT and Arizona Department of Water Resources referenced in this chapter are:

- **Watercourse System Sediment Balance, Arizona State Standard SS5-96** (ADWR, SS5-96) is used for estimating long term scour and degradation in natural channels.
- **Scour at Sill Structures**, ADOT (Laursen and Flick, 1983) is used for computing scour at drop structures.
- **Urban Highways Channel Lining Design Guidelines** (ADOT, 1989) provides guidance for design of reinforced concrete channel linings. (Appendix 3D)

These documents contain all applicable theory, equations, charts, and figures needed for analysis and design of channels and bank protection. Throughout this chapter the reader will be directed to the appropriate documents along with Arizona specific guidance in their application. Therefore, equations, procedures, and figures from these readily available documents will not be included in this chapter. The foregoing is a listing of primary guidance documents that the reader should have on hand for use in application of this chapter. As such, it is not a comprehensive list of all documents referenced within the chapter. For a complete list of references see Section 3.8.

### 3.2 STREAM MORPHOLOGY

The form assumed by a natural stream channel, which includes its cross-sectional shape and its planform, is a function of many variables for which cause-and-effect relationships are difficult to establish. The stream may be in equilibrium with respect to long time periods, which means that on average it discharges the same amount of sediment that it receives, although there may be short-term adjustments in its bedforms in response to seasonal flood flows. In contrast, the stream reach of interest may be aggrading or degrading as a result of deposition or scour in the reach, respectively. The planform of the stream may be straight, braided or meandering. These complexities of stream morphology can be assessed by inspecting aerial photographs and topographic maps for changes in slope, width, depth, meander form and bank erosion with time.

A qualitative assessment of the river response to proposed highway facilities is possible with a thorough knowledge of river mechanics and accumulation of engineering experience. General guidance on these topics may be found in HEC 20 *Stream Stability at Highway Structures* (HEC 20, 2012).

Equilibrium sediment load calculations can be made by a variety of techniques and compared from reach to reach to detect an imbalance in sediment inflow and outflow and thus identify an aggradation/degradation problem. Section 3.5.3.3 identifies techniques which can be used to evaluate sedimentation within a channel system. *River Engineering for Highway Encroachments* (HDS 6, 2001) provides guidance for evaluating sedimentation problems and identifying effective mitigation measures. The proposed methodology should be approved by ADOT Drainage Section.
The natural stream channel will assume a geomorphic form compatible with the sediment load and discharge history, which it has experienced over time. To the extent that a highway structure disturbs this delicate balance by encroaching on the natural stream channel, the consequences of flooding, erosion and deposition can be significant and widespread. The hydraulic analysis of a proposed highway structure should include consideration of the extent of these consequences.

3.2.1 Degradation

Degradation in streams can cause the loss of bank protection and lateral channel movement due to caving banks. Changes in bed vertical alignment, such as construction of a below-grade culvert, can propagate bed degradation upstream as a migrating headcut. Other common causes are in-stream material extraction and changes in sediment supply or transport.

Estimating long-term degradation is important for the design of in-channel features such as bank lining, bank protection, culvert outlets, bridge features, etc. Methods for estimating long-term scour are described in Arizona Department of Water Resources State Standard SS5-96 Watercourse System Sediment Balance (ADWR, SS5-96).

If long-term degradation estimates are prohibitively large, mitigation measures may be used. A grade control structure (also known as a check dam), which is a sill constructed across a channel, is one of the most successful techniques for halting degradation. Grade control structure design considerations are discussed in Section 3.5.2.2. Channel lining may be effective at preventing degradation, but only if the extent of the lining continues to the degradation source or a “hard point” at which degradation is limited (such as a culvert crossing or grade control structure).

3.2.2 Aggradation

Aggradation describes a sediment imbalance in which sediment supply exceeds transport. In Arizona, aggradation often occurs in constructed channels during events of a lesser magnitude than the design event.

Measures to alleviate aggradation problems at highways include use of an incised low-flow channel, bridge modification, continued maintenance or combinations of these. In practice, most ADOT structures experiencing aggradation rely upon maintenance to mitigate impacts.

Sheet flow may occur in overland aggradational areas. Arizona Department of Water Resources State Standard SS4-95 Identification of and Development within Sheet Flow Areas (ADWR, SS4-95) provides sheet-flow analysis methods.

3.2.3 Stream Bank Erosion

Erosion of stream banks can develop due to a variety of causes. In the context of the stream system, these causes may be meander migration, channel avulsion, or simply passage of a discharge greater than the geomorphic channel forming event. An understanding of the overall channel system is necessary to identify potential for stream bank erosion and design of appropriate countermeasures.
3.3 OPEN CHANNEL FLOW

Analysis and design of both natural and constructed channels proceeds according to the basic principles of open-channel flow (see Chow, 1959 and Henderson, 1966). The basic principles of fluid mechanics—continuity, energy, and momentum—can be applied to open-channel flow with the additional complication that the position of the free surface is usually one of the unknown variables. The determination of this unknown is one of the principal (or primary) objectives of open-channel flow analysis.

3.3.1 Flow Classification

Open channel flow can be classified as follows:

**Steady Flow:**
1. Uniform Flow
2. Non-uniform Flow
   a. Gradually Varied Flow
b. Rapidly Varied Flow

Unsteady Flow:

1. Unsteady Uniform Flow (rare)
2. Unsteady Non-uniform Flow
   a. Gradually Varied Unsteady Flow
   b. Rapidly Varied Unsteady Flow

The steady, uniform flow case and the steady, non-uniform flow case are the most fundamental types of flow treated in highway hydraulics.

3.3.1.1 Steady and Unsteady Flow

A steady flow is one in which the discharge passing a given cross-section is constant with respect to time. The maintenance of steady flow in any reach requires that the rates of inflow and outflow be constant and equal. When the discharge varies with time, the flow is unsteady.

3.3.1.2 Uniform Flow and Non-uniform Flow

A non-uniform flow is one in which the velocity and depth vary in the direction of motion, while they remain constant in uniform flow. Uniform flow can only occur in a prismatic channel, which is a channel of constant cross-section, roughness and slope; however, non-uniform flow can occur either in a prismatic channel or in a natural channel with variable properties.

3.3.1.3 Gradually Varied and Rapidly Varied Flow

A non-uniform flow in which the depth and velocity change gradually enough in the flow direction that vertical accelerations can be neglected is referred to as a gradually varied flow; otherwise, it is considered to be rapidly varied.

3.3.2 Continuity Equation

The continuity equation is the statement of conservation of mass in fluid mechanics. For the special case of steady flow of an incompressible fluid, it assumes the simple form:

\[ Q = A_1 V_1 = A_2 V_2 \]  

where: 
\[ Q \] = discharge (ft³/s),  
\[ A \] = cross-sectional area of flow (ft²), and  
\[ V \] = mean cross-sectional velocity (ft/s), (which is perpendicular to the cross-section).

The subscripts 1 and 2 refer to successive cross-sections along the flow path.
3.3.3 Manning’s Equation

For a given depth of flow in an open channel with a steady, uniform flow, the mean velocity, \( V \), can be computed with Manning’s equation:

\[
V = \left(\frac{1.486}{n}\right) R^{2/3} S^{1/2}
\]

where: \( V \) = velocity (ft/s), 
\( n \) = Manning’s roughness coefficient (dimensionless), 
\( R \) = hydraulic radius = \( A/P \) (ft), 
\( P \) = wetted perimeter (ft), and 
\( S \) = slope of the energy grade line, (ft/ft). Note: For steady uniform flow, \( S = \) channel slope.

The selection of Manning’s \( n \) is generally based on observation; however, considerable experience is essential in selecting appropriate \( n \)-values. The selection of Manning’s \( n \) is discussed in Section 3.4.2. Tables of typical \( n \)-values may be found in Appendix 3A for natural and constructed channels.

The continuity equation can be combined with Manning’s equation to obtain the steady, uniform flow discharge as:

\[
Q = \left(\frac{1.486}{n}\right) AR^{2/3} S^{1/2}
\]

For a given channel geometry, slope and roughness and a specified value of discharge \( Q \), a unique value of depth occurs in steady, uniform flow. It is called normal depth and is computed from Equation 3.3 by expressing the area and hydraulic radius in terms of depth. The resulting equation may require a trial-and-error solution. See Section 3.4.3 for a more detailed discussion of the computation of normal depth.

3.3.4 Energy Principles

While important for all analyses of open channel flow, energy principles are especially important for the design of supercritical flow channels.

Flowing water possesses energy of two forms: potential and kinetic. Potential energy at a particular point is represented by the depth of the water plus the elevation of the channel bottom above a datum (elevation head). The plot of the potential energy head from one cross-section to the next defines the hydraulic grade line. For open channel flow, the hydraulic grade line is coincident with the water surface. The kinetic energy is represented by the velocity head. The total energy head or specific energy is the sum of potential energy head and kinetic energy head.

The relationship between potential energy and kinetic energy for a given discharge varies as the depth of flow varies. This relationship is illustrated on the specific energy diagram (Figure 3–2).
Specific energy is the sum of the potential and kinetic energy components according to the following equation.

\[ E = \frac{V^2}{2g} + y \]

where

- \( E \) = specific energy (ft),
- \( V \) = average velocity (fps),
- \( g \) = 32.2 ft/s, and
- \( y \) = flow depth (ft).

**Figure 3–2 Specific Energy Diagram for Rectangular Channels (HDS 6, 2001)**

The ratio of the inertia forces and gravity forces of the flow is the Froude number. The specific energy relationship and the importance of the Froude number are described in standard hydraulics textbooks and won’t be elaborated here, however, for design of open channels, it is important to understand that the condition of minimum specific energy \( (E_c) \) occurs with a Froude number of one. The flow depth associated with this condition is called critical depth \( (y_c) \). If the normal depth is greater than critical depth, the slope is classified as a mild slope, while on a steep slope, the normal depth is less than critical depth. Thus, uniform flow is subcritical on a mild slope and supercritical on a steep slope. When flow is near critical depth it has a tendency to fluctuate between sub- and supercritical flow which results in an unstable flow condition where flow depth
fluctuates significantly creating surface waves. As a result, channel design should ensure that the Froude number is not close to 1. The channel should be designed as either fully supercritical or fully subcritical. When supercritical flow transitions to subcritical flow it passes through critical depth and creates a hydraulic jump. The hydraulic jump can be used in channel design to force a transition from supercritical to subcritical flow and is an effective means of dissipating energy.

3.3.5 Momentum and the Hydraulic Jump

In classical physics, momentum is the product of mass and velocity and is a vector quantity, but in fluid mechanics it is treated as a longitudinal quantity (i.e. one dimension) evaluated in the direction of flow. Additionally, it is evaluated as momentum per unit time, corresponding to the product of mass flow rate and velocity, and therefore it has units of force. The most common form of the momentum equation is,

\[ F = \rho Q (V_2 - V_1) \]

where:
- \( F \) = force (lbs),
- \( \rho \) = fluid density (slugs/ft\(^3\)),
- \( Q \) = Discharge (ft\(^3\)/s), and
- \( V_1, V_2 \) = Velocity at upstream and downstream section (ft/s).

The momentum forces considered in open channel flow are dynamic force – dependent on depth and flow rate – and static force – dependent on depth – both affected by gravity. The principle of conservation of momentum in open channel flow is applied in terms of specific force, or the momentum function. The conjugate depth equation, which describes the depths on either side of a hydraulic jump, can be derived from the conservation of momentum in rectangular channels, based upon the relationship between momentum and depth of flow.

Hydraulic jumps are an important consideration in channel design when channel Froude numbers are near 1 or exceed 1 as these conditions indicate the potential for development of a hydraulic jump. The energy from a hydraulic jump can be highly erosive. The length of a hydraulic jump is influenced by the contrast between upstream supercritical flow and downstream subcritical flow and has particular application at channel or structure transitions such as flow contractions, flow expansions, and culvert outlets.

The jump occurs as an abrupt transition from supercritical to subcritical flow in the flow direction. There are significant changes in depth and velocity in the jump and significant energy is dissipated. For this reason, the hydraulic jump is often employed to dissipate energy and control erosion at highway drainage structures under specifically designed conditions.

A hydraulic jump will not occur until the ratio of the supercritical flow depth \( y_1 \) in the approach channel to the subcritical flow depth \( y_2 \) in the downstream channel reaches a specific value. The depth before the jump is called the initial depth \( y_i \), and the depth after the jump is the sequent depth \( y_s \). The downstream water surface must be close to the sequent depth \( y_s \) for a good jump to form. If the downstream water surface is too low the jump will “wash out” and not form. If the downstream water surface is too high the jump will be drowned out and will not
form. In both cases, the efficiency of the jump is greatly diminished and it is possible that downstream velocities will be higher than if a good jump had formed. When a hydraulic jump is used as an energy dissipator, controls to ensure sufficient tailwater depth are often necessary to control the location of the jump and to ensure that a jump will occur during the desired range of discharges. Sills can be used to control a hydraulic jump if the tailwater depth is less than the sequent depth. If the tailwater depth is higher than the sequent depth, a drop in the channel floor must be used to ensure that a jump forms (see Chow, 1959 and Chapter 6 of HEC 14, 2006). The location of hydraulic jumps should be identified in the design and the channel should be designed to withstand the energy associated with the hydraulic jump.

3.3.6 Shear Stress

Shear stress is the force water exerts on the bed and bank of a channel as it flows over them. Assuming steady-uniform flow, average shear stress at the bed of the channel may be simply expressed as:

\[ \tau_o = \gamma R S_o \]  

where \( \tau \) = shear stress at bed (lbs/ft\(^2\)),
\( \gamma \) = the density of the water/sediment flow (lbs/ft\(^3\)),
\( R \) = the hydraulic radius (ft), and
\( S_o \) = the longitudinal bed slope (ft/ft).

Shear stress is one of the most commonly used parameters to assess the suitability of a channel bank lining material. The bank lining should be designed such that it can resist the shear stress imposed by the design flow with a suitable factor of safety.

3.4 HYDRAULIC ANALYSIS

The hydraulic analysis of a channel determines the depth and velocity at which a given discharge will flow in a channel of known geometry, roughness and slope. The depth and velocity of flow are necessary for the design or analysis of channel linings, bank protection and highway drainage structures.

Two methods are commonly used in hydraulic analysis of open channels.

1. The single-section method is a simple application of Manning’s equation. This method is often used to determine tailwater rating curves for culverts, analyze roadside channels, or for other situations in which uniform or nearly uniform flow conditions exist and backwater impacts are negligible.

2. The step-backwater method is used to compute the complete water surface profile in a stream reach, to evaluate the water surface elevations for bridge hydraulic design or to analyze other gradually varied flow conditions in streams.

While step-backwater analysis is generally recommended for complex analysis/design in channels of varying geometry, the single-section method is all that is necessary for some design
(e.g., uniformly sloped and shaped channels such as a standard roadside channel or ditch, culverts, storm drain, outfalls).

Depending on the complexity of the design, the designer may need to use special analysis techniques, a more detailed method of analysis than either the single-section method or the computation of a water surface profile using the step-backwater method. Special analysis techniques include two-dimensional analysis, water and sediment routing, and unsteady flow analysis.

### 3.4.1 Cross-sections

Cross-sectional geometry of streams is defined by coordinates of lateral distance and ground elevation that locate individual ground points. The cross-section is taken normal to the flow direction along a single, straight line where possible but, in wide floodplains or bends, it may be necessary to use a section along intersecting straight lines; i.e., a “dog-leg” section. It is especially important to make a plot of the cross-section to reveal any inconsistencies or errors (See Figure 3–4).

Cross-sections should be located to be representative of the subreach. Stream locations with major breaks in bed profile, abrupt changes in roughness or shape, control sections such as free overfalls, bends and contractions, or other abrupt changes in channel slope or conveyance will require cross-sections taken at shorter intervals to better model the change in conveyance.

Cross-sections should be subdivided with vertical boundaries where there are abrupt lateral changes in geometry and/or roughness as for overbank flows. The conveyance of each subsection is computed separately to determine the flow distribution and are then added to determine the total flow conveyance (See Figure 3–3). The subsection divisions must be chosen carefully so that the distribution of flow or conveyance is nearly uniform in each subsection (Davidian, 1984).

![Figure 3–3 HEC-RAS Default Cross-section Conveyance Subdivision (HEC-RAS, 2010)]
State Standard SS9-02 *Floodplain Hydraulic Modeling* (ADWR, SS9-02), Section 3.3.1 provides detailed instructions regarding appropriate cross-section location and orientation.

### 3.4.2 Manning’s n Value Selection

See Appendix 3A for typical n-values for natural and constructed channels. The values in the Appendix are adequate for initial estimation and approximation of n-values for some engineered features. As with all n-value determination methods, engineering judgment is required in selection and application.

The following publications should be consulted for detailed information specific to Arizona streams:


Several of the published methods (Phillips and Ingersoll, 1998, notably), require an estimate of hydraulic parameters (i.e. hydraulic radius) to calculate the appropriate Manning’s roughness. In these cases which rely upon empirical calculations, iterative solutions are required to converge on an appropriate roughness value and a Manning’s roughness convergence tolerance of 0.005 units should be used. If this convergence cannot be achieved, the designer should note the location of the roughness and describe possible causes of a non-convergent solution in the drainage report.

Since the design condition is typically a relatively rare event, the roughness of the watercourse during such an event may be drastically different than what is observed in the field. The development of dune bedforms in channels is an example of this phenomenon and should be considered for high-flow events.

**3.4.2.1 Event-Varied Roughness**

During high-flow events, the roughness of vegetated channels may be reduced due to vegetation “flattening.” Alternately, heavily vegetated channels may be prone to debris accumulation which can increase roughness during high-flow events. Shear stress should be inspected at the design discharge to determine if vegetative linings will be compromised due to applied hydraulic stresses. In modeling these conditions, a high roughness condition will produce a more conservative water surface elevation with higher applied shear stress; a low roughness condition will produce a higher velocity condition with lower applied shear stress. Depending upon the
purpose of the model, the most appropriately conservative roughness condition should be modeled (i.e. high roughness for bank height and low roughness for scour).

### 3.4.2.2 Calibration and Verification

When possible, roughness equations should be calibrated with historical high-water marks and/or gaged stream flow data to ensure that they accurately represent local channel conditions. The following parameters, in order of preference, should be used for calibration: Manning’s $n$, slope, discharge, and cross-section. For best results, the calibration discharge should be approximately equal to the design discharge.

### 3.4.3 Single-Section Analysis

The single-section analysis method (slope-area method) is simply a solution of Manning’s equation for the normal depth of flow given the discharge and cross-section properties including geometry, slope, and roughness. It implicitly assumes the existence of steady, uniform flow; however, uniform flow rarely exists in either constructed or natural channels. Nevertheless, the single-section method is often used to design constructed channels for uniform flow and to develop a stage-discharge rating curve in a natural channel for tailwater determination at a culvert or storm drain outlet.

The best approach, especially for natural channels, is to use a computer program such as FHWA Hydraulic Toolbox (FHWA, 2013) to obtain the normal depth.

There may be locations where a stage-discharge relationship has already been measured in a channel. These usually exist at gaging stations on streams monitored by the USGS. Measured stage-discharge curves will generally yield more accurate estimates of water surface elevation and should take precedence over the analytical methods described above.

### 3.4.4 Step-Backwater Analysis

Step-backwater analysis is useful for determining unrestricted water surface profiles where a highway crossing is planned and for analyzing how far upstream the water surface elevations are affected by a planned or existing culvert or bridge. Step-backwater analysis is not purely appropriate for analysis within a pressure-flow condition, but many step-backwater packages contain sub-routines for hydraulics in specialized pressure-flow scenarios.

Because the calculations involved in this analysis are tedious and repetitive, it is recommended that an accepted computer program such as HEC-RAS (HEC-RAS, 2010a) be used. Special analysis techniques (see Section 3.4.7) should be considered for complex situations where a standard one-dimensional step-backwater analysis might not give the desired level of accuracy.

Guidance related to use of the HEC-RAS modeling platform is provided below.

### 3.4.4.1 Model Composition

HEC-RAS models are comprised, at a minimum, of flow data, roughness data, and geometry data.
Flow data utilized within HEC-RAS models should be documented in the provided description and notes fields of the model project files. Appropriate data to list includes the hydrology source, hydrologic method, assumptions, and boundary condition source, type, and value.

Roughness data should be documented based upon the references listed in Section 3.4.2. Documentation should be included with the HEC-RAS model and in the project drainage report and include site photographs with location references. Depending upon the level of detail being modeled, horizontal n-value variation may be necessary within individual cross-sections.

Geometry data may be derived from a variety of sources. The mapping source from which the geometry data is based should be documented in the drainage report and within the provided description/notes fields within HEC-RAS. HEC-RAS allows for multiple geometry files in a single project. If multiple geometries are bundled in a project, descriptions of relevant differences in each geometry should be noted with their respective geometry files (in the provided description/notes field). Care should be taken to document the vertical and horizontal datums of project mapping and document datum adjustments if used (VERTCON, etc.).

Digital project files should be included in the drainage report.

### 3.4.4.2 HEC-RAS Review Guidance

The following suggestions are to be considered in developing and reviewing HEC-RAS water surface profile models.

**Warning Messages and Tabular Output**

HEC-RAS may generate warning messages at each cross-section. Review of warning messages, notes and cautions should be undertaken and any unresolved items should be noted in the drainage report. Graphical and tabular output should be inspected for reasonableness. Specific examples of unreasonable results include unexpected changes in water surface profile, velocity, flow distribution, top width, divided flow and length between cross-sections including the main channel and overbanks.

**Cross-Sections**

Cross-section spacing should be established based upon the channel bed slope and any substantial changes in flow parameters such as width and slope. In general, the cross-section spacing is determined by the need to adequately define energy losses (friction, flow expansion, and flow contraction) between consecutive cross-sections and at selected locations. (See Figure 3–4).
Figure 3–4  Streamlines and Cross-section Locations (ADWR SSA9-02, Figure 3.1)
In general, skewed cross-sections should be avoided by cutting cross-sections perpendicular to the direction of flow. If used, skewed cross-sections should be corrected by using the projected cross-section length. The projected cross-section length is found by projecting the skewed length onto a plan perpendicular to the direction of flow. In some cross-sections, the skew may apply only to a part of the cross-section such as the channel, and not the overbanks. For bridge applications, cross-sections should only be skewed if the bridge is skewed to flow.

The output should be reviewed for vertical extension warnings. HEC-RAS “side-boards” cross-sections such that flow which would extend beyond the end of the cross-section is contained within the section by vertical boundaries at the start and end points of the cross-section. In the event of this error, cross-sections should be extended based upon topographic data or lateral flow elements incorporated to estimate the volume of discharge leaving the section. For cross-sections with spatially defined cross-section cut-lines, the cut-lines should be extended to conform to the extension of the section.

**Energy Slope**

A review of the energy slope should be undertaken to see if rapid changes occur. If the slope increases or decreases rapidly between consecutive cross-sections, decreasing the cross-section spacing by adding cross-sections may be necessary.

**Critical Depth**

The presence of critical depth at an isolated cross-section may indicate an error in cross-section geometry, or in rapid widening of sections between adjacent cross-sections. Rapid changes in energy slope may also generate isolated critical depth cross-sections. Isolated critical depth sections should be inspected and additional cross-sections added or existing cross-sections modified to transition in and out of the critical depth condition.

When a subcritical flow regime is specified within HEC-RAS, several cross-sections with critical depth may be an indication of supercritical flow. To accurately model this type of condition, a mixed-flow regime should be run. If a model is to be submitted to FEMA for floodplain delineation, only subcritical flow regime submittals are permitted.

**Top Width**

Substantial changes in flow top-width between adjacent cross-sections should be inspected. Rapid changes are often associated with transitions between supercritical and subcritical flow. If extreme changes in top-width are found to be appropriate, review of the related expansion and contraction coefficients should be made to verify the use of values appropriate to the situation.

Overtopping of levees and “non-permanent” ineffective flow areas can also cause rapid changes in top width. Top-width changes should also be reviewed with respect to these elements.
Flow Distribution

A large change in the distribution of flow in the channel and overbanks may indicate a need for additional cross-sections.

Indications of divided flow should be checked for consistency with topography. The divided flow should be hydraulically connected to be run in a single model.

Profiles/Plans

Multiple profiles for a given geometry should be checked for reasonableness of results. For example, the definition of ineffective flow areas and roughness values when modeling a relatively-low discharge plan may not be applicable at a higher discharge and generate comparatively unreasonable results.

3.4.5 Bends

One-dimensional hydraulic computations such as single-section normal-depth analysis and step-backwater analysis, do not explicitly consider two-dimensional phenomenon such as superelevation in bends. For one-dimensional models, superelevation of flow in bends must be manually calculated and considered separately from the computer model results. On the outside of bends, the computed superelevation should be added to the one-dimensional freeboard requirement. Generally, calculation of superelevation separately is not necessary when using two-dimensional models.

In step-backwater models, modification of expansion and contraction coefficients is not an acceptable means for accounting for the presence of a bend. Cowan (Cowan, 1956) presents suggestions for adjusting the channel roughness to account for the effects of a bend.

For subcritical flow, the magnitude of superelevation at the edge of the wetted section is given by:

\[ S = \frac{wV^2}{gR_c} \]  

where:  
- \( S \) = superelevation (ft),  
- \( w \) = channel top width at the water surface (ft),  
- \( V \) = mean channel velocity (ft/sec),  
- \( g \) = gravitational acceleration (32.2 ft/s²), and  
- \( R_c \) = radius of curvature at channel centerline (ft).

Equation 3.7 is only valid when \( R_c > 3w \); further a channel is considered straight and superelevation negligible under subcritical flow conditions when \( R_c > 10w \). (See Figure 3–8)

For supercritical flow, superelevation is calculated in the same manner as for subcritical flow, however the minimum allowable radius of curvature is represented by (USACE, 1994):
\[ R_{cmin} = \frac{4wV^2}{gy} \]

where:  
- \( R_{cmin} \) = minimum radius of curvature, at channel center line, for supercritical flow (ft),  
- \( w \) = channel top width at the water surface (ft),  
- \( V \) = mean channel velocity (ft/sec),  
- \( g \) = gravitational acceleration (32.2 ft/s\(^2\)), and  
- \( y \) = depth of flow (ft).

Additionally, under supercritical conditions, waves may develop across the cross-section and the calculated superelevation value should be applied in design as though it occurs across the entire section.

### 3.4.6 Pit/Depression Analysis

Natural or man-made depressions can be situated in-channel or off-channel (in the overbank area). A common cause of hydraulically significant depressions in Arizona is active or dormant sand and gravel extraction operations. When modeling pit/depressions, an appropriate number of cross-sections should be selected to represent the change(s) in pit dimensions. At a minimum, cross-sections are required at the beginning and end of the pit/depression with an additional cross-section located at the widest part of the pit. In plan, the pit perimeter at the cross-sections would form a diamond shape.

When considering pits in hydraulic analysis, conventional, fixed-bed analysis may not be adequate for evaluation. *Effects of In-Stream Mining on Channel Stability* (Li et al, 1989) provides guidelines for estimation of upstream and downstream impacts with respect to the channel bed. Active mining pits should possess a mining plan or permit; the mining plan or permit should be considered during analysis.

### 3.4.7 Special Analysis Techniques

Open channel flow problems sometimes arise that require a more detailed analysis than a single-section analysis or the computation of a water surface profile using the step-backwater method. More detailed analysis techniques include two-dimensional analysis, sediment routing and unsteady flow analysis. Computer programs are available for the analysis techniques discussed in this section.

#### 3.4.7.1 Two-Dimensional Analysis

Two-dimensional hydraulic models may be used to compute flow characteristics and hydraulics for two-dimensional flow conditions. Two-dimensional flow conditions are defined by flow components in multiple directions. Common instances of two-dimensional flow include flow across roadway intersections, channel bends, and channel confluences.
Model Selection Factors

The selection and application of a two-dimensional hydraulic model will depend upon the following factors:

Project Type/Constraints

While two-dimensional flow exists to some degree under many situations, the importance of capturing the detail, extents, and impact of the two-dimensional flow characteristics may not be significant enough to warrant the additional level of analysis represented by two-dimensional analysis.

Flow Characteristics

Complex flow situations or patterns with non-linear structures and other non-structural features, junctions, flow splits, and other scenarios where a single, uniform flow orientation cannot be defined may benefit from two-dimensional hydraulic analysis.

Agency Approval

Due to the specialized nature of the application of two-dimensional modeling and the large amount of modeler experience and physical data and data processing required to build a two-dimensional model, acceptance and approval of a two-dimensional modeling approach by ADOT and, as applicable, other project partners, is important and necessary. Two-dimensional models should only be selected when alternative methods with lower levels of complexity are not appropriate. Approval of model selection by the ADOT Roadway Drainage Section Manager and/or the Bridge Drainage Section Manager, as applicable, is required prior to implementation of a two-dimensional model.

Economics

Projects with high construction costs may benefit from incremental changes in hydraulic design parameters generated by two-dimensional analysis when compared to one-dimensional analysis. However, it should be recognized that design parameters resulting from two-dimensional modeling will not necessarily result in a lower cost design when compared to the parameters derived from one-dimensional analysis.

Model Application

Fundamentally, two-dimensional hydraulic modeling involves computation of hydraulic parameters into two orthogonal components. Multiple modeling engines have been developed; in general, these modeling engines can be separated into two types of models: finite element models and finite difference models.

Finite element models rely upon a mesh of nodes. Properties such as elevation, roughness, inflow, and outflow are defined at the nodal locations and the computational engine performs calculations at nodal locations and calculates flow characteristics at these nodes.
Finite difference models use a fixed, rectangular grid to define the computational domain. The computational engine performs calculations at each grid location and calculates flux along two directions at the grid locations.

Prior to selecting or applying a two-dimensional model for a project, the limitations and assumptions inherent in the model should be understood and considered with respect to the project goals.

**Geometry**

Development of the spatial computational domain will vary depending upon the model type utilized. Finite element models can be sensitive to mesh development and require progressively refined node densities to achieve the desired level of detail near points of interest. Finite difference model grids must be developed to establish sufficient model resolution to capture the necessary level of detail. With both model types, a greater number of grids/nodes correspond with greater model run-times, file sizes, and complexity.

In addition to selection of an appropriate mesh/grid density, the method used to sample element elevations can impact model output and performance. Two-dimensional models typically utilize geometry pre-processors to calculate node/grid elevations. Assumptions used in the calculations of node/grid elevations, coupled with nodal locations can greatly affect model accuracy, particularly in areas which are sensitive to minor variations. Generally, when mesh/grid spacing is increased, error due to selective sampling is also increased. Effort should be made to identify significant topographic features and align mesh/grid elements to represent these features as accurately as possible.

**Discharges and Boundary Conditions**

Two-dimensional models are typically unsteady flow models. Steady flow conditions can be simulated in an unsteady model through use of a protracted hydrograph with a constant peak discharge. If steady flow is to be modeled, the hydrograph peak must be maintained, at a minimum, for the greatest time of concentration present in the model. Greater peak durations may be necessary to achieve a steady-condition throughout the model. The modeling results obtained in this manner may display minor oscillations due to non-linearity in the flow modeling approach. In such a case, results must be interpreted appropriately to reflect a conservative approach.

Similar to one-dimensional models, two-dimensional models rely upon boundary conditions, hydraulic roughness, and inflow and outflow conditions. The computational boundary should be sufficiently offset from areas of interest to avoid boundary effects.

Some models couple hydrologic calculations with hydraulic calculations and do not explicitly require inflows. For these models, application of two-dimensional hydrologic tools is addressed in DDM Volume 2 - Hydrology.
Roughness and Model Calibration

Roughness parameter selection should generally follow the guidelines established for one-dimensional modeling shown in Appendix 3A as well as Section 3.4.2. Roughness should be utilized during model calibration and may vary from that used in one-dimensional modeling. During model calibration, if roughness values appear unreasonable when adjusted to match observed values, care should be taken to eliminate other possible sources of error.

Calibration to known flow events is ideal for two-dimensional model development. Due to potential changes in roughness during flow regimes, a calibrating event of approximately the same magnitude as the design event is preferred. In cases where calibration to a known event is not possible, comparison to one-dimensional model results may be useful diagnostically. While two-dimensional models produce different output than one-dimensional models, some results should be of the same general magnitude. For 1D-2D comparisons, comparison of cross-sectional output is often useful.

Due to diverse computational algorithms, each two-dimensional model utilizes semi-unique conditioning parameters and assumptions. Adjustment and modifications of these parameters can directly influence model run-times, stability, and accuracy. The conditioning parameters and assumptions used in a given model should be documented and justified prior to acceptance of the results.

Model Results

When applying two-dimensional model results in design, care must be taken to select appropriate values from the model output. When appropriate, resolved vector magnitudes and directions should be utilized rather than individual vector components. Similarly, selection of appropriate spatial locations for sampling is important. Judgment may be required in selecting the appropriate sampling location for a given value as output density may be greater than from a one-dimensional model.

Available Models

At the time of the writing of this manual, numerous two-dimensional hydraulic models are available. Publicly available models include:

- RMA2, developed by USACE
- SRH-2D, developed by USBR
- FESWMS, developed by FHWA
- HEC-RAS v5.0 (currently in Beta testing)

Each of these models, except for HEC-RAS v5.0, use finite-element meshes. While the model engines are available publicly, a geometry pre-processor is generally used to develop the model input and a post-processing engine is generally used to view model output. Programs to perform these pre- and post-processing functions are generally proprietary.
Several wholly proprietary models are also commonly used and include:

- RiverFlow2D
- TUFLOW

TUFLOW is a finite-difference model and utilizes finite-difference grids while RiverFlow2D is a finite element model like RMA2, SRH-2D, and FESWMS, described above.

### 3.4.7.2 Sediment Routing

Joint water and sediment routing is used to determine long term scour or aggradation for a stream reach. This analysis is typically required for bridge design and for determining the toe depth for channel bank lining design.

Several hydraulic modeling packages incorporate coupled sediment routing components. These models, often referred to as “moving bed” models, dynamically adjust channel bed elevations during the model simulations based upon sediment transport rates and spatial erosion and deposition characteristics. These models are appropriate for estimating long-term sediment trends, but should not be used for calculation of severe or complicated scour scenarios.

In general, the following elements are required in moving bed models, in addition to hydraulic considerations discussed in Section 3.3.

#### Unsteady Flow Simulation

Total sediment volume is directly related to hydrograph volume. To estimate the volume of sediment transported, the model must route a known hydrograph through the stream reach of interest. Further, multi-year analysis is needed to evaluate long-term performance. For this type of analysis, a series of defined hydrographs must be modeled sequentially to simulate the changes in sedimentation over a maintenance period or design life. Hydrograph series should be developed based upon project hydrology and with a design period. For example, a sediment basin with a desired 2-year maintenance interval may only require a sequence of storms representing 10-years of probable hydrographs. This series could be represented by 4 two-year recurrence interval hydrographs, 1 five-year recurrence interval hydrograph, and 1 ten-year recurrence interval hydrograph.

Some models (HEC-RAS, specifically), utilize quasi-unsteady flow which discretizes a steady-flow hydrograph to facilitate computations. When establishing discretization levels for quasi-unsteady models, designers must balance model accuracy with the complexity of the quasi-unsteady hydrograph.

#### Sediment Boundary Conditions

The inflow sediment characteristics and flow conditions of the area to be modeled must be known or estimated. If no sediment inflow is input, a sediment-lean condition is modeled. In a long-enough system, sediment-lean models will initially scour sediment in the upstream end of the model until an appropriate sediment transport rate is established. If the upstream sediment
inflow is not known, but is believed to be greater than zero, the model can be artificially extended upstream to allow sediment transport to develop and avoid a sediment-lean condition. The length of the extension should be based upon observed behavior within the model.

**Bank and Bed Material**

The sediment gradation of the bank and bed material must be estimated. Transport by size fraction varies and phenomenon such as armoring may develop. Armoring describes the preferential transport of smaller sediment fractions until only larger, non-transportable sediment remains. The non-transportable sediment layer is referred to as an armoring layer.

Care should be taken when evaluating bed material to identify depositional areas. Within areas of deposition, the surficial bed material may not represent the underlying material. Depositional areas are often differentiated by the presence of bars and pronounced differences between bed and bank material.

**Models**

Many modern computer models are capable of performing coupled water and sediment routing. Most of these computer models allow for the selection of a sediment transport function from an array of available functions. Selection of the appropriate sediment transport function for a given stream is one of the most important factors in sediment routing. Care should be taken in the selection of the most appropriate sediment transport function and the selected function and rationale for selection should be documented in the drainage report.

Beyond the sediment transport function, individual computer models may apply differing methods to distribute aggradation/degradation within a given cross-section. Before selecting a sediment-routing model, the cross-sectional distribution method should be reviewed.

Commonly available models include:

- HEC-RAS
- BRISTARS
- HEC-6 (HEC-6T)

**3.4.7.3 Unsteady Flow Analysis**

One-dimensional, unsteady flow can be analyzed with the HEC-RAS (HEC-RAS, 2010a) computer program. Some of the features of HEC-RAS are the network simulation of split flow and combined flow. The effect of storage areas can also be analyzed. This feature is useful when the effects of natural channel and/or overbank floodwater storage areas are sufficient to allow a significant reduction in peak discharge rates approaching a drainage structure or series of structures. HEC-RAS can provide realistic estimates of headwater produced at a series of closely spaced highway drainage structures. HEC-RAS can be used to analyze lateral overflow into storage areas over a gated spillway, weir, levee, through a culvert or a pumped diversion. The user can apply several external and internal boundary conditions, including flow and stage hydrographs, gated and controlled spillways, bridges, culverts and levee systems.
Two-dimensional, unsteady flow can be analyzed with any of the two-dimensional modeling packages described in Section 3.4.7.1.

### 3.5 CHANNEL LINING

The design process for channels typically starts with the determination of a design storm and discharge. The range of channel design discharges shall be selected based upon the class of roadway, consequences of traffic interruption, flood hazard risks, economics, and local site conditions. The design process then proceeds with two general phases: channel capacity and lining/bank protection. Note that natural channels are assessed (modeled), but not explicitly designed.

During channel capacity design, calculations are performed to assess channel capacity based upon principles of open-channel flow and Manning’s equation. Assumptions regarding the channel lining material are necessary to determine appropriate geometric parameters and hydraulic roughness values.

The results of the capacity design are then used to evaluate the suitability of the assumptions related to channel lining material. If the assumed lining is found to be inadequate based upon the calculated hydraulics, a new lining is assumed and the capacity design is iterated until the capacity and lining solutions converge.

This section describes properties and design of various lining types and suggestions for selecting the appropriate lining type.

#### 3.5.1 Types of Channels

##### 3.5.1.1 Ditches

Ditches are small drainage conveyances which are designed secondarily to other transportation features. If lined, ditches will typically not be lined with rigid linings such as concrete or soil-cement.

##### 3.5.1.2 Roadside Channels

Roadside channels collect and convey surface runoff for a larger area than ditches, which typically drain only the area immediately adjacent to the ditch. Roadside channel lining may consist of any of the materials outlined in this section including continuous channel bank lining.

##### 3.5.1.3 Natural Channels

Natural channels may be lined with any of the materials/methods outlined in this section. Environmental constraints may be a factor in the design selected. Additionally, bank protection features such as guide banks are more likely to be used in a natural channel than a roadside channel.
3.5.2 Channel Lining Selection

Selection of an appropriate channel lining depends upon a variety of factors including hydraulics, material availability, highway safety considerations, scale/volume of project, and permitting.

The preferred option, if acceptable, is excavation of the channel in the in-situ, native material without a constructed lining. This is generally the lowest-cost option. Hydraulic analysis of the ditch/channel should be completed to determine if the native material is adequate to resist erosion. If native material is found to be inadequate to resist erosion, grade control structures may be an effective alternative to limit erosion to acceptable levels. If native material with or without grade control is still inadequate, constructed channel lining materials from Table 3–1 are approved for use on ADOT projects.

<table>
<thead>
<tr>
<th>Name</th>
<th>Construction Standard Drawing(s)</th>
<th>ADOT Standard Specifications Section</th>
<th>Primary Design Reference(s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Continuous Concrete Lining</td>
<td>-</td>
<td>-</td>
<td>ADOT, 1989 (Appendix 3D)</td>
</tr>
<tr>
<td>Loose Riprap (Dumped Riprap)</td>
<td>-</td>
<td>913</td>
<td>HEC 15</td>
</tr>
<tr>
<td>Grouted Riprap</td>
<td>-</td>
<td>913</td>
<td>UDFCD, 2008</td>
</tr>
<tr>
<td>Wire-Tied Rock (gabions)</td>
<td>-</td>
<td>913</td>
<td>HEC 23, DG 10</td>
</tr>
<tr>
<td>Rail Bank</td>
<td>C-17.10, C-17.15, C-17.20</td>
<td>913</td>
<td>HEC 23, DG 6</td>
</tr>
<tr>
<td>Articulated Concrete Blocks</td>
<td>-</td>
<td>-</td>
<td>HEC 23, DG 8</td>
</tr>
<tr>
<td>Soil-cement/CSA</td>
<td>-</td>
<td>Supp 9130401 (913SLCMT)</td>
<td>HEC 23, DG 7</td>
</tr>
</tbody>
</table>

3.5.2.1 Native Material

To determine if the native material requires erosion protection, a comparison between the tractive shear of the design flow and the resistive shear of the in-situ material is necessary.

Based upon permissible tractive shear stress, a lining is stable when the permissible or resistive shear of the material is greater than the applied, or tractive shear stress multiplied by a safety factor. HEC-15, *Design of Roadside Channels with Flexible Linings* (HEC 15, 2005) presents a methodology for assessing and adjusting tractive shear stress to a number of situations including bed stresses, bank stresses, and stresses in bends.
The permissible velocity design approach is similar to the allowable shear approach. Reference data for permissible velocity is readily available for a given lining.

Table 3–2 shows recommended values for permissible shear stress for a variety of materials. Detailed permissible shear calculation methods for a wider range of materials and conditions, as well as more specific guidance, may be found in Chapter 4 of HEC 15.

<table>
<thead>
<tr>
<th>Lining Category</th>
<th>Lining Type</th>
<th>( \tau_p ) lb/ft²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bare Soil¹ Cohesive (PI = 10)</td>
<td>Clayey Sands</td>
<td>0.037-0.095</td>
</tr>
<tr>
<td></td>
<td>Inorganic Silts</td>
<td>0.027-0.11</td>
</tr>
<tr>
<td></td>
<td>Silty Sands</td>
<td>0.024-0.072</td>
</tr>
<tr>
<td>Bare Soil¹ Cohesive (PI &gt;= 20)</td>
<td>Clayey Sands</td>
<td>0.094</td>
</tr>
<tr>
<td></td>
<td>Inorganic Silts</td>
<td>0.083</td>
</tr>
<tr>
<td></td>
<td>Silty Sands</td>
<td>0.072</td>
</tr>
<tr>
<td></td>
<td>Inorganic Clays</td>
<td>0.14</td>
</tr>
<tr>
<td>Bare Soil² Non-cohesive (PI &lt; 10)</td>
<td>Finer than Coarse Sand D₇₅ &lt; 0.05 in.</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td>Fine Gravel D₇₅ = 0.3 in.</td>
<td>0.12</td>
</tr>
<tr>
<td></td>
<td>Gravel D₇₅ = 0.6 in.</td>
<td>0.24</td>
</tr>
<tr>
<td>Gravel Mulch³</td>
<td>Coarse Gravel D₅₀ = 1.0 in.</td>
<td>0.4</td>
</tr>
<tr>
<td></td>
<td>Very Coarse Gravel D₅₀ = 2.0 in.</td>
<td>0.8</td>
</tr>
</tbody>
</table>

¹Based on HEC 15, Equation 4.6 assuming a soil void ratio of 0.5
²Based on HEC 15, Equation 4.5
³Based on HEC 15, Equation 6.7 with Shield’s Parameter equal to 0.047

3.5.2.2 Grade Control Structures

If native material cannot adequately resist erosion based upon an initial design slope, use of grade control or drop structures may be effective to reduce the slope to a point where other linings are not required. Grade control structures are typically used in degrading streams to limit the lowering of the channel bottom. The concept of equilibrium slope is used to determine the ultimate channel slope that would be expected to be established over time by repeated flow events. Grade control structures should only be used when the reduced design slope resulting from their application produces a tractive shear stress sufficiently less than the resistive shear of
the soil (or channel/bank lining) to preclude erosion. Equilibrium slope can be calculated via methods presented in *Computing Degradation and Local Scour* (USBR, 1984).

Grade control structures may be used in isolated locations or in series. Isolated grade control structures are an effective means of limiting the upstream propagation of a head-cut. When used in series, grade control structures can be used to establish a design channel slope without continuous armoring. Grade control structures are typically cost-prohibitive for all but long channel reaches. For short reaches, channel lining methods are generally less-costly when native soil is not adequate.

Properly designed grade control structures will account for long-term degradation of the channel bed immediately downstream of the structure as well as local scour generated at the base of the sill due to plunging flow during discharge events. Grade control structures must also be suitably anchored into both banks as flanking could result in failure of the structure. Design of grade control structures is presented in *Hydraulic Design of Energy Dissipators for Culverts and Channels* (HEC 14, 2006).

The height of grade control structure is measured from the ultimate bed elevation immediately downstream of the structure to the top of the structure sill. The height and profile of grade control structures are design variables which may be dictated by scour considerations, or non-hydraulic considerations such as maintenance access and safety considerations. If more height is required than can be accomplished with a single grade control structure, multiple structures in series can be used.

To determine the appropriate spacing for grade control structures in series, the following relationship may be used:

\[
L_r = \frac{h}{S_I - S_D}
\]

3.9

where:  
\(L_r\) = spacing between grade control structures (ft),  
\(h\) = grade control structure height measured from the ultimate bed elevation immediately downstream of the structure to the structure sill (ft),  
\(S_I\) = initial channel bed slope (ft/ft), and  
\(S_D\) = design channel bed slope (ft/ft).

For economic and technical reasons, grade-control structures should be spaced no closer than twelve times the local scour depth generated at the toe of the structure. For sloping sill structures, the spacing should be no closer than twelve times the local scour depth as calculated per *Scour at Sill Structures* (Laursen and Flick, 1983).

One-dimensional and two-dimensional hydraulic models should not be relied upon to model the complex hydraulic conditions immediately downstream of grade-control structures. The models may provide a general depiction of the complex hydraulic conditions downstream, but are not capable of providing detailed analysis of the subsequent turbulence and 3-dimensional flow characteristics generated below the drop.
Maintenance experience has shown a tendency for erosion to develop at the edges of grade-control structures. Erosion protection at these locations should be considered in the design.

### 3.5.3 Channel Lining Design

Channel linings can be placed continuously on both bed and banks or can be placed solely on the channel banks. Continuous channel linings are typically limited to constructed channels. Bank lining with an unlined bed can be accomplished with several alternative materials as shown in Table 3–1. Linings can be categorized as rigid or flexible linings each with different advantages and disadvantages and their own particular design requirements. This section describes important considerations in design of channels, the longitudinal and vertical extent of required lining protection which is applicable for all lining types, and guidance for filter design and channel transitions.

#### 3.5.3.1 Design Considerations

The following considerations should be addressed during channel lining design:

- Safety of the general public is an important consideration in the selection of cross-sectional geometry of constructed channels and in the selection of an appropriate erosion prevention method/material.
- Stability of the channel is a goal for all constructed channels.
- Environmental impacts resulting from channel modifications, including disturbance of habitat, wetlands, and stream bank stability shall be assessed.
- The design of constructed channels shall consider the frequency and type of maintenance expected and provide for access by maintenance equipment.
- Changes in water surface shall not significantly increase flood damage to property.
- Changes in velocity should not significantly alter the channel behavior nor significantly increase damage to adjacent property.
- Erosion of the channel banks should not occur during the design event.

Based upon experience and input from ADOT maintenance staff, a successful channel design over its useful life will account for the following concepts:

#### Sedimentation

Sediment accumulation within channels is a common maintenance problem. Channels designed for large magnitude flows (such as the 100-year discharge) are often not effective at conveying sediment from more frequent flows due to inadequate sediment transport capacity. The build-up of sediment from more frequent flows creates a need for ongoing channel maintenance procedures. The sedimentation issue is compounded by the growth of vegetation in sediment accumulations, which further decreases sediment transport capacity for subsequent events, and causes more sediment accumulation. Incorporating features to allow conveyance of frequent events with as little top width as possible, such as pilot channels or pronounced cross-slopes in the channel bottom, will help reduce sediment accumulation during small magnitude events.
Maintenance Access

The channel should be easily accessed by maintenance personnel. Specifically, ongoing removal of sediment and vegetation should be considered when designing the channel. Maintenance access ramps should be located in such a way as to allow a haul truck and loading equipment to work in the channel simultaneously.

Inflow Points

Channel inflow points (See Figure 3–5) should be considered and designed appropriately. Where flow enters project right-of-way as sheet flow, site grading should collect the flow at specific points where it is then conveyed via pipes, hardened spillways or ditches into the main channel. This concept is especially important for channels with hardened banks as transverse flows at the top of the hardened bank can cause a build-up of pore water pressure which can contribute to failure of the bank lining. In non-hardened banks, failure to control incoming flows may create localized erosion as flow concentrates.

If maintenance roads are located parallel to the channel and right-of-way is available, a general design recommendation is to grade the maintenance road with a uniform slope away from the channel with a collector ditch located to catch drainage from the maintenance road and any other flows that might enter the ditch. Periodic down-drains across the maintenance road should transition flow from the ditch into the channel.

![Figure 3–5 Channel Maintenance Road and Interceptor Swale Grading](image-url)
SWPPP Features

While Stormwater Pollution Prevention Plans (SWPPPs) are generally developed after the channel design is completed, post-construction use of SWPPP elements, such as rock check dams, may impact the channel design. Specific impacts include loss of freeboard, erosive flanking, and development of local scour around the SWPPP-dictated features. If SWPPP features are required to remain in-place following construction, design consideration of the consequences of the features is warranted.

Single Bank Armoring

In general, single bank armoring should be avoided. Protecting only one bank can impact the sediment supply of the channel and cause flow to draw additional sediment from the bed or opposite bank as well as encourage flow path relocation to the un-armored bank. If single bank armoring cannot be avoided, the potential for channel relocation should be evaluated and documented.

Instances where only one bank is within project right-of-way are a common condition for single-bank armoring. In these cases, potential impacts beyond the project right-of-way must be considered.

3.5.3.2 Longitudinal Extent

Straight Section

The longitudinal extent of protection required for a particular bank protection design is highly dependent on local site conditions. In general, the revetment should be continuous for a distance greater than the length impacted by channel flow forces severe enough to cause dislodging and/or transport of bank material. Although this is a vague criterion, it demands serious consideration. Review of existing bank protection sites has revealed that a common error in stream bank protection is to provide protection too far upstream and not far enough downstream.

Bend Section

One criterion for establishing the longitudinal limits of protection required is illustrated in Figure 3–6. As illustrated, the minimum distances recommended for bank protection are an upstream distance of 1.0 channel width and a downstream distance of 1.5 channel widths from corresponding reference lines. All reference lines pass through tangents to the bend at the bend entrance or exit. This criterion is based on an analysis of flow conditions in symmetric channel bends under ideal laboratory conditions.

Real-world conditions are rarely as simplistic. In actuality, many site-specific factors have a bearing on the actual length of bank that should be protected. A designer will find the above criteria difficult to apply on mildly curving bends or on channels having irregular, non-symmetric bends. Also, other channel controls (e.g., bridge abutments) might already be producing a
stabilizing effect on the bend so that only a part of the channel bend needs to be stabilized. The magnitude or nature of the design flow event might cause erosion problems only in a very localized portion of the bend, requiring that only a short channel length be stabilized; therefore, the above criteria should be used only as a starting point. Additional analysis of site-specific factors is necessary to define the actual extent of protection required.

Field reconnaissance is a useful tool for the evaluation of the longitudinal extent of protection required, particularly if the channel is actively eroding. In straight channel reaches, scars on the channel bank may be useful to help identify the limits required for channel bank protection. In this case, it is recommended that upstream and downstream limits of the protection design be extended a minimum of one channel width beyond the observed erosion limits.

![Figure 3–6 Longitudinal Extent of Revetment Protection (HEC 11, 1989)](image)

In curved channel reaches, the scars on the channel bank can also be used to establish the upstream limit of erosion. Here, a minimum of one channel width should be added to the observed upstream limit to define the limit of protection. The downstream limit of protection required in curved channel reaches is not as easy to define. Because the natural progression of bank erosion is in the downstream direction, the present visual limit of erosion might not define the ultimate downstream limit. Additional analysis based on consideration of flow patterns in the channel bend may be required.
3.5.3.3 Vertical Extent

The vertical extent of protection required of a revetment includes design height and foundation or toe depth.

**Design Height**

The design height of a revetment should be equal to the design high-water elevation plus freeboard. Freeboard is provided to ensure that the desired degree of protection will not be compromised by such factors as:

- Superelevation in channel bends
- Hydraulic jumps
- Flow irregularities due to piers, transitions and flow junctions

In addition, erratic phenomena (e.g., unforeseen embankment settlement, the accumulation of silt, trash and debris in the channel, aquatic or other growth in the channels) should be considered when setting freeboard heights.

![Figure 3–7 Scour/Degradation](image)

**Toe Depth and Scour**

Undermining the revetment toe is one of the primary mechanisms of revetment failure. In the design of bank protection, estimates of the depth of scour are needed so that the protective layer
is placed sufficiently low in the streambed to prevent undermining. The ultimate depth of scour must consider channel degradation and natural scour and fill processes.

The term “scour” describes the lowering of a channel bed in response to flow. For revetment design, a design scour depth must be calculated.

The design scour depth is defined as the sum of individual scour components as shown below (modified from FCDMC, 2013).

\[
d_s = SF_{LT}d_{LT} + SF\left(d_g + d_b + d_{bf} + d_L\right)
\]

where:
- \(d_s\) = total scour depth (ft),
- \(SF_{LT}\) = safety factor for long-term scour,
- \(d_{LT}\) = long-term scour depth (ft),
- \(SF\) = safety factor for event-based scour, 1.3 minimum,
- \(d_g\) = general scour depth (ft),
- \(d_b\) = bend scour depth (ft),
- \(d_{bf}\) = bed-form scour depth (ft), and
- \(d_L\) = local scour depth (ft).

The minimum design scour depth for natural channels shall be the greater of the calculated value per Equation 3.10 or 5 ft. In lined channels with a lining resistant to scour at the design discharge, scour can be assumed to be zero only if long-term scour/degradation (defined below) can be shown to be zero. In application, the depth of scour, \(d_s\), should be measured from the lowest elevation in the cross-section. It should be assumed that the low point in the cross-section may eventually move adjacent to the protection (even if this is not the case in the current condition).

**Long-Term Scour**

Unlike other scour components, long-term scour is not associated with a specific flood event. Long-term scour, or channel degradation, describes the lowering of a channel bed over a long period of time and many flow events. The magnitude of long-term scour is often influenced by reach and system-wide variables. The period which long-term scour is assumed to occur should be equal to the design life of the facility at a minimum. For cases where use of this period is cost prohibitive, development of a maintenance plan and monitoring plan for the facility may defer some of the initial costs.

ADWR has published guidelines for estimating long-term scour for watercourses in Arizona in State Standard 5-96, Guideline 2 – Channel Degradation Estimation for Alluvial Channels in Arizona (ADWR, SS5-96). The primary method for determining long-term scour trends is historical observation. Bridge inspection reports can provide an excellent record of long-term bed elevation trends for watercourses near bridge locations. Historical conditions should be compared with current conditions to assess the applicability of the historic record. The method for estimating long-term scour should be documented in the drainage report.
Suitable grade control structures for limiting long-term scour are constructed or natural geologic features which prevent or severely limit the degradation of a channel. Channel confluences, or the current bed elevation of a receiving watercourse, does not represent the limit of long-term scour in a tributary.

Selection of an appropriate safety factor for long-term scour is somewhat different than for other scour components. Typical long-term safety factor values should range from 1.5 to 1 depending upon the available data and methods used to estimate the long-term scour depth. Decisions as to whether the safety factor should be reduced should be based upon the methods used to determine the long-term scour depth, scour limiting mechanisms/features, and consequences of underestimation.

**General Scour**

General scour describes the deepening of a channel associated with passage of a specific discharge. Calculation of general scour should follow methodology presented in *Computing Degradation and Local Scour* (USBR, 1984).

For clear-water conditions, the Blench equation applies.

\[
d_g = \frac{Zq_f^{2/3}}{F_{b0}^{1/3}}
\]

where:
- \(d_g\) = general scour depth (ft),
- \(Z\) = channel curvature coefficient
  - = 0.6 for straight channels,
- \(q_f\) = unit discharge for design discharge (cfs/ft), and
- \(F_{b0}\) = Blench’s zero bed factor.

The Flood Control District of Maricopa County (FCDMC, 2013) has developed the following relationships to define Blench’s zero bed factor, based upon figures from the original study:

\[
F_{b0} = \begin{cases} 0.5672 \ln(D_{50}) + 5.0302 & \text{if } D_{50} \leq 0.0411 \text{ ft} \\ 1.3698 \ln(D_{50}) + 7.589 & \text{if } D_{50} > 0.0411 \text{ ft} \end{cases}
\]

where:
- \(D_{50}\) = median diameter of bed material (ft).
For live-bed conditions where upstream sediment is being supplied, the Lacey equation applies (USBR, 1984):

\[
d_{g} = Z \left( 0.47 \left[ \frac{Q^{1/3}}{f} \right] \right)
\]

where:
- \(d_{g}\) = general scour depth (ft),
- \(Z\) = channel curvature coefficient
- = 0.25 for straight channels,
- \(Q\) = design discharge (cfs)
- \(f\) = Lacey’s silt factor = \(1.76(D_{m})^{1/2}\), and
- \(D_{m}\) = mean grain size which may be approximated by \(D_{50}\) (mm).

Both equations, as presented in Computing Degradation and Local Scour (USBR, 1984), include a factor to account for channel curvature. The definitions of these factors are imprecisely stated in the document and only \(Z\)-values for straight channel reaches should be used. Scour in bends should be computed as described below.

### Bend Scour

Bend scour describes the deepening of a channel along the outer edge of a bend section.

Bend scour should be calculated as presented in the Standards Manual for Drainage Design and Floodplain Management in Tucson, Arizona (City of Tucson, 1998). This methodology presents both the vertical and longitudinal extents of scour to be expected. While the method presented is intended for use in sand-bed channels, it may conservatively be applied to channels with coarser bed material.

The magnitude of bend scour is expressed as:

\[
d_{b} = \frac{0.0685Y_{\text{max}}V_{\text{m}}^{0.8}}{Y_{h}^{0.4}S_{e}^{0.3}} \left[ 2.1 \left( \sin^{2} \left( \frac{\alpha}{2} \right) / \cos(\alpha) \right)^{0.2} - 1 \right]
\]

where:
- \(d_{b}\) = bend scour depth (ft)
- = 0 when \(r_{c}/T \geq 10\), or \(\alpha < 17.8^\circ\)
- = computed value when \(0.5 < r_{c}/T < 10\) or \(17.8^\circ < \alpha < 60^\circ\)
- = computed value at \(\alpha = 60^\circ\) when \(r_{c}/T \leq 0.5\) or \(\alpha > 60^\circ\),
- \(Y_{\text{max}}\) = maximum depth of flow immediately upstream of bend (ft),
- \(Y_{h}\) = hydraulic depth of flow immediately upstream of bend (ft),
- \(V_{\text{m}}\) = average velocity of flow immediately upstream of bend (ft/s),
- \(S_{e}\) = energy slope immediately upstream of bend (ft/ft), and
- \(\alpha\) = angle formed by projection of channel centerline from point of curvature to a point which meets tangent line to the outer bank of the channel, (degrees).
For a simple circular curve, the following relationship exists between the bend angle, $\alpha$, the ratio of centerline radius, $r_c$, and the channel top width, $T$.

\[
\frac{r_c}{T} = \frac{\cos \alpha}{4 \sin^2(\alpha/2)}
\]

where:

- $r_c$ = radius of curvature along centerline of channel (ft), and
- $T$ = channel top width (ft)

Figure 3–8  Radius of Curvature for Bend Scour

If the bend deviates significantly from a simple circular curve, the curve should be subdivided into a series of circular curves. The bend scour should be calculated for each segment of the curve based on the angle applicable to that segment. As the interaction of bend scour with other occurring channel forming activities is not known, the bend scour shall be considered to act independent of the other activities and shall be added to any other considered changes in the channel shape.
Bend scour shall be considered to have a lateral extent downstream of the channel PT a distance x estimated as:

\[ x = \frac{0.6Y^{1.17}}{n} \]  

where:
- \( x \) = distance from downstream end of channel curvature (point of tangency, PT), to the downstream point at which secondary currents have dissipated (ft),
- \( n \) = Manning’s roughness coefficient, and
- \( Y \) = maximum depth of flow, exclusive of scour (ft).

**Bed Form Scour**

Bed form scour describes the deepening of the channel bed associated with the development of dunes or anti-dunes in the channel bottom.

During lower-regime flows, when the channel Froude number is less than 0.7, dunes form in the channel bottom. The bed form scour associated with dunes may be expressed by (FCDMC, 2013):

\[ d_{bf} = 0.5CY_h \]  

where:
- \( d_{bf} \) = bed form scour depth (ft),
- \( C \) = dune height coefficient, 0.15 < C < 0.3, and
- \( Y_h \) = hydraulic depth of flow (ft).

During upper regime flows, when the Froude number is greater than 1, anti-dunes form in the channel bottom. The bed form scour associated with anti-dunes may be expressed by (FCDMC, 2013):

\[ d_{bf} = 0.0135V_m^2 \]  

where:
- \( d_{bf} \) = bed form scour depth (ft), and
- \( V_m \) = average channel velocity (ft/s).

When the channel Froude number is greater than or equal to 0.7 and less than or equal to 1, the greater of Equations 3.17 and 3.18 should be used.

**Local Scour**

Local scour describes localized, exacerbated scour specific to a location or feature. Most local scour phenomenon will require scour formulation specific to the application such as scour at vertical walls or bends which exceed the tolerances of Equation 3.14. Specific elements of local scour associated with culverts and bridges are described in other chapters.
3.5.3.4 Filter Design

In most channel lining applications, a filter is placed between the channel lining material and the subgrade soils. These filters provide two important functions:

1. Prevent leaching of subgrade materials through the lining
2. Assist in draining the subgrade and preventing build-up of pore water pressure.

ADOT does not use granular filters. Geotextile filters or geotextile/geocomposite strips are used to reduce sediment leaching and convey groundwater flows to downdrains, respectively. Bank protection fabric requirements are addressed in Section 913-2.05 of the ADOT Standard Specifications.


3.5.3.5 Channel Transitions

Channel transitions are required when a lined channel section passes through a structure such as a bridge or culvert or when the channel cross-section is changed. Energy losses, wave disturbances, and velocity changes must be considered at channel transitions. The analysis and design of transitions should be performed using methods which incorporate energy and momentum principles.

Vertical wall transitions as shown in Figure 3–9 are desirable. This type of transition simplifies construction, resists earth pressure and eliminates potential compressive problems in a warped slab. While this type of transition does not have hydraulic properties as favorable as the warped method, the above characteristics make the vertical wall transition preferable.

Table 3–3 presents allowable transition rates at channel contraction or expansion sections. With the transition rates shown in Table 3–3, contraction and expansion head loss coefficients of 0.2 and 0.4, respectively, should be used for the vertical wall transition hydraulic modeling.

<table>
<thead>
<tr>
<th>Table 3–3 Channel Expansion and Contraction Transition Rates</th>
</tr>
</thead>
<tbody>
<tr>
<td>Velocity (ft/s)</td>
</tr>
<tr>
<td>---------------------</td>
</tr>
<tr>
<td>Less than 10</td>
</tr>
<tr>
<td>10 to 15</td>
</tr>
<tr>
<td>15 to 30</td>
</tr>
</tbody>
</table>
Figure 3–9 Open Channel Vertical Wall Transition
3.5.4 Continuous Concrete Lining

Concrete channel lining is generally used only for continuous lining of a channel. While bank-only lining is possible, the rigid nature of the revetment and cost to construct toe protection generally results in a continuous lined surface between banks. 

3.5.4.1 Design Guidelines

Guidelines for design of concrete channel lining is presented in *Urban Highways Channel Lining Design Guidelines*, (ADOT, 1989) which can be found in Appendix 3D. The guidelines were developed for use on urban freeway projects. Concrete channel lining is a rigid, monolithic, means of bank protection. When complete, it forms a continuous surface extending from bank to bank including the channel bed. Due to the rigid nature of the lining, design details to manage development of pore water pressure behind the lining is an important consideration. An example of failure of a lining due to hydrostatic uplift is shown on Figure 3–10.

![Concrete Channel Lining Failure Near Kingman, AZ Due to Pore Water Pressure Build-Up](image-url)
Urban Highways Channel Lining Design Guidelines provides guidance for concrete reinforcement, mix design, lining thickness, allowable side slopes, construction joints, cutoff wall placement, transitions, access ramp placement, subgrade treatment and pressure relief.

An additional consideration in concrete channel lining design is the surface texture. Depending on the smoothness required for hydraulics, a float or sand finish may be specified or, if roughness is desired, plans may call for surface roughening by raking the surface after the initial set. In design, consideration should be made for “tining” which is the practice of constructing grooves in the pavement surface to discourage recreational use (skateboards, etc.). Tining is typically constructed with grooves perpendicular to the direction of flow which can greatly increase channel roughness. If a smoother surface is desired or required and recreational-use deterrents are also desired, tining should be specified in the longitudinal direction, parallel to flow.

**3.5.4.2 Typical Cross-Section**

A typical concrete channel lining cross-section is illustrated on Figure 3–11. Typical concrete channel lining consists of the bank lining, a toe section, a head section, cutoff or stub walls, weep holes and a filter layer.

**Edge Treatment**

To minimize accumulation of water behind the channel lining a cutoff wall is constructed at the top of channel. Grading along the channel edges should prevent off-site inflows from spilling over the top of channel edge. Side inflows should be collected and conveyed within a swale on the upstream side of the maintenance road opposite from the channel and directed into the channel at planned locations with a protected apron and spillway. The maintenance road is typically constructed with a cross-slope away from the channel into the swale. The runoff in the swale is then directed across the maintenance road and into the channel (See Figure 3–5). A concrete dip crossing can be provided for the maintenance road where the runoff crosses the road.

**Bottom Cross-Slope**

The channel bottom should be wide enough for maintenance vehicle access and should be sloped to one side to aid in low flow hydraulics and sediment carrying capacity.
3.5.5 Loose Riprap

Loose riprap is also referred to as “dumped” riprap and is a specially graded rock mix of varying size which is placed along banks to prevent erosion. Loose riprap is a flexible lining which can be placed in a stationary configuration along a bank as well as a mobile configuration in a launch trench at the toe of an embankment.

3.5.5.1 Design Guidelines

Two procedures are provided for design of loose riprap; Design of Roadside Channels with Flexible Linings (HEC 15, 2005) is applicable for small applications such as roadside channels and swales with flow depths up to five feet and riprap $D_{50}$ sizes up to 22-inches. Bridge Scour and Stream Instability Countermeasures; Experience, Selection, and Design Guidance (HEC 23, 2009) is recommended for larger applications beyond the limitations of HEC 15. The procedure recommended in Design Guideline 4 of HEC 23 is adopted from the U.S. Army Corps of Engineers Engineering Manual EM1110-2-1601, Hydraulic Design of Flood Control Channels (USACE, 1994).

3.5.5.2 Manning’s Roughness

Manning’s roughness coefficient for loose riprap can be determined using the Blodgett equation presented in Design of Roadside Channels with Flexible Linings (HEC 15, 2005). Blodgett (equation 6.1 in HEC 15) estimates Manning’s $n$ as a function of both flow depth and relative flow depth. The relative flow depth is the ratio of flow depth ($d_a$) to riprap $D_{50}$ size. Since the $n$-value is a function of flow depth, $n$-values should be considered for the range of expected discharges up to the design discharge or the 100-year discharge. The range of applicability for this equation is for values of $d_a/D_{50}$ between 1.5 and 185 which should cover the full range of depths encountered under normal hydraulic conditions.
In cases where the Blodgett equation is not applicable, such as in steep channels with low flow depths, HEC 15 recommends the use of the Bathurst equation (equation 6.2 in HEC 15) which is applicable for values of $d_s/D_{50}$ between 0.3 and 8.

Manning’s $n$ values can be estimated using these equations for both small and large channels as a basis for design of loose riprap channel lining.

### 3.5.5.3 Stone Size and Gradation

#### Small Channels (HEC 15, 2005)

The loose riprap design procedures presented in Chapter 6 of HEC 15 are applicable for small uniform prismatic channels with depths up to 5 feet and riprap $D_{50}$ sizes from 5 to 22-inches. The loose riprap design procedure in HEC 15 is based on a permissible shear stress approach to determine the stone size required to resist the applied shear stress. The permissible shear stress should be computed using a specific unit weight of 150 lb/ft$^3$ as allowed in the Standard Specifications unless heavier stone is available. A factor of safety between 1.0 and 1.5 should be applied to the permissible shear stress as computed in accordance with HEC 15.

**Gradation**

The gradation of the riprap should be included on the project plans and in the project specifications. The guidelines in Table 3–4 should be used in developing a project specific gradation curve.

<table>
<thead>
<tr>
<th>Stone Size Range (ft)</th>
<th>Stone Weight Range (lb)</th>
<th>Percent of Gradation Smaller Than</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5$D_{50}$ to 1.7$D_{50}$</td>
<td>3.0$W_{50}$ to 5.0$W_{50}$</td>
<td>100</td>
</tr>
<tr>
<td>1.2$D_{50}$ to 1.4$D_{50}$</td>
<td>2.0$W_{50}$ to 2.75$W_{50}$</td>
<td>85</td>
</tr>
<tr>
<td>1.0$D_{50}$ to 1.15$D_{50}$</td>
<td>1.0$W_{50}$ to 1.5$W_{50}$</td>
<td>50</td>
</tr>
<tr>
<td>0.4$D_{50}$ to 0.6$D_{50}$</td>
<td>0.1$W_{50}$ to 0.2$W_{50}$</td>
<td>15</td>
</tr>
</tbody>
</table>

#### Large Channels (HEC 23, 2009)

The loose riprap design procedures presented in Design Guideline 4 of HEC 23 are applicable for channels that are too large for use of the HEC 15 methods. The loose riprap design procedure in HEC 23 uses both velocity and depth as its primary design parameters and is based on procedures from the US Army Corps of Engineers *Hydraulic Design of Flood Control Channels* (USACE, 1994). The riprap sizing equation (equation 4.1 in HEC 23, DG4) can be used with uniform or gradually varying flow. Coefficients are included to account for the desired factor of safety, specific gravity of the riprap stone, bank slope, and bendway character. A specific unit weight of 150 lb/ft$^3$ should
be used in the equation as allowed in the Standard Specifications unless heavier stone is available.

**Gradation**

Table 3–5, reproduced from HEC 23, provides recommended gradations for ten standard classes of riprap based on the median particle diameter \(D_{50}\) as determined by the dimension of the intermediate ("B") axis. The gradation criteria are based on a nominal or "target" \(D_{50}\) and a uniformity ratio \(D_{85}/D_{15}\) that results in riprap that is well graded. The target uniformity ratio \(D_{85}/D_{15}\) is 2.0 and the allowable range is from 1.5 to 2.5.

### Table 3–5 Minimum and Maximum Allowable Particle Size in Inches (HEC 23, 2009)

<table>
<thead>
<tr>
<th>Nominal Riprap Class by Median Particle Diameter</th>
<th>(D_{15})</th>
<th>(D_{50})</th>
<th>(D_{85})</th>
<th>(D_{100})</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class</td>
<td>Size</td>
<td>Min</td>
<td>Max</td>
<td>Min</td>
</tr>
<tr>
<td>I</td>
<td>6 in</td>
<td>3.7</td>
<td>5.2</td>
<td>5.7</td>
</tr>
<tr>
<td>II</td>
<td>9 in</td>
<td>5.5</td>
<td>7.8</td>
<td>8.5</td>
</tr>
<tr>
<td>III</td>
<td>12 in</td>
<td>7.3</td>
<td>10.5</td>
<td>11.5</td>
</tr>
<tr>
<td>IV</td>
<td>15 in</td>
<td>9.2</td>
<td>13.0</td>
<td>14.5</td>
</tr>
<tr>
<td>V</td>
<td>18 in</td>
<td>11.0</td>
<td>15.5</td>
<td>17.0</td>
</tr>
<tr>
<td>VI</td>
<td>21 in</td>
<td>13.0</td>
<td>18.5</td>
<td>20.0</td>
</tr>
<tr>
<td>VII</td>
<td>24 in</td>
<td>14.5</td>
<td>21.0</td>
<td>23.0</td>
</tr>
<tr>
<td>VIII</td>
<td>30 in</td>
<td>18.5</td>
<td>26.0</td>
<td>28.5</td>
</tr>
<tr>
<td>IX</td>
<td>36 in</td>
<td>22.0</td>
<td>31.5</td>
<td>34.0</td>
</tr>
<tr>
<td>X</td>
<td>42 in</td>
<td>25.5</td>
<td>36.5</td>
<td>40.0</td>
</tr>
</tbody>
</table>

Note: Particle size \(D\) corresponds to the intermediate ("B") axis of the particle.

### 3.5.5.4 Layer Thickness

All stones should be contained reasonably well within the riprap layer thickness to provide maximum resistance against erosion. Oversize stones, even in isolated spots, may cause riprap failure by precluding mutual support between individual stones, providing large voids that expose filter and bedding materials, and creating excessive local turbulence that removes smaller stones. The following criteria apply to the riprap layer thickness:

1. It should not be less than the spherical diameter of the \(D_{100}\) stone or less than 2.0 times the spherical diameter of the \(D_{50}\) stone, whichever results in the greater thickness.
2. It should not be less than 12 in. for practical placement.

3. The thickness determined by either number one or number two should be increased by 50 percent where the riprap is placed underwater to provide for uncertainties associated with this type of placement.

An increase in thickness of 6 to 12-inches, accompanied by an appropriate increase in stone size, should be provided where riprap revetment will be subject to attack by floating debris.

3.5.5.5 Filter Requirements

Geotextile fabric is required beneath all loose riprap lining. Section 913-2.05 “Bank Protection Fabric” in the Standard Specifications indicates the standard requirements for high survivability, nonwoven filter fabric.

3.5.5.6 Edge Treatment

The edges of riprap revetments (flanks, toe and head) require special treatment to prevent undermining. The flanks of the revetment should be designed as illustrated in Figure 3-12. The upstream and downstream flank is illustrated in Section A-A of this figure.

Undermining of the revetment toe is one of the primary mechanisms of riprap failure. The toe of the riprap should be designed as illustrated in Section B-B from Figure 3-12. The toe material should be placed in a toe trench along the entire length of the riprap blanket. The bottom elevation of riprap within the toe trench shall be located below the channel thalweg.

The size of the toe trench or the alternative stone toe is controlled by the anticipated depth of scour along the revetment. As scour occurs the stone in the toe will launch into the eroded area as illustrated on Figure 3-13. Observation of the performance of these types of rock toe designs indicates that the riprap will launch to a final slope of approximately 2H:1V.

The volume of rock required for the toe must be equal to or exceed 1 ½ times the volume of rock required to extend the riprap blanket (at its design thickness and on a slope of 2H:1V) to the anticipated depth of scour. Dimensions should be based on the required volume using the thickness and depth determined by the scour evaluation. The alternative location can be used where the amount of rock required would not constrain the channel. Establishing a design scour depth is covered in Section 3.5.3.3.
Figure 3–12  Typical Riprap Installation: Plan and Flank Details
3.5.5.7 Design Procedure

Loose riprap design procedures outlined in the following sections are comprised of three primary sections; preliminary data analysis, rock sizing, and revetment detail design. The individual steps in the procedure are numbered consecutively throughout each of the sections. Note: The rock-sizing procedures described in the following steps are designed to prevent riprap failure from particle erosion.

**Step 1** Compile all necessary field data including channel cross-section surveys, soils data, aerial photographs and history of problems at site.

**Step 2** Determine design discharge.

**Step 3** Develop design cross-section(s).

**Step 4** Compute design water surface:

- When evaluating the design water surface, Manning’s n should be estimated. An estimate of the rock size may be required to determine the roughness coefficient.
- Perform hydraulic computations using Manning’s equation.

**Step 5** Determine design shear stress/depth:

- Average depth should be determined for the design section in conjunction with the computations of Step 4. In general, the average depth in the main flow channel should be used.
If riprap is being designed to protect channel banks, adjustments to applied shear stress and riprap stability must be applied.

**Step 6** Determine riprap size/thickness required to resist particle erosion.

**Step 7** If an assumed $D_{50}$ was used in determination of Manning’s $n$ for backwater computations and roughness, repeat Steps 4 through 6 until Manning’s roughness matches within 0.005 units for computed roughness.

**Step 8** Select final $D_{50}$ riprap size, set material gradation and determine riprap layer thickness. If final $D_{50}$ riprap size is different than that derived from Step 7, repeat Steps 4 through 8.

**Step 9** Determine longitudinal extent of protection required (Section 3.5.3.2).

**Step 10** Determine appropriate vertical extent of revetment (Section 3.5.3.3).

**Step 11** Select geotextile filter (Section 3.5.3.4 and this section).

**Step 12** Design edge details (flanks and toe) (See this section).

### 3.5.6 Grouted Riprap

Grouted riprap is rock slope paving with voids filled with concrete grout forming a monolithic armor. Because fully grouted riprap is a rigid structure, it will not conform to bank settlement or toe undermining as loose riprap does. Therefore, fully grouted riprap is susceptible to mass failure, especially if pore water is not allowed to drain properly. Although the revetment is rigid, it is not particularly strong and even a small loss of toe or bank support can result in the failure of large portions of the structure.

#### 3.5.6.1 Design Guidelines

Guidance for design of grouted riprap is presented in *Design of Riprap Revetment* (HEC 11, 1989). Grouted riprap is also addressed in the more recent *Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance* (HEC 23, 2009) which cites the potential for mass failure of the structure. As a result, design guidance is not provided for conventional grouted riprap in HEC 23. Due to the potential for mass failure inherent in this design, the HEC 11 method for grouted riprap design is not recommended for ADOT projects.

To minimize the risk of mass failure of the revetment, grouted riprap for use on ADOT projects should meet the following:

- Riprap rock ($D_{50}$) should be sized to resist the hydraulic forces and remain stable without reliance on the grout.
- Grout should be placed from the bottom of the riprap layer filling all the voids up to the specified grout thickness.
- A geotextile filter/drainage fabric should be provided with all grouted riprap installations.
- A method to relieve pore water pressure must be included in the design.
3.5.6.2 Manning’s Roughness

Per *Design of Roadside Channels with Flexible Linings* (HEC 15, 2005), roughness for grouted riprap installations range from 0.028 to 0.040 with a typical value of 0.030.

3.5.6.3 Stone Size and Gradation

The characteristic $D_{50}$ stone size for grouted riprap should be determined using the same procedures as required for loose riprap described in Section 3.5.5.3. Because of the use of grout to fill the voids instead of reliance on smaller stone sizes, a narrower gradation can be used for grouted riprap installations. The recommended gradations for grouted riprap also have a smaller maximum size which may be beneficial in situations where large rock is not economically available. With loose riprap gradations the maximum rock size ranges from approximately 1.5 to 2.1 times the $D_{50}$ size. The recommended grouted riprap gradations have maximum rock sizes ranging from 1.1 to 1.4 times the $D_{50}$ size.

The rock gradation presented below in Table 3–6 is adapted from Table 6 in HEC 11 (HEC-11, 1989) and includes six classes of grouted rock bank lining.

<table>
<thead>
<tr>
<th>Equivalent Diameter (in)</th>
<th>Class (A)</th>
<th>Class (B)</th>
<th>Class (C)</th>
<th>Class (D)</th>
<th>Class (E)</th>
<th>Class (F)</th>
</tr>
</thead>
<tbody>
<tr>
<td>42</td>
<td>95-100</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>33</td>
<td>0-50</td>
<td>95-100</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>27</td>
<td>--</td>
<td>0-50</td>
<td>95-100</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>21</td>
<td>0-5</td>
<td>--</td>
<td>0-50</td>
<td>95-100</td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>0-5</td>
<td>--</td>
<td>0-50</td>
<td>95-100</td>
<td></td>
<td></td>
</tr>
<tr>
<td>12</td>
<td></td>
<td>0-5</td>
<td>0-5</td>
<td>0-50</td>
<td>95-100</td>
<td></td>
</tr>
<tr>
<td>Minimum Grout Thickness (in)</td>
<td>24</td>
<td>18</td>
<td>14</td>
<td>10</td>
<td>8</td>
<td>6</td>
</tr>
</tbody>
</table>

Another approach to design of grouted riprap is presented in Urban Drainage and Flood Control District, *Drainage Criteria Manual* (UDFCD, 2008). UDFCD uses the term “grouted boulders” to describe uniformly sized boulders with no fines that require filling of the entire void space with grout starting at the bottom of the layer to a depth equal to one-half the boulder size and keeping the upper one-half ungrouted and clean. The boulders are classified by size from 18 inch to 48-inch. The variation in boulder size for each class varies by 3 to 6-inches, respectively. Guidance from UDFCD Drainage Criteria Manual should be followed if this design approach is used.
3.5.6.4 Grout and Rock Layer Characteristics

The grout for grouted riprap should be placed from the bottom of the rock layer, filling all the voids up to the specified thickness. The rock layer thickness should be the same as the largest size rock used in the layer. The minimum grout thickness is shown in Table 3-6 for each class of riprap. The grout depth is one-half the boulder size if the UDFCD approach is used.

3.5.6.5 Filter Requirements

Relief of hydrostatic pressure is one of the primary design considerations for grouted riprap installations. Use of geocomposite drains and weep holes is the preferred means of facilitating passage of groundwater to weep holes.

Geocomposite drains are required under all grouted riprap revetments to provide a zone of high permeability to carry off seepage water and prevent damage to the overlying structure from uplift pressures.

Weep holes should be provided in the revetment to relieve hydrostatic pressure buildup behind the grout surface (see Figure 3-14). Weep holes should extend through the grout surface to the interface with the underdrain. Weep holes should consist of 3-inch diameter pipes having a maximum horizontal spacing of 6 feet and a maximum vertical spacing of 10-feet. The buried end of the weep hole should be covered with wire screening or a fabric filter of a gage that will prevent passage of the underlying material.

3.5.6.6 Layout

The edges of grouted riprap revetments (the head, toe, and flanks) require special treatment to prevent undermining. The revetment toe should extend to a depth below anticipated scour depths or to bedrock. The toe should be designed as illustrated in Figure 3-14. After excavating to the desired depth, the rock slope protection should be extended to the bottom of the trench and grouted. The remainder of the excavated area in the toe trench should be filled with grout-free riprap. The grout-free riprap provides extra protection against undermining at the bank toe.

To prevent outflanking of the revetment, various edge treatments are required. Recommended designs for these edge treatments are similar to loose riprap as shown in Figure 3-12.
3.5.7 Wire-Tied Rock

Wire-tied rock revetment consists of rock held in place with a wire mesh. The most common types of wire-tied revetments are mattresses and stacked blocks. Rail bank is a form of wire-tied revetment that is a continuous framework rather than individual interconnected baskets.

3.5.7.1 Design Guidelines

ADOT Standard Specifications differentiate between stacked gabion applications (wire-tied riprap), gabion mattresses (slope mattress riprap), and rail bank. All three are presented in ADOT Standard Specifications Section 913.

Guidance for design of wire-tied rock is contained primarily in two FHWA documents. Design of Riprap Revetment (HEC 11, 1989) provides comprehensive guidelines for layout, bank and foundation preparation, basket size and configuration, stone size, stone quality, basket or enclosure fabrication, edge treatment and filter design. Bridge Scour and Stream Instability Countermeasures; Experience, Selection, and Design Guidance (HEC 23, 2009) provides guidelines for rock sizing and selection of factors of safety based on research more current than HEC 11. Applicable guidance from HEC 23 is provided in:

- Design Guideline 6, “Wire Enclosed Riprap Mattress” (Rail Bank)
- Design Guideline 10, “Gabion Mattresses at Bridge Piers”
3.5.7.2 Manning’s Roughness

Manning’s roughness for wire-tied rock is the same as for loose riprap of the same size. The procedure for computing Manning’s roughness in Section 3.5.5.2 is used for wire-tied rock described within this section.

3.5.7.3 Stone Size and Layer Thickness

Design of wire-tied rock revetment is based on the concept of permissible shear stress. The procedure is described in Section 10.3 of HEC 23, Design Guideline 10 (HEC 23, 2009). The design shear stress is the computed shear stress applied to the revetment as determined by hydraulic analysis. The permissible shear stress is the shear stress that can be resisted by the wire enclosure filled with a particular rock size. The permissible shear stress used for design of the revetment should be equal to or greater than the design shear stress times the selected factor of safety. The recommended factor of safety is based on selection of a base factor of safety for the application and multiplying the base factor of safety by multipliers to account for consequence of failure and model uncertainty.

The presence of the enclosing mesh makes the permissible shear stress greater for this application than for the same size stone in a loose riprap installation. Exceedance of the permissible threshold indicates material will move within the cells which can compromise the integrity of the lining by creating an alternatingly bulging and thinning profile along the revetment.

Successful long-term performance of wire-tied rock depends largely on the integrity of the wire. Due to the potential for abrasion by coarse bed load, all three forms of wire-tied rock discussed in this section are not appropriate for gravel bed streams and should only be considered for use in sand or fine-bed streams. Additionally, water quality of the stream must be noncorrosive (i.e., nonsaline and nonacidic). A polyvinyl chloride (PVC) coating should be used for applications where the potential for corrosion exists.

Stone size, quality and gradation, as well as factor of safety for permissible shear and layer thickness should be developed per HEC 23 guidance. The selected rock size will dictate the minimum thickness of the gabion basket. The next standard basket size larger than the selected rock size should be specified. If the selected rock size exceeds the size specified in the ADOT Standard Specifications, a special provision will need to be included in the contract documents with the required rock size as a supplement to the Standard Specifications.

3.5.7.4 Filter Requirements

Geotextile fabric is required beneath all wire-tied rock revetment. Section 913-2.05 “Bank Protection Fabric” in the Standard Specifications indicates the standard requirements for high survivability, nonwoven filter fabric.
3.5.7.5 Gabion Mattresses

Gabion mattress revetments consist of flat wire baskets or units filled with rock laid end to end and side to side on a prepared channel bed and/or bank. The individual mattress units are wired together to form a continuous revetment mattress. A typical gabion mattress channel bank lining installation is shown on Figure 3–15.

![Figure 3–15 Gabion Mattress Channel Bank Lining](image)

**Layout**

When used for channel bank lining the longitudinal and vertical extent of the protection should be as described in Section 3.5.3. However, instead of excavating to the full scour depth for placement of the gabion mattresses, a toe apron may be used. The toe apron should have the gabion mattresses oriented with the long axis normal to the flow direction and extend across the bed 1.5 times the expected maximum depth of scour in the vicinity of the revetment toe. In areas where little toe scour is expected, the apron can be replaced by a single-course gabion toe wall that helps to support the revetment and prevent undermining. Where an excessive amount of
toe scour is anticipated, both an apron and a toe wall can be used. The concept of gabion toe apron launching is illustrated on Figure 3–16.

To provide extra strength at the revetment flanks, it is recommended that mattress units having additional thickness be used at the upstream and downstream edges of the revetment (See Figure 3–17). It is further recommended that a thin layer of topsoil be spread over the flank units to form a soil layer to be seeded when the revetment installation is complete.

Figure 3–16  Gabion Toe Apron Launching
(HEC 23, 2009)
Mattress Size and Configuration

Gabion mattresses are available in a number of sizes. Gabion mats are available in 99 foot long units which are suitable for bank lining applications on large projects and are quicker to install than standard gabion baskets. Revet mattresses (slope mattress riprap) are smaller units ranging in size from 6 to 12-inches thick, 6 feet wide with lengths of 9 and 12-feet. Table 3–7 shows standard sizes for gabion mats and Table 3–8 shows standard sizes for Revet mattress units.

**Table 3–7  Standard Gabion Mat Sizes**

<table>
<thead>
<tr>
<th>Thickness (ft)</th>
<th>Width (ft)</th>
<th>Length (ft)</th>
<th>Wire-Mesh Opening Size (in. × in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>6</td>
<td>99</td>
<td>3.25 × 4.0</td>
</tr>
<tr>
<td>1.0</td>
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<td>99</td>
<td>3.25 × 4.0</td>
</tr>
<tr>
<td>1.5</td>
<td>9</td>
<td>99</td>
<td>3.25 × 4.0</td>
</tr>
</tbody>
</table>

**Table 3–8  Standard Revet Mattress Sizes**

<table>
<thead>
<tr>
<th>Thickness (ft)</th>
<th>Width (ft)</th>
<th>Length (ft)</th>
<th>Wire-Mesh Opening Size (in. × in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td>6</td>
<td>9</td>
<td>2.5 × 3.25</td>
</tr>
<tr>
<td>0.5</td>
<td>6</td>
<td>12</td>
<td>2.5 × 3.25</td>
</tr>
<tr>
<td>0.75</td>
<td>6</td>
<td>9</td>
<td>2.5 × 3.25</td>
</tr>
<tr>
<td>0.75</td>
<td>6</td>
<td>12</td>
<td>2.5 × 3.25</td>
</tr>
<tr>
<td>1.0</td>
<td>6</td>
<td>12</td>
<td>2.5 × 3.25</td>
</tr>
</tbody>
</table>
Design Procedure

Section 10.3.5 of HEC 23 (HEC 23, 2009, DG10) contains a gabion mattress design example. The example is presented in a series of steps that can be followed by the designer in order to select the appropriate thickness of the gabion mattress based on a pre-selected target factor of safety. The primary criterion for product selection is if the computed factor of safety for the armor meets or exceeds the pre-selected target value. The steps are summarized below. The computations required for each step are described within HEC 23.

**Step 1** Determine a target factor of safety for the project.

**Step 2** Calculate design shear stress.

**Step 3** Calculate permissible shear stress.

**Step 4** Calculate factor of safety.

**Step 5** Specify the gabion mattress.

### 3.5.7.6 Stacked Gabions

Stacked block gabion revetments consist of rectangular wire baskets that are filled with stone and stacked in a stepped-back fashion to form the revetment surface. They are also commonly used at the toe of embankment slopes as toe walls that help to support other upper bank revetments and prevent undermining.

The rectangular basket or gabion units used for stacked configurations are of more uniform dimensions than those typically used for mattress designs. That is, they typically have a square cross-section. Commercially available gabions used in stacked configurations are available in various sizes but the most common have a 3-feet width and thickness. A typical stacked gabion channel bank lining installation is shown on Figure 3–18.
Layout

Stacked gabion revetments are used instead of gabion mattress designs where the slope to be protected is greater than 3H:1V or where the purpose of the revetment is for flow training. They can be used as retaining structures where space limitations prohibit bank grading to a slope suitable for other revetments. When used for channel bank lining the longitudinal and vertical extent of the protection should be as described in Section 3.5.3. Stacked-gabion revetments must be based on a firm foundation. The foundation or base elevation of the structure should be well below the computed scour depth. Where the placement of the stacked gabions below the design scour depth is impractical, a gabion mattress toe apron should be implemented as illustrated on Figure 3–19. The minimum gabion toe apron length should be 1.5 times the anticipated scour depth below the apron. This length can be increased in proportion to the level of uncertainty in predicting the local toe scour. Toe aprons are also recommended in alluvial streams where channel bed fluctuations are common and where the estimated scour depth is uncertain.

The flanks of stacked-block gabion revetments also require special attention. The upstream and downstream flanks of these revetments should include counterforts; see Figure 3–19. The counterforts should be placed 12 to 18-feet from the upstream and downstream limits of the structure and should extend a minimum of 12-feet into the bank.
**Gabion Size and Configuration**

Gabion baskets used for stacked block gabion revetment are typically the 3 foot by 3 foot size which are available in lengths of 6, 9, and 12-feet. Standard gabion basket sizes are shown in Table 3–9.
### Table 3–9 Standard Gabion Sizes

<table>
<thead>
<tr>
<th>Thickness (ft)</th>
<th>Width (ft)</th>
<th>Length (ft)</th>
<th>Wire-Mesh Opening Size (in. × in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0</td>
<td>3</td>
<td>6</td>
<td>3.25 × 4</td>
</tr>
<tr>
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<td>9</td>
<td>3.25 × 4</td>
</tr>
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<td>3.25 × 4</td>
</tr>
<tr>
<td>3.0</td>
<td>3</td>
<td>12</td>
<td>3.25 × 4</td>
</tr>
</tbody>
</table>

#### 3.5.7.7 Rail Bank

Rail bank protection is a special form of wire-tied rock. Rail bank utilizes sections of railroad rail to anchor a metal mesh which is backfilled with stone varying in size from 4 to 12-inches. (ADOT Standard Specifications Section 913-2.01G). Standard drawings for rail bank may be found in ADOT Standard Drawings C-17.10, C-17.15, and C-17.20. Additional specifications for materials and construction of rail bank are located in ADOT Standard Specifications Section 913.

Construction of rail bank is usually faster than gabions or gabion mattresses, and it also requires less wire mesh because internal junction panels are not used. It has been used for bank protection, guide bank slope protection, and in conjunction with gabions placed at the toe of slope. Rail bank may be constructed in multiple configurations. In cases where a bank is available, Types 1, 2, 3, 4, 5, and 6 rail bank are best suited. Types 7, 8, and 9 rail bank are intended for construction as free-standing structures.

**Layout**

Rail bank is subject to the same hydraulic design guidelines as gabions. Rail bank must be designed such that the toe of the rock is below the design scour depth. If the design scour depth exceeds the toe depth, the rock backfill may be undermined and fail.
3.5.8 Articulated Concrete Blocks

Articulating concrete block systems (ACBs) provide a flexible alternative to riprap, wire-tied rock and rigid revetments. These systems consist of preformed units which either interlock, are held together by cables, or both to form a continuous blanket or block matrix.

3.5.8.1 Design Guidelines

Guidance for design of ACBs is contained in Design Guideline 8 of Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance (HEC 23, 2009).

3.5.8.2 Manning’s Roughness

The Manning’s n-value used for computing the hydraulic properties should be the roughness for the selected ACB system. As a result, the hydraulic model may need to be run for each ACB system under evaluation. The final hydraulic model should reflect the roughness of the selected ACB system. Manning’s n-values for each system should be provided by the manufacturer along with other required design parameters.

3.5.8.3 Hydraulic Design

Application 1 in Section 8.3 of Design Guideline 8 “Articulating Concrete Block Systems” (HEC 23, 2009) provides a hydraulic design procedure for ACB systems for channel bank lining or bed
armoring. The design procedure is based on hydraulic stability using a tractive shear approach whereby the permissible shear of the ACB system is greater than the tractive shear applied by the flowing water to meet a desired minimum factor of safety. The selection of a factor of safety utilizes a base value based on the application. The base value is increased to account for consequence of failure and uncertainty in the hydrologic/hydraulic modeling.

### 3.5.8.4 Layout

When used for channel bank lining the longitudinal and vertical extent of the protection should be as described in Section 3.5.3. The toe down should extend to the full scour depth. Figure 3–20 shows the recommended layout detail for top termination and toe down. Similar termination trenches are recommended for the upstream and downstream limits of the ACB revetment.

![Recommended ACB Revetment Layout Detail](HEC 23, 2009)

### 3.5.8.5 Filter Requirements

A suitable filter layer beneath the blocks, and in some cases a drainage layer of granular or synthetic material, are considered to be an integral component(s) of the overall ACB system. A drainage layer may be used in conjunction with an ACB system. A drainage layer lies between the blocks and the geotextile and/or granular filter. This layer allows "free" flow of water beneath the block system while still holding the filter material to the subsoil surface under the force of
the block weight. This free flow of water can relieve sub-block pressure and has appeared to significantly increase the hydraulic stability of ACB systems.

3.5.8.6 Design Procedure

The design of an ACB system consists of the following steps which are elaborated on in Design Guideline 8 (HEC 23, 2009).

Step 1 Determine a target factor of safety for the project.

Step 2 Calculate design shear stress.

Step 3 Obtain ACB properties.

Step 4 Calculate the factor of safety parameters for each product. There are nine equations that must be calculated for each candidate block system.

Step 5 Select and specify the appropriate block.

3.5.9 Soil-cement

Soil-cement is a densely compacted mixture of portland cement, soil/aggregate, other cementitious materials (possibly), and water. In areas where high quality rock is scarce, the use of soil-cement can provide a practical countermeasure alternative for channel stability and scour protection. Soil-cement has been used to construct drop structures and to armor embankments, dikes, levees, and channels.

3.5.9.1 Design Guidelines

Guidance for design of soil-cement channel bank lining is contained in Design Guideline 7 of Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance (HEC 23, 2009). While ADOT does not have a Standard Specification for soil-cement, a typical specification is provided in Supp 9130401 (913SLCMT) of the stored specifications. Cement Stabilized Alluvium (CSA) is a special form of soil-cement that utilizes a larger aggregate size of up to 3-inches that is used primarily for riverine revetment projects. Standard soil-cement is limited to a maximum aggregate size of 1.5 inches.

3.5.9.2 Manning’s Roughness

Design of Roadside Channels with Flexible Linings (HEC 15, 2005) lists a range of Manning’s roughness values for soil-cement from 0.020 to 0.025 with a typical value of 0.022.

3.5.9.3 Layout

On most projects, soil-cement is constructed in stair-step fashion by placing and compacting the soil-cement in horizontal layers stair-stepped up the embankment (Figure 3–22). A compacted layer, or “lift”, thickness of 6-inches is most widely used, with the recommended maximum being 9-inches for efficient, uniform compaction and facilitates placement using common highway
construction equipment. Embankment slopes from 1H:1V to 2H:1V are most common for stair-step construction. A minimum thickness of 3.5 feet, measured normal to the face of the slope or projected slope, is required and the horizontal layer width may need to be adjusted to provide this thickness.

An important consideration in the layout of soil-cement revetment is to ensure that all extremities are tied into non-erodible sections or abutments. Adequate freeboard and carrying the soil-cement to the paved roadway, plus a lower-section detail as shown in Figure 3–22, will minimize erosion from behind the crest and under the toe of the facing. The ends of the facing should terminate smoothly into the bank and be tied into the bank with counterforts.

Where miscellaneous structures (e.g., culverts) extend through the facing, the area immediately adjacent to such structures is constructed by placing and compacting the soil-cement by hand or with small equipment or by using a lean-mix concrete.

3.5.9.4 Design Considerations

Important design considerations for soil-cement include: types of materials and equipment used, mix design and methods, handling, placing, and curing techniques. These items are addressed in Design Guideline 7 (HEC 23, 2009) and in the ADOT stored specifications.

A wide variety of soils can be used to make durable soil-cement slope protection. The Portland Cement Association (PCA) has data on soil types, gradations, costs and testing procedures. The PCA also has data on placement and compaction methods.
Subsidence

Embarkment subsidence results from a compressible foundation, settlement within the embankment itself or both. Analyzing the possible effects of such a condition involves a number of assumptions by the designer concerning the embankment behavior. Combining these assumptions with the characteristics of the facing, a structural analysis of the condition can be made. The layer effect can usually be ignored.

Note: The post-construction appearance of a pattern of narrow surface cracks approximately 10 to 20-feet apart is evidence of normal hardening of the soil-cement. Substantial embankment subsidence conceivably could allow the facing to settle back in large sections coinciding with the normal shrinkage crack pattern. If such settlement of the soil-cement with separation at the shrinkable cracks occurs, the slope remains adequately protected unless the settlement is large enough to allow the outer face of a settling section to move past the inner face of an adjoining section.

Rapid Drawdown

Rapid drawdown exceeding 15-feet or more within a few days theoretically produces hydrostatic pressure from moisture trapped in the embankment against the back of the facing. Three design concepts that may be used to prevent damage due to rapid drawdown-induced pressure are:

1. Design the embankment so that its least permeable zone is immediately adjacent to the soil-cement facing, which ensures that seepage through cracks in the facing will not build up a pool of water sufficient to produce damaging hydrostatic pressure.
2. Evaluate the stability of the soil-cement mass using a gravity wall approach.
3. Provide free drainage behind, through or under the soil-cement facing to prevent adverse hydrostatic pressure.

3.6 SPURS AND GUIDE BANKS

Large channels and highway embankments may require protection from design discharges, but may not require a continuous revetment across the channel due to cost or permitting reasons. In these instances, use of bank protection elements, such as spurs or guide banks, which contact and redirect flows away from channel banks and highway embankments, may be warranted.

Design of these structures comprises two general phases. First, the structure must be designed to perform the necessary hydraulic diversion. This phase of design involves establishing adequate structure spacing, length, height, and orientation relative to flow. The second phase involves designing the structure to withstand the imposed hydraulic forces due to contact with flow. This phase involves design of suitable armoring of the structure including scour estimation.

For channel applications, these structures are not as common as channel bank lining in ADOT projects to provide erosion protection. This is likely due to a variety of factors including increased design complexity and scour susceptibility.
Additionally, analysis of impacts from these features requires specialized hydraulic modeling techniques. In general, use of single-section analysis is not appropriate for determining the impacts of these structures due to their backwater-generating nature. Step-backwater analysis is required at a minimum and two-dimensional modeling may be warranted depending upon the level of detail required. Single-section analysis may be useful for computation of hydraulic parameters at the point of maximum impingement, but should not be relied upon to determine the flow behavior in transitional areas or between structures.

Spurs and guide banks do not represent an exhaustive list of bank protection structures, but they are representative of the bank protection structures most commonly used in ADOT projects. Other bank protection features may be incorporated into designs with approval from ADOT. HEC 23, Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance (HEC 23, 2009) provides a more extensive listing of potential countermeasures.

3.6.1 Spurs

Spurs are structures which jut into the flow from a channel bank or embankment. The angle relative to the flow and vertical profile of the spur can be adjusted to achieve a variety of effects and impacts the spacing between spurs.

Detailed guidance related to the design of spurs is provide in HEC 23 Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance (HEC 23, 2009), Design Guideline 2 – Spurs.

Calculation of scour and armoring needs at spurs are essential for their continued performance. Due to their flow impingement, spurs are subject to local and contraction scour components, in addition to scour components within the broader channel. Scour at the leading toe of the spur is especially common and may lead to failure of the spur during high flow events if adequate volume or toe-down are not provided.

3.6.2 Guide Banks

Guide banks (also known as “spur dikes”) are generally located at areas where channel width transitions abruptly such as at a bridge crossing. Guide banks are used to promote flow efficiency through the constriction and reduce the potential for lateral erosion caused by rapid constriction of the flow.

Detailed guidance related to the design of guide banks is provided in HEC 23 Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance (HEC 23, 2009).

3.7 DESIGN PROCEDURES

3.7.1 Ditch Step-by-Step Procedure

Each project is unique, but the following basic design steps are normally applicable:

Step 1 Establish a Roadside Plan.
Collect available site data.

Identify potential ditch inlet and outlet locations and transitions.

**Step 2** Obtain or Establish Cross-section Data.

- Provide channel depth adequate to drain the sub-base.
- Choose ditch side slopes based on geometric design criteria including safety, economics, soil, aesthetics, and access.
- Identify features that may restrict cross-section design:
  i) Right-of-way limits,
  ii) Roadway clear-zone/recovery areas,
  iii) Utilities, and
  iv) Existing drainage facilities.

**Step 3** Determine Ditch Grades.

- Ditch grades will be primarily determined by surrounding grades. Slopes in roadside ditch in cuts are usually controlled by highway grades.
- Provide minimum grade per DDM Volume 1 - Policy & Guidelines to minimize ponding and sediment accumulation.
- Consider influence of type of lining on grade.

**Step 4** Check Flow Capacities and Adjust as Necessary (note that Step 4 and Step 5 may be inter-related and iterative).

- Compute the design discharge at the downstream end of a ditch (see DDM Volume 2 - Hydrology).
- Set preliminary values of ditch size, roughness coefficient and slope.
- Determine maximum allowable flow depth of ditch including freeboard.
- Check flow capacity using Manning’s equation and single-section analysis or step backwater analysis, as appropriate.
- If capacity is inadequate, possible adjustments are as follows:
  i) Increase bottom width,
  ii) Make channel side slopes flatter,
  iii) Make channel slope steeper, and
  iv) Provide smoother channel lining.
Step 5 Determine Channel Lining/Protection Needed.

- Analyze suitability of in-situ material. If found inadequate, investigate options under Step 4, item 5. If these types of alterations cannot be achieved cost effectively, then:
  - Select appropriate lining materials/methods from Sections 3.5.2 and Section 3.5.
  - Estimate the flow depth and choose an initial Manning’s n value based upon the lining selected.
  - Calculate normal flow depth, \( y_o \) (ft), at design discharge using Manning’s equation or step-backwater analysis. Determine if the roughness assumed in the Step 5.b matches the lining selected and flow conditions. If not, adjust roughness and iterate until allowable tolerance is achieved.

- Assess lining adequacy based upon design procedures outlined in Section 3.5. Otherwise consider the following options (in no particular order):
  - Choose a more erosion-resistant lining,
  - Decrease ditch slope, and
  - Increase ditch width and/or flatten side slopes.

Step 6 Analyze Outlet Points and Downstream Effects.

- Identify any adverse impacts (e.g., increased flooding or erosion to downstream properties) that may result from one of the following at the ditch outlet:
  - Increase or decrease in discharge,
  - Increase in velocity of flow,
  - Concentration of sheet flow, or
  - Change in outlet water quality.

- Mitigate any adverse impacts identified in Step 6.1. Possibilities include:
  - Enlarge ditch outlet and/or install control structures to provide detention of increased runoff in ditch,
  - Install velocity-control structures (energy dissipators),
  - Increase capacity and/or improve lining of downstream channel,
  - Install sedimentation/infiltration basins, and
v) Install diversion dike and culvert/storm drain connection.

3.7.2 Roadside Channel Step-by-Step Procedure

Each project is unique, but the following six basic design steps are normally applicable.

**Step 1** Establish a Roadside Plan.
- Collect available site data.
- Obtain or prepare existing and proposed plan-profile layout including highway, culverts and bridges.
- Determine and plot on the plan the locations of natural basin divides and roadside channel outlets.

**Step 2** Obtain or Establish Cross-section Data.
- Identify features that may restrict cross-section design:
  - Right-of-way limits,
  - Trees or environmentally sensitive areas,
  - Utilities, and
  - Existing drainage facilities.
- Provide required channel depth per DDM Volume 1 - Policy & Guidelines.
- Choose channel side slopes based on geometric design criteria including safety, economics, soil, aesthetics, and access.
- Establish bottom width of trapezoidal channel. This may be set by typical section or as necessary for rock containment or maintenance.

**Step 3** Determine Initial Channel Grades.
- Plot initial grades on plan-profile layout. (Slopes in roadside ditch in cuts are usually controlled by highway grades).
- Provide minimum grade to minimize ponding and sediment accumulation.
- Consider influence of type of lining on grade.
- Where possible, avoid features that may influence or restrict grade (e.g., utility locations).

**Step 4** Check Flow Capacities and Adjust as Necessary.
- Compute the design discharge at the downstream end of a channel segment (see DDM Volume 2 - Hydrology).
Set preliminary values of channel size, roughness coefficient and slope.

Determine maximum allowable depth of channel including freeboard.

Check flow capacity using Manning’s equation and single-section analysis or step-backwater analysis, as appropriate.

If capacity is inadequate, possible adjustments are as follows:
   i) Increase bottom width,
   ii) Make channel side slopes flatter,
   iii) Make channel slope steeper,
   iv) Provide smoother channel lining, and
   v) Provide smooth transitions at changes in channel cross-sections.

Provide extra channel storage where needed to replace floodplain storage and/or to reduce peak discharge.

Step 5  Determine Channel Lining/Protection Needed.

Analyze suitability of in-situ material. If found inadequate, investigate options under Step 4, item 5. If these types of alterations cannot be achieved cost effectively, then:
   i) Select appropriate lining materials/methods from Sections 3.5.3.5 and Section 3.5.
   ii) Estimate the flow depth and choose an initial Manning’s n value based upon the lining selected.
   iii) Calculate normal flow depth, $y_0$ (ft), at design discharge using Manning’s equation or step-backwater analysis. Determine if the roughness assumed in the Step 5.b matches the lining selected and flow conditions. If not, adjust roughness and iterate until allowable tolerance is achieved.
   iv) Assess lining adequacy based upon design procedures outlined in Section 3.5. Otherwise consider the following options (in no particular order):
      1. Choose a more erosion-resistant lining,
      2. Decrease channel slope (with or without grade control structures), and
      3. Increase channel width and/or flatten side slopes.

Step 6  Analyze Outlet Points and Downstream Effects.
Identify any adverse impacts (e.g., increased flooding or erosion to downstream properties) that may result from one of the following at the channel outlet:

i) Increase or decrease in discharge,
ii) Increase in velocity of flow,
iii) Concentration of sheet flow,
iv) Change in outlet water quality, or
v) Diversion of flow from another watershed.

Mitigate any adverse impacts identified in Step 6.1. Possibilities include:

i) Enlarge outlet channel and/or install control structures to provide detention of increased runoff in channel,
ii) Install velocity-control structures (energy dissipators),
iii) Increase capacity and/or improve lining of downstream channel, and
iv) Install sedimentation/infiltration basins.

3.7.3 Natural Channel Step-by-Step Procedure

The analysis of a natural channel in most cases is in conjunction with the design of a highway hydraulic structure such as a culvert or bridge. In general, the objective is to convey the water along or under the highway such that it will not cause damage to the highway, stream or adjacent property. An assessment of the existing channel is usually necessary to determine the potential for problems that might result from a proposed action. The detail of studies necessary should be commensurate with the risk associated with the action and with the environmental sensitivity of the stream and adjoining floodplain.

Although the following step-by-step procedure may not be appropriate for all possible applications, it does outline a process that will usually apply.

Step 1  Assemble Site Data and Project File.

Data Collection (see Data Collection Chapter):

i) Topographic, site, and location maps,
ii) Roadway profile,
iii) Photographs,
iv) Field reviews,
v) Design data at nearby structures,
vi) Gaging records, and
vii) Historic flood data and local knowledge.

- **Studies by other agencies:**
  i) Flood insurance studies,
  ii) Floodplain studies, and
  iii) Watershed studies.
- **Environmental constraints:**
  i) Floodplain encroachment,
  ii) Floodway designation,
  iii) Fish and wildlife habitat, and
  iv) Commitments in review documents.
- **Design criteria:**
  i) See Section 3.2.

**Step 2** Determine the Project Scope.

- **Determine level of assessment:**
  i) Stability of existing channel,
  ii) Potential for damage, and
  iii) Sensitivity of the stream.
- **Determine type of hydraulic analysis:**
  i) Qualitative assessment,
  ii) Single-section analysis,
  iii) Step-backwater analysis, and
  iv) Special analysis techniques.
- **Determine additional survey information:**
  i) Extent of streambed profiles,
  ii) Locations of cross-sections,
  iii) Elevations of flood-prone property,
  iv) Details of existing structures, and
Step 3
Evaluate Hydrologic Variables and Compute Discharges for Selected Frequencies (see DDM Volume 2 - Hydrology).

Step 4
Perform Hydraulic Analysis:

- Either single-section analysis (Section 3.4.3)
  - Select representative cross-section (Section 3.4.1),
  - Select appropriate n-values (Section 3.4.2), and
  - Compute stage-discharge relationship.
- Or Step-backwater analysis (Section 3.4.3).
  - Or special analysis techniques (Section 3.4.6), and
  - Calibrate with known high-water, if possible.

Step 5
Perform Stability Analysis.

- Geomorphic factors.
- Hydraulic factors.
- Stream response to change.

Step 6
Design Countermeasures.

- Criteria for selection:
  - Erosion mechanism,
  - Stream characteristics,
  - Construction and maintenance requirements,
  - Vandalism considerations, and
  - Cost.
- Types of countermeasures:
  - Meander migration countermeasures,
  - Bank stabilization (see Bank Protection Chapter).
  - Bend control countermeasures,
  - Channel braiding countermeasures,
  - Degradation countermeasures, and
vi) Aggradation countermeasures.

For additional information:

i) HEC 20 *Stream Stability at Highway Structures* (HEC 20, 2001),

ii) HDS No. 6 *River Engineering for Highway Encroachments* (HDS 6, 2001), and

iii) See Reference List.

Step 7 Documentation.

Prepare report and file with background information.

3.8 REFERENCES


ADWR. State Standard SS4-95 *Identification and Development within Sheet Flow Areas*. Arizona Department of Water Resources. 1995. (ADWR, SS4-95)


Li, Ruh-Ming; Cotton, George K., Zeller, Michael Z.; Simons, Daryl B., Deschamps, Patricia O. *Effects of In-Stream Mining on Channel Stability*. Arizona Department of Transportation. June 1989. (Li et. al., 1989)


U.S. Army Corps of Engineers. HEC-6, Scour and Deposition in Rivers and Reservoirs, User’s Manual. The Hydrologic Engineering Center, Davis, California, August 1993. (HEC-6, 1993)


U.S. Army Corps of Engineers. Users Guide to RMA2 WES Version 4.5. Engineer Research and Development Center, Waterways Experiment Station, Coastal and Hydraulics Laboratory, Vicksburg, MS, 2006. (RMA2, 2006)

APPENDIX 3A
MANNING’S n
### Table 3–10 Values of Manning’s Roughness Coefficient n (Uniform Flow) (Chow, 1959)

<table>
<thead>
<tr>
<th>Type of Channel and Description</th>
<th>Minimum</th>
<th>Normal</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>EXCAVATED OR DREDGED</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Earth, straight and uniform</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clean, recently completed</td>
<td>0.016</td>
<td>0.018</td>
<td>0.02</td>
</tr>
<tr>
<td>Clean, after weathering</td>
<td>0.018</td>
<td>0.022</td>
<td>0.025</td>
</tr>
<tr>
<td>Gravel, uniform section, clean</td>
<td>0.022</td>
<td>0.025</td>
<td>0.03</td>
</tr>
<tr>
<td>With short grass, few weeds</td>
<td>0.022</td>
<td>0.027</td>
<td>0.033</td>
</tr>
<tr>
<td>Earth, winding and sluggish</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>No vegetation</td>
<td>0.023</td>
<td>0.025</td>
<td>0.03</td>
</tr>
<tr>
<td>Grass, some weeds</td>
<td>0.025</td>
<td>0.03</td>
<td>0.033</td>
</tr>
<tr>
<td>Dense weeds or aquatic plants in deep channels</td>
<td>0.03</td>
<td>0.035</td>
<td>0.04</td>
</tr>
<tr>
<td>Earth bottom and rubble sides</td>
<td>0.025</td>
<td>0.03</td>
<td>0.035</td>
</tr>
<tr>
<td>Stony bottom and weedy sides</td>
<td>0.025</td>
<td>0.035</td>
<td>0.045</td>
</tr>
<tr>
<td>Cobble bottom and clean sides</td>
<td>0.03</td>
<td>0.04</td>
<td>0.05</td>
</tr>
<tr>
<td>Dragline-excavated or dredged</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>No vegetation</td>
<td>0.025</td>
<td>0.028</td>
<td>0.033</td>
</tr>
<tr>
<td>Light brush on banks</td>
<td>0.035</td>
<td>0.05</td>
<td>0.06</td>
</tr>
<tr>
<td><strong>Rock cuts</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Smooth and uniform</td>
<td>0.025</td>
<td>0.035</td>
<td>0.04</td>
</tr>
<tr>
<td>Jagged and irregular</td>
<td>0.035</td>
<td>0.04</td>
<td>0.05</td>
</tr>
<tr>
<td><strong>Channels not maintained, weeds, and brush uncut</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dense weeds, high as flow depth</td>
<td>0.05</td>
<td>0.08</td>
<td>0.12</td>
</tr>
<tr>
<td>Clean bottom, brush on sides</td>
<td>0.04</td>
<td>0.05</td>
<td>0.08</td>
</tr>
<tr>
<td>Same, highest stage of flow</td>
<td>0.045</td>
<td>0.07</td>
<td>0.11</td>
</tr>
<tr>
<td>Dense brush, high stage</td>
<td>0.08</td>
<td>0.1</td>
<td>0.14</td>
</tr>
<tr>
<td><strong>NATURAL STREAMS</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Minor streams (top width at flood stage &lt; 100 ft)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Streams on Plain</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clean, straight, full stage, no rifts or deep pools</td>
<td>0.025</td>
<td>0.03</td>
<td>0.033</td>
</tr>
<tr>
<td>Same as above, but more stones/weeds</td>
<td>0.03</td>
<td>0.035</td>
<td>0.04</td>
</tr>
<tr>
<td>Clean, winding, some pools/shoals</td>
<td>0.033</td>
<td>0.04</td>
<td>0.045</td>
</tr>
<tr>
<td>Same as above, but some weeds/stones</td>
<td>0.035</td>
<td>0.045</td>
<td>0.05</td>
</tr>
<tr>
<td>Same as above, lower stages, more ineffective slopes and sections</td>
<td>0.04</td>
<td>0.048</td>
<td>0.055</td>
</tr>
<tr>
<td>Same as 4, but more stones</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sluggish reaches, weedy, deep pools</td>
<td>0.045</td>
<td>0.05</td>
<td>0.06</td>
</tr>
</tbody>
</table>
### Table 3-10 Values of Manning’s Roughness Coefficient n (Uniform Flow) (Chow, 1959)

<table>
<thead>
<tr>
<th>Type of Channel and Description</th>
<th>Minimum</th>
<th>Normal</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush</td>
<td>0.05</td>
<td>0.07</td>
<td>0.08</td>
</tr>
<tr>
<td>Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages</td>
<td>0.075</td>
<td>0.1</td>
<td>0.15</td>
</tr>
<tr>
<td>Bottom: gravels, cobbles and few boulders</td>
<td>0.030</td>
<td>0.040</td>
<td>0.030</td>
</tr>
<tr>
<td>Bottom: cobbles with large boulders</td>
<td>0.040</td>
<td>0.050</td>
<td>0.070</td>
</tr>
<tr>
<td>Floodplains</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pasture, no brush</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Short grass</td>
<td>0.025</td>
<td>0.030</td>
<td>0.035</td>
</tr>
<tr>
<td>High grass</td>
<td>0.030</td>
<td>0.035</td>
<td>0.050</td>
</tr>
<tr>
<td>Cultivated area</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>No crop</td>
<td>0.020</td>
<td>0.030</td>
<td>0.040</td>
</tr>
<tr>
<td>Mature row crops</td>
<td>0.025</td>
<td>0.035</td>
<td>0.045</td>
</tr>
<tr>
<td>Mature field crops</td>
<td>0.030</td>
<td>0.040</td>
<td>0.050</td>
</tr>
<tr>
<td>Brush</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Scattered brush, heavy weeds</td>
<td>0.035</td>
<td>0.050</td>
<td>0.070</td>
</tr>
<tr>
<td>Light brush and trees, in winter</td>
<td>0.035</td>
<td>0.050</td>
<td>0.060</td>
</tr>
<tr>
<td>Light brush and trees, in summer</td>
<td>0.040</td>
<td>0.050</td>
<td>0.080</td>
</tr>
<tr>
<td>Medium to dense brush, in winter</td>
<td>0.045</td>
<td>0.070</td>
<td>0.110</td>
</tr>
<tr>
<td>Medium to dense brush, in summer</td>
<td>0.070</td>
<td>0.100</td>
<td>0.160</td>
</tr>
<tr>
<td>Trees</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dense willows, summer, straight</td>
<td>0.110</td>
<td>0.150</td>
<td>0.200</td>
</tr>
<tr>
<td>Cleared land with tree stumps, no sprouts</td>
<td>0.030</td>
<td>0.040</td>
<td>0.050</td>
</tr>
<tr>
<td>Same as above, but with heavy growth of sprouts</td>
<td>0.050</td>
<td>0.060</td>
<td>0.080</td>
</tr>
<tr>
<td>Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches</td>
<td>0.080</td>
<td>0.100</td>
<td>0.120</td>
</tr>
<tr>
<td>Same as above, but with flood stage reaching branches</td>
<td>0.100</td>
<td>0.120</td>
<td>0.160</td>
</tr>
<tr>
<td>Major Streams (top width at flood stage &gt;100 ft)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Regular section with no boulders or brush</td>
<td>0.025</td>
<td>--</td>
<td>0.160</td>
</tr>
<tr>
<td>Irregular and rough sections</td>
<td>0.035</td>
<td>--</td>
<td>0.100</td>
</tr>
</tbody>
</table>
APPENDIX 3B
PERMISSIBLE VELOCITIES
### Table 3-11 Grass and Earth-Lined Channels (HEC 14, FHWA)

<table>
<thead>
<tr>
<th>Channel Slope</th>
<th>Lining</th>
<th>Permissible Velocity (ft/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-5%</td>
<td>Bermuda grass</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>Reed canary grass</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>Tall fescue</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>Kentucky bluegrass</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>Grass-legume mixture</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>Red fescue</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>Redtop</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>Sericea lespedeza</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>Annual lespedeza</td>
<td>4</td>
</tr>
<tr>
<td>5-10%</td>
<td>Bermuda grass</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>Reed canary grass</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>Tall fescue</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>Kentucky bluegrass</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>Grass-legume mixture</td>
<td>3</td>
</tr>
<tr>
<td>&gt;10%</td>
<td>Bermuda grass</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>Reed canary grass</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>Tall fescue</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>Kentucky bluegrass</td>
<td>3</td>
</tr>
</tbody>
</table>

- For highly erodible soils, decrease permissible velocities by 25%
Table 3–12 Permissible Velocities for Water Flow Conditions (Bare Soil)  
(Special Committee on Irrigation Research, ASCE, 1926)

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Clear-water (ft/sec)</th>
<th>w/ Fine Silts (ft/sec)</th>
<th>w/ Sand and Gravel (ft/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fine Sand (noncolloidal)</td>
<td>1.5</td>
<td>2.5</td>
<td>1.5</td>
</tr>
<tr>
<td>Sandy Loam (noncolloidal)</td>
<td>1.7</td>
<td>2.5</td>
<td>2</td>
</tr>
<tr>
<td>Silt Loam (noncolloidal)</td>
<td>2</td>
<td>3</td>
<td>2</td>
</tr>
<tr>
<td>Ordinary Firm Loam</td>
<td>2.5</td>
<td>3.5</td>
<td>2.2</td>
</tr>
<tr>
<td>Fine Gravel</td>
<td>2.5</td>
<td>5</td>
<td>3.7</td>
</tr>
<tr>
<td>Graded, Loam to Cobbles</td>
<td>3.7</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>Graded, Silt to Cobbles (noncolloidal)</td>
<td>4</td>
<td>5.5</td>
<td>5</td>
</tr>
<tr>
<td>Alluvial Silts (noncolloidal)</td>
<td>2</td>
<td>3.5</td>
<td>2</td>
</tr>
<tr>
<td>Alluvial Silts (colloidal)</td>
<td>3.7</td>
<td>5</td>
<td>3</td>
</tr>
<tr>
<td>Coarse Gravels (noncolloidal)</td>
<td>4</td>
<td>6</td>
<td>6.5</td>
</tr>
<tr>
<td>Cobbles and Shingles</td>
<td>4</td>
<td>6</td>
<td>6.5</td>
</tr>
<tr>
<td>Shales and Hard Pan</td>
<td>6</td>
<td>6</td>
<td>5</td>
</tr>
</tbody>
</table>
APPENDIX 3C
EXAMPLE COMPUTATIONS
An example of channel hydraulic calculations are provided below. This examples is included to demonstrate the application of the procedures.

**Example No. 3-1  Trapezoidal Channel**

**Problem:**

Find normal depth and velocity for a trapezoidal channel

**Given:**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Q</td>
<td>800 cfs</td>
</tr>
<tr>
<td>B</td>
<td>8 ft. (Channel Bottom Width)</td>
</tr>
<tr>
<td>Z</td>
<td>4 (Side Slope 4:1)</td>
</tr>
<tr>
<td>n</td>
<td>0.03</td>
</tr>
<tr>
<td>S&lt;sub&gt;o&lt;/sub&gt;</td>
<td>0.005 ft/ft</td>
</tr>
</tbody>
</table>

**Solution:**

1. Determine normal depth

   The Manning’s/Continuity Equation is calculated as:

   \[
   Q = \frac{1.49}{n} S_0^{0.5} A^{5/3} P^{2/3}
   \]

   For a trapezoidal channel, the flow area, \( A \), can be calculated as a function of the bottom width, \( B \), flow depth, \( d \), and side slopes, \( Z \):

   \[
   A = Bd + d^2Z
   \]

   Similarly, the wetted perimeter can be calculated as a function of the bottom width, \( B \), flow depth, \( d \), and side slopes, \( Z \):

   \[
   P = B + 2d(Z^2 + 1)^{0.5}
   \]

   The solution for \( Q \) is non-linear and a trial and error method may be used to solve for the flow depth, \( d \), as follows:
<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>b (ft)</th>
<th>Z</th>
<th>n</th>
<th>S₀</th>
<th>Flow Area (sq ft)</th>
<th>Wetted Perimeter (ft)</th>
<th>Calculated Discharge (cfs)</th>
<th>Q (cfs)</th>
<th>Error (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>8</td>
<td>4</td>
<td>0.03</td>
<td>0.05</td>
<td>12.00</td>
<td>16.25</td>
<td>108.90</td>
<td>800</td>
<td>-691.10</td>
</tr>
<tr>
<td>2</td>
<td>8</td>
<td>4</td>
<td>0.03</td>
<td>0.05</td>
<td>32.00</td>
<td>24.49</td>
<td>424.73</td>
<td>800</td>
<td>-375.27</td>
</tr>
<tr>
<td>2.5</td>
<td>8</td>
<td>4</td>
<td>0.03</td>
<td>0.05</td>
<td>45.00</td>
<td>28.62</td>
<td>675.83</td>
<td>800</td>
<td>-124.17</td>
</tr>
<tr>
<td>2.7</td>
<td>8</td>
<td>4</td>
<td>0.03</td>
<td>0.05</td>
<td>50.76</td>
<td>30.26</td>
<td>795.78</td>
<td>800</td>
<td>-4.22</td>
</tr>
<tr>
<td>2.7067</td>
<td>8</td>
<td>4</td>
<td>0.03</td>
<td>0.05</td>
<td>50.96</td>
<td>30.32</td>
<td>800.00</td>
<td>800</td>
<td>0.00</td>
</tr>
</tbody>
</table>

For the given channel, a normal flow depth of 2.71 ft is developed at a discharge of 800 cfs.

2. Determine channel velocity.

Using the general form of the continuity equation

\[ Q = VA \]
\[ V = \frac{Q}{A} \]
\[ V = \frac{800}{50.96} \]
\[ V = 15.7 \text{ ft/s} \]
Example No. 3-2  Design Riprap Channel Bank Lining

Problem:
Determine riprap $D_{50}$ for channel bank lining

Given:

- $B = 25$ ft. (Channel Bottom Width)
- $Q = 800$ cfs.
- $Z = 4$ (Side Slope 4:1)
- $S_o = 0.005$ ft/ft.
- $n = 0.028$ (native soil)
- $S_g = 2.5$
- Riprap Angle of Repose = 38°

Solution:

1. Assume initial $n$-value of 0.035 for riprap channel bank lining, initial riprap $D_{50} = 0.5$ ft.
2. Next determine preliminary riprap size based upon HEC-15 (Equations 6.8/6.15)
   \[ D_{50} > K_1 SF d_s / K_2 (F(S_g - 1)) \]
3. Hydraulic values for this example were calculated using FHWA's Hydraulic Toolbox (v4.2) with the "irregular" channel option. The irregular channel option was used to allow for varying $n$-values to be applied at the channel banks.
A spreadsheet is useful for iterating selection of appropriate n-value and riprap $D_{50}$. The initial assumptions of n-value and riprap size are evaluated for reasonableness and iterated as necessary. Riprap is typically available in sizes such as 6, 12, 18, and 24-inches and calculated values should be rounded-up to reflect reasonably available materials.

<table>
<thead>
<tr>
<th>n (input)</th>
<th>d (ft)</th>
<th>V (ft/s)</th>
<th>$Re$</th>
<th>F</th>
<th>SF</th>
<th>$K_1$</th>
<th>$K_2$</th>
<th>Calc’d $D_{50}$ (ft)</th>
<th>Use $D_{50}$ (in)</th>
<th>da/ $D_{50}$</th>
<th>Calc’d n</th>
<th>n Error</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.035</td>
<td>3.17</td>
<td>0.71</td>
<td>4.34E+04</td>
<td>0.05</td>
<td>1.01</td>
<td>0.93</td>
<td>0.63</td>
<td>0.32</td>
<td>6</td>
<td>6.33</td>
<td>0.0493</td>
<td>0.0143</td>
</tr>
<tr>
<td>0.0493</td>
<td>3.27</td>
<td>0.73</td>
<td>4.42E+04</td>
<td>0.05</td>
<td>1.01</td>
<td>0.93</td>
<td>0.63</td>
<td>0.33</td>
<td>6</td>
<td>6.54</td>
<td>0.0490</td>
<td>0.0003</td>
</tr>
</tbody>
</table>

General Table note: Refer to HEC 15 for definition and selection of appropriate values

Based upon these calculations, the initial assumption of $D_{50} = 0.5$ ft was appropriate, but the initially estimated n-value had an error of 0.0143 units when compared to the calculated value using the Blodgett method shown in HEC-15. The 2nd iteration resulted in an appropriately close result which indicates that a riprap $D_{50} = 0.5$ ft (6 in) is adequate and will generate a flow depth of 3.27 ft.
APPENDIX 3D
CHANNEL LINING DESIGN GUIDELINES
URBAN HIGHWAYS CHANNEL LINING DESIGN GUIDELINES

Introduction

This publication is intended as an Executive Summary for channel lining designs. It was prepared to establish design criteria for concrete channel linings for the East Papago/ Hohokam Freeway project. This criteria can also be adopted for similar channel lining designs on other Arizona Department of Transportation (ADOT) projects. The study was conducted by gathering, reviewing and analyzing various published documents from Federal, State, and local agencies which are involved in the design and construction of concrete channel linings. These documents are included in a separate publication titled Report on Concrete Lined Drainage Channels dated February, 1989, which also contains this Executive Summary.

Design Criteria

There are three distinct approaches to the design of channel lining reinforcement for canals and hydraulic drainage structures. Two of these approaches utilize expansion and/or contraction joints with the reinforcement varying from 0.0 to 0.5 percent depending on joint spacing. The third approach utilizes no joints with continuous reinforcing varying from 0.3 to 0.4 percent depending on climatic conditions. The design criteria of most agencies reviewed fall within one or more of these approaches and the resulting linings have performed satisfactorily for the most part.

The following items were considered in reaching the conclusion that a continuously reinforced lining without joints will provide the most cost-effective and serviceable channel lining:

- Review of lining performance relative to minimal cracking in continuously reinforced channels.
- Reported and observed maintenance problems of weeds growing in unsealed joints.
- High maintenance cost of replacing damaged joints.
- Moderate difference in construction cost between continuous versus discontinuous reinforcement.
- Potential local compressive buckling of lining due to open joints being filled with incompressible material.
- Potential infiltration of water through open expansion or contraction joints into moisture sensitive soils.

A 0.3 percent longitudinal reinforcement for a moderate climate was found reasonable for adoption to the Phoenix area, (it is performing well on the U.S. Army Corps of Engineers (Corps) Arizona Canal Diversion Channel). A minimum 0.2 percent transverse reinforcement for moderate climates is also considered reasonable for narrow to medium width channels up to 70 feet wide. For wider channels, it is considered realistic to increase the percentage based on the subgrade drag method to a maximum of 0.3 percent.

The criteria for the channel invert lining thickness follows two basic approaches and relates either to velocity or the presence of corrosive materials in the channel bed. Review of collected data
indicates that the current general practice for establishing base slab thickness for channels without corrosive material follows closely the U.S. Soil Conservation Service (SCS) nomograph for thickness versus water velocity and it is concluded that bottom lining thickness should be based on this criteria. The minimum thickness, however, is dictated by two considerations—reinforcing clearance, and access for maintenance vehicles. Based on the support capacity of the stabilized moisture sensitive soils, a minimum thickness of 6-inches is required for maintenance vehicle access. A minimum 3-inch clearance for corrosion protection limits the minimum lining thickness to 6-inches for tied reinforcement and 5-inches for flat mesh.

Special consideration of thickness needs on a site specific basis will be required for unusual hydraulic and soil conditions. Energy dissipators and extreme breaks in grade will require special design to ensure that lining thickness can resist potential negative pressures. Areas of collapsing or expansive soils require evaluation of thickness in conjunction with subgrade treatment to ensure a serviceable lining.

Slope paving thickness criteria (excluding SCS) generally approximates 80 percent of bottom thickness and is considered a reasonable approach. The limitations on minimum thickness for reinforcement and vehicle access applies to slope paving as well as bottom thickness. It was found that slope paving has even been placed vertically in transition areas, but it is the general consensus that a 1.5:1 maximum slope is more reasonable for maintenance of a quality product. In no circumstance should the slope exceed the soils angle of repose without being formed and designed as a retaining wall. In keeping with a continuously reinforced lining, it is concluded that transitions from trapezoidal channels to rectangular cross-sections should be accomplished without warped slope paving which the Corps is using on the ACDC.

A review of concrete qualities in the various standards indicates a wide range of values but with a general need for shrinkage and crack control. Concrete mix designs will be required which achieve a low drying shrinkage, while also maintaining strength requirements and constructability.

A review of soil conditions in the Phoenix area indicates the area is well suited for construction of concrete lined channels. Most areas will require only a scarification and recompack of surface soils without pressure relief. Minor areas of moisture sensitive collapsing or expansive soils will require partial or total removal and replacement with compacted fill based upon a site specific evaluation. Pressure relief and seepage barrier requirements in these moisture sensitive areas will also require special consideration.

The method of pressure relief on channel lining has generally been gravel pockets with weep holes. Problems with silting have been identified which create a maintenance requirement. Use of pressure relief flap valves in conjunction with geocomposite drainage strips is considered a better solution than weep holes. The need for pressure relief will require evaluation on a site specific basis not only for soil condition but for potential groundwater infiltration from heavy irrigation and special conditions such as parallel or crossing utilities.

The use of transverse cutoff walls has generally been eliminated from most design criteria (except at the start and end of a lining), and are not considered to be needed for elimination of seepage.
or progressive failure. A need does exist for transverse stiffening or stabilizing walls in continuously reinforced linings at movement sensitive structures and where unbalanced compressive forces may occur.

General details were reviewed to reduce construction and maintenance problems. Top of slope paving cutoff walls are a general practice and are needed to eliminate erosion and ground water seepage. A vertical wall set back from the top of slope sufficiently to allow machine trenching provides easier construction. A 2 percent cross slope to one side of the channel bottom (a minimum slope of 6-inches is recommended) provides a means to transport sediment during low flows. Access ramps should be located on the high side of the channel and slope downstream where possible to reduce hydraulic disturbance and sediment buildup. Grade control structures should be constructed with sufficient open area at the channel floor to allow flushing of sediment during low flows.

**Recommendations**

We recommend the following “Design Guidelines for Concrete Lined Drainage Channels”:

1. Channel lining shall be continuously reinforced without expansion or tooled joints except as follows. Construction joints shall be located at the end of a day’s pour or when concrete placement stops for more than 45 minutes and between longitudinal paving strips. Longitudinal construction joints shall be located 1-foot up the side slope and in the bottom slab as dictated by channel width but not within the low flow section. Reinforcing steel shall be continuous through lining construction joints and through joints with box culverts and other hydraulic structures.

2. Reinforcing steel shall be Grade 60 or flat sheet welded wire fabric and have the following percentage ratios (p) of reinforcement to cross-section area of concrete.  

   **Longitudinal Reinforcement:**  
   \[ p = 0.30\% \]

   **Transverse Reinforcement:**  
   \[ \text{Channel Width} * \quad p \]
   
   less than 70 feet \quad 0.20%  
   70 to 90 feet \quad 0.25%  
   more than 90 feet \quad 0.30%  

   *Total width including side slopes

   Reinforcing steel shall have a minimum 3-inch clearance to grade and a maximum size of #4 for 6 inch lining thickness.

3. Minimum lining thickness for trapezoidal channels shall be:

   **Bottom Slab**

   **Mean Water Velocity (fps)**  
   **Thickness (inches)**
less than 10  
10 to 15  
15 to 20  
more than 20

Mean Water Velocity (fps)  
Thickness (inches)

less than 15  
15 to 20  
more than 20

Side Slopes

*Minimum slab thickness of 6-inches is required for use of tied reinforcement and in channels wide enough to accommodate maintenance vehicles.

Lining thickness and channel profile shall be investigated on a site specific basis where negative pressures might occur such as a change from a light to a steeper slope per Corps manual EM 1110-2-1602 and 1603.

Lining thickness and reinforcement shall be investigated on a site specific basis in conjunction with subsoil treatment where collapsing or expansive soils occur.

4. Side slopes on main channels should not exceed 1.5 horizontal to 1.0 vertical or the recommended maximum safe cut slope (Table 2) and preferably should not exceed 2.0 to 1.0.

If side slopes which are steeper than the recommended safe cut slope are used for warped transitions, lining shall be designed as retaining walls for lateral earth pressures listed in Table 2.

5. Sealed vertical expansion joints shall be provided at bridge piers and abutments.

6. Transverse cutoff or stiffening walls which are rigidly attached to the paving shall be installed in the following locations:
   a. At the beginning and end of concrete lining unless terminating in a movement stable structure.
   b. Where new lining abuts an existing concrete lining that is not designed with continuous reinforcement. A transverse sealed expansion joint should be provided between new and existing linings.
   c. At the upstream or start of a transition section to widen the channel.
   d. At breaks in channel profile where the increase in slope exceeds 0.5 degree or 0.009 ft./ft.
e. Immediately upstream and downstream of movement sensitive structures such as intersecting drainage channels. This shall be evaluated on a project specific basis.

7. Continuous 12-inch deep vertical cutoff walls with a top elevation 6-inches below natural grade shall be provided at the top of side slopes 2-foot back of the top of slope. The 2-foot horizontal section shall have a 2 percent slope toward the channel. Cutoff walls shall be increased to 24-inches deep where substantial flows occur.

8. Bottom lining shall have a cross slope to one side of 2 percent with a minimum of 6-inches of slope.

9. Access ramps shall be located upstream and downstream of box culverts and other hydraulic structures that will not allow vehicular access. Ramps should be located on the high side of the channel invert and slope in a downstream direction where possible.

10. Transitions from a trapezoidal cross-section to a rectangular cross-section should be made with a varying height vertical retaining wall (Figure 3–9) instead of warped side slopes. The retaining walls are to be designed for earth pressures listed in Table 2.

11. 0-Gee control structures should be constructed with a 30 to 50 percent opening at the base slab for flushing.

12. Subgrade treatment shall be on a site specific basis in accordance with recommendations for the five typical subsurface profile cases in Table 1 and shall result in a minimum Modulus of Subgrade Reaction of 200 pci. Detailed discussions of subsurface profile cases and recommendations are found in Appendix A of the East Papago/Hohokam Freeway “Design Guidelines for Concrete Lined Drainage Channels.”

13. Pressure relief of channel linings shall be accomplished with geotextile or geocomposite drainage strips and 4” diameter PVC weepholes through the lining in accordance with recommendations for the five subsurface profile cases in Table 1. Weepholes should be located 1-foot vertically above channel bottom and slope down 3” from back to face of lining. Plastic flap type relief valves should be considered if available and a workable detail can be developed.

   Project and site specific evaluation will be required based on subsurface investigations, potential future changes in ground water levels, where structural backfill occurs adjacent to channel, and at parallel or crossing utilities.

14. Concrete strengths, mix design and drying shrinkage evaluation shall be in accordance with recommendations in Table 3. Detailed recommendations for concrete design mix and shrinkage criteria are found in Appendix A of the East Papago/Hohokam Freeway “Design Guidelines for Concrete Lined Drainage Channels.”
Table 1
Recommended Subgrade Treatment, Drainage & Pressure Relief Procedure
For The Five Typical Subsurface Profiles In The Greater Phoenix Area

<table>
<thead>
<tr>
<th>Subsurface Profile Case</th>
<th>Description</th>
<th>Subgrade Treatment</th>
<th>Drainage &amp; Pressure Relief</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Clean sands or sands &amp; gravels</td>
<td>• No special treatment required</td>
<td>• Pressure relief not required unless potential exists for groundwater to rise above canal bottom</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Scarification &amp; recompaction of surface soils</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Cemented desert alluvium</td>
<td>• No special treatment required</td>
<td>• Low risk of water accumulation: Pressure relief not required</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Scarification &amp; recompaction of surface soils</td>
<td>• High risk of water accumulation: ex: Extended flow periods adjacent water/sewer lines, heavy landscaping, potential groundwater rise</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>• Geocomposite drainage strips with pressure relief weepholes</td>
</tr>
<tr>
<td>3</td>
<td>Moisture sensitive soils over poorly drained cemented desert alluvium</td>
<td>Collapsing soils:</td>
<td>Low risk of water accumulation: Pressure relief not required</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• 4-feet thick: partial over-excavation, wetting, vibratory compaction &amp; replacement with compacted fill</td>
<td>High risk of water accumulation: ex: Extended flow periods adjacent water/sewer lines, heavy landscaping, potential groundwater rise</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Full removal &amp; replacement with compacted fill</td>
<td>• Geocomposite drainage strips with pressure relief weepholes</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Expansive Soils:</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Partial or total removal &amp; replacement with compact fill</td>
<td>Low risk of water accumulation: Pressure relief not required</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Geomembrain underliner as seepage barrier</td>
<td>High risk of water accumulation: ex: Extended flow periods adjacent water/sewer lines, heavy landscaping, potential groundwater rise</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>• Geocomposite drainage strips with pressure relief weepholes</td>
</tr>
<tr>
<td>Subsurface Profile Case</td>
<td>Description</td>
<td>Subgrade Treatment</td>
<td>Drainage &amp; Pressure Relief</td>
</tr>
<tr>
<td>-------------------------</td>
<td>-----------------------------------------------------------------------------</td>
<td>------------------------------------------------------------------------------------------------------</td>
<td>---------------------------------------------------------------------------------------------</td>
</tr>
</tbody>
</table>
| 4                       | Moisture sensitive soils over free draining granular stata                    | Collapsing soils  
- 4-feet thick: partial over-excavation, wetting, vibratory compaction & replacement with compacted fill  
- Full removal & replacement with compacted fill  
Expansive soils  
- Partial or total removal & replacement with compacted fill  
- Geomembrane underliner as seepage barrier | Pressure relief not required unless potential exists for groundwater to rise above canal bottom |
| 5                       | Expansive clays throughout profile                                           | Over excavate & replace with nonexpansive compacted fill: depth of over excavation as required to limit potential expansion to tolerable limits | Low risk of water accumulation: Pressure relief not required  
High risk of water accumulation: ex: Extended flow periods adjacent water/sewer lines, heavy landscaping, potential groundwater rise  
Geocomposite drainage strips with pressure relief weepholes |

Note: Methodology for design of geocomposite drainage systems can be found in "Designing for Rows:”, A.M. Koerner, Civil Engineering, Volume 56, No. 10, October 1986, and "Designing with Geosynthetics”, A.M. Koerner, Prentice Hall International, New Jersey, 1986
### Table 2

**Recommended Engineering Design Parameters**

For Subsurface Conditions 1 Through 5

<table>
<thead>
<tr>
<th>Case</th>
<th>Subsurface Conditions</th>
<th>Slopes</th>
<th>Modules of Subgrade Reaction, pci</th>
<th>Lateral Earth Pressures Against Retaining Walls</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Cut</td>
<td>Fill</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0</td>
<td>10</td>
</tr>
<tr>
<td>1</td>
<td>Clean sands or sand &amp; gravel</td>
<td>2:1</td>
<td>2:1</td>
<td>600</td>
</tr>
<tr>
<td>2</td>
<td>Moderately to strongly cemented alluvial soils</td>
<td>1:1</td>
<td>1:1</td>
<td>750</td>
</tr>
<tr>
<td>3</td>
<td>Moisture sensitive (collapsing or expansive soils over cemented alluvium)</td>
<td>1:1</td>
<td>1:1</td>
<td>200</td>
</tr>
<tr>
<td>4</td>
<td>Moisture sensitive (collapsing or expansive) soils over granular free-draining soils</td>
<td>2:1</td>
<td>2:1</td>
<td>200</td>
</tr>
<tr>
<td>5</td>
<td>Medium to highly expansive clays throughout entire profile</td>
<td>1:1</td>
<td>1:1</td>
<td>600</td>
</tr>
</tbody>
</table>

**Notes:**

1. Recommended slope ratios are horizontal to vertical. Slopes are maximum safe slopes. In most cases, slopes will be controlled by construction considerations and will be no steeper than 1.5:1.
2. Moduli of subgrade reaction for Cases 3, 4, and 5 for wet conditions are based on the moisture sensitive soils not being stabilized or replaced with structural fill. Values for dry conditions for these cases apply to stabilized moisture sensitive soils of structural fills.
3. “Active” case for lateral earth pressures applied to conditions in which the retaining wall is free to move at the top. The “At Rest” case applies where walls are restrained from movement at the top. The angle \(\beta\) refers to the slope angle the backfill from the horizontal.
Design Mix

Design mix should meet the general specification requirements of ADOT 1006-3

Strength

Compressive strength should be 3,000 psi at 28 days.

Aggregates

Aggregates should meet minimum requirements of ADOT Standard Specification 1006-3. Coarse aggregate should be size 57. Coarse aggregate should have minimum of 75 percent crushed faces.

Mineral Filler

Ninety (90) pounds of fly ash Class F (ASTM C618) shall be used as a filler. Loss on ignition should be a maximum of 3.0 percent. Fly ash should not be considered as a replacement for cement. Fly ash should have an R factor less than 2.5. The R factor is defined as (C-5%)/F, where C is the calcium oxide content expressed as a percentage and F is the ferric oxide content expressed as percentage. The R factor requirement may be waived if the contractor furnishes documented test results that the soil in contact with the Portland Cement concrete contains less than 0.10 percent water soluble sulfate, (as S04) and/or the water in contact with the Portland Cement concrete contains less than 150 milligrams per liter sulfate (as S04). The tests for sulfates should be performed in accordance with the requirements of California Department of Transportation Test Method No. 417. Calcium and ferric oxide content should be determined accordance with the requirements of ASTM C311

Chemical Admixtures

Should meet the requirements of ADOT 1006-2.04.

Water

Should meet the requirements of ADOT 1006-2.02.

Cement

Should be Portland Cement Type II, meeting the requirements of ASTM C150.

Slump

Maximum 4-inches (AASHTO T119).
Air Content

5 plus or minus 2 percent by volume (AASHTO T-152).

Curing

Should meet the requirements of ADOT 1006-6 A.
Subgrade shall be moistened and free of excess standing water prior to placement of concrete.

Hot Weather Concreting

Should meet the requirements of ADOT 1006-5.02.

Minimum Cement Content

Not applicable.

Drying Shrinkage Evaluation

Mortar Shrinkage Tests

ASTM C157, "Length Change of Hardened Cement Mortar and Concrete," testing should be performed on the cement proposed for the project design concrete mix. If other than previously approved Type II cement is proposed, the shrinkage of the cement should be equal to or less than the value obtained in the control specimens made from previously approved cements which result in the lowest practicable shrinkage.

Field Shrinkage Tests

Test panels should be prepared with the proposed concrete design mix for the purpose of evaluating drying shrinkage properties. Test panels should be made in accordance with the Kraai Method outlined in Concrete Construction, Volume 30, No. 9, September, 1985, 9 pp. 775-788. Test panels shall be 2 by 3 feet in plan dimension and 2-inches in thickness.

The Control Test Panel should be made from an established reference mix design. Locally produced Salt River aggregate should be used. Minimum compressive strength should be 3,000 psi at 28 days. Fly ash, as a pozzolanic material, should be utilized as a mineral filler at a maximum of 90 pounds per cubic yard of concrete. A water reducing admixture should be used meeting the requirements of ADOT 1006-2.04.

The project design mix acceptance should have an equal or reduced number and size of shrinkage cracking as compared to the Control Mix Test Panel.
Chapter 4
CULVERTS

This chapter contains details on the following:

- Culvert design concepts, process and procedures
- Culvert types and hydraulics

4.1 INTRODUCTION

This chapter provides guidance for design of culverts crossing roadways. The chapter includes an overview of culvert hydraulics to aid in understanding the conditions that control or limit culvert performance, including inlet control and outlet control. The FHWA computer software package HY-8 can be used to automate the design computations. The chapter also describes the culvert design process recommended for use on ADOT projects. Many of ADOT’s projects involve the extension or modification of existing culvert installations. Therefore, a section is included to provide guidance for design of culvert extensions and retrofits.

4.1.1 Design Resources

The hydraulics of culverts is described in FHWA’s Hydraulic Design Series No. 5, Hydraulic Design of Highway Culverts (HDS 5, 2012). HDS 5 is the definitive work on culvert design and includes the applicable charts, equations, and theory needed for culvert analysis and design. It is anticipated that culvert designers will utilize HDS 5 as a primary resource. Therefore, equations, procedures, and figures from HDS 5 are not included in this chapter. The procedures contained in HDS 5 have been incorporated into the HY-8 Computer program (HY-8, 2014). These resources are recommended for use on ADOT projects and are summarized below:

- **Hydraulic Design of Highway Culverts, Hydraulic Design Series No. 5**, Third Edition (HDS 5, 2012) – This manual is the principal culvert design handbook that ADOT utilizes for its culvert design procedures.
- **HY-8 Computer Program** (HY-8, 2014) – HY-8 offers steady-state hydraulic analysis of all culvert types discussed in HDS 5. HY-8 also provides a direct computational interface with many of the energy dissipation structures discussed in Chapter 5.

Another important ADOT resource for culvert design is:

- **Pipe Selection Guidelines and Procedures** (ADOT, 1996) - provides guidelines and procedures to determine pipe materials to be included in the construction plans as bid
alternates. A methodology for achieving the minimum required service life for approved pipe materials is also provided.

## 4.2 HYDRAULIC ANALYSIS

### 4.2.1 Culvert Performance

Culvert analysis is based upon the assumption that water to be passed through the culvert will generally be in a ponded condition at the inlet end of the culvert. As a result, the upstream headwater elevation is the ponded water surface elevation with no velocity component. Culvert design involves selecting a culvert size and configuration such that the upstream headwater elevation for the design storm event meets established criteria. The headwater elevation for a particular culvert configuration and discharge can be governed by a number of possible conditions as described in the following section.

### 4.2.1.1 Inlet Control vs. Outlet Control

The point within the culvert system that controls the discharge capacity for a given upstream headwater elevation can be at the culvert inlet or at the culvert outlet. An understanding of inlet control and outlet control will help optimize the culvert design. Culvert capacity control locations are illustrated on Figure 4–1. Under inlet control conditions, the culvert inlet acts as a weir under low headwater conditions or as an orifice under submerged inlet conditions. As long as the culvert is operating in inlet control, the downstream conditions will have no impact on the upstream headwater elevation and therefore, the culvert capacity. If the culvert barrel is long, relatively flat or rough, or if the downstream tailwater elevation is high, the inlet orifice may be “drowned out” and the limiting headwater condition may be under outlet control. Outlet control can result from a high tailwater elevation created from the downstream channel hydraulics or it may be from friction losses within the culvert barrel. As long as the culvert is in outlet control, the inlet conditions will have minimal impact on the upstream headwater elevation and therefore, the culvert capacity.
Types of flow control are further described in HDS 5. The emphasis here is that an understanding of the flow control will enable the designer to optimize the design. An optimal hydraulic design exists when the barrel capacity and the inlet capacity are equal and sufficient for the design.
discharge. Additionally, the most cost effective combination of culvert layout, size, and material that meets the optimal hydraulic design would be the optimal culvert configuration.

4.2.1.2 Culvert Performance Illustration

The design implications of inlet and outlet control can be illustrated with an example. The example is based on a single barrel 6x5 foot reinforced concrete box culvert with a length of 250 feet and discharge of 300 cfs. The maximum allowable headwater for design is 1.5 times the culvert height, which is 7.5 feet. Figure 4–2 shows the inlet control and outlet control headwater elevations for a range of culvert slopes. As you can see, the inlet control headwater is relatively independent of culvert slope and is approximately 7.2 feet. At very mild slopes, the culvert is in outlet control but as the slope is steepened it quickly becomes inlet controlled. The optimal design where inlet and outlet control are nearly matched is near a slope of 0.001. As the culvert slope is increased, the outlet control headwater continues to drop. At a slope of 0.015 the normal depth of flow in the box culvert is 2.6 feet which is slightly more than half full. At a slope of 0.02 the culvert is flowing less than half full. Based on the fact that inlet control is governing the design, the culvert inlet cannot be downsized. Note, however, that the culvert barrel has a lot of excess capacity which is unused. Once the flow is directed into the barrel a smaller barrel size would be adequate for the flow. In the case of costly long or large culverts that are in inlet control there may be an economic benefit to selecting a smaller culvert barrel and designing an improved inlet. Alternatively, if the site conditions allow, a flatter culvert slope may be designed which would take advantage of some of the otherwise unused barrel capacity to slow the flow velocity which would then reduce the impacts or energy dissipator requirements at the culvert outlet. A profile of the example culvert generated by HY-8 is also shown on Figure 4–2 with the water surface profile illustrating the unused barrel capacity.
4.2.1.3 Culvert Inlet Analysis

Cross culverts under a highway in Arizona frequently function under inlet control conditions. As a result, the inlet configuration plays an important role in the culvert capacity. A sharp square inlet edge is the least efficient inlet shape due to the contraction of flow as it accelerates from the upstream ponded condition into the culvert barrel. Making the flow contraction smoother and moving the contraction upstream of the culvert barrel will improve the inlet efficiency. Adding a simple bevel at the inlet of concrete culverts is a cost effective way to improve the inlet (See Figure 4–2). More elaborate means to improve inlet performance include side-tapered and slope-tapered inlets.

Beveled Inlets

The box culvert example in the previous section was completed using a 1:1 inlet bevel on the box culvert which is consistent with ADOT standard details. To illustrate the benefit of the inlet bevel, the same culvert from the previous example was reanalyzed for the 0.015 ft/ft slope condition with a square edge inlet. Using a square edge instead of the 1:1 bevel inlet increases the headwater depth from 7.16 feet to 7.89 feet for the design flow of 300 cfs. The headwater increase of 0.73 feet causes the culvert to exceed the allowable headwater elevation of 7.5 feet which would require that the culvert be upsized if the square edge inlet were to be used.

Figure 4–2 Culvert Performance (HY8)
Orienting the grooved end of a reinforced concrete pipe at the inlet provides similar performance improvements as providing a bevel. Premanufactured end sections are available for corrugated steel and concrete pipe to improve inlet performance to be similar to a headwall with a bevel.

**ADOT Standard Inlets**

ADOT standard details include bevels which should be accounted for in the culvert analysis. Box culvert inlets used on ADOT projects include a 1:1 bevel. Pipe culvert inlet headwalls for pipes that are 48-inches and larger include a 1:1 bevel. Reinforced concrete end sections can be used for concrete pipes from 24 to 42-inches in diameter. Corrugated metal end sections can be used for corrugated metal pipes from 18 to 42-inches in diameter.

**4.2.1.4 Culvert Barrel Losses**

The roughness, length, and slope of the culvert barrel impact the culvert performance in outlet control conditions. The headloss through the barrel is included in the outlet control headwater computation. Table B.1 in HDS 5 presents Manning’s n-values for commonly used culvert materials.

**4.2.1.5 Culvert Outlet Analysis**

Important parameters at the culvert outlet include the tailwater depth and outlet velocity. The outlet velocity is important in determining the potential for scour at the outlet as the flow...
transitions to the natural downstream flow condition. The computed outlet velocity will be used in the scour hole and energy dissipator designs discussed in Chapter 5.

Tailwater Conditions

Determination of the tailwater conditions is important for determining the tailwater elevation used in culvert design calculations and to determine the natural stream velocity downstream of the culvert. Tailwater is defined as the depth of water measured from the invert of the culvert to the water surface elevation at a location just downstream of the outlet. As a key component to culvert design, it is important to correctly evaluate the tailwater hydraulics. This is normally done by computing a rating table of discharge versus water surface elevation (or depth as measured above the invert of the culvert outlet) for the design location, with sufficient data points to encompass all storm events being analyzed. If only a single discharge is being evaluated, then only one tailwater elevation is required.

Single-Event Tailwater Considerations

When evaluating tailwater for a single event the following should be considered:

- If an upstream culvert outlet is located near the inlet of a downstream culvert, such as an open median on a divided highway, the headwater elevation of the downstream culvert may define the tailwater depth for the upstream culvert.
- For culverts which discharge to an open channel, the tailwater may be equal to the normal depth of flow in that channel. Normal depth may be calculated using Manning’s equation.
- For culverts that discharge to an open channel with variable roughness, slope and geometry, or if there is a control structure located downstream of the culvert, the tailwater may have to be determined by performing a step-backwater calculation using a program like HEC-RAS. In these cases, it may be more efficient to model the entire culvert reach.

Confluence Tailwater Considerations

Estimation of tailwater conditions for culverts located at or near confluences with major rivers, or that discharge to a lake, pond, or other major water body, should consider the following:

- Use of the high-water elevation for the receiving water that has the same frequency as the culvert design flood, is appropriate only if events are known to occur concurrently on both watercourses (the watersheds are statistically dependent). Otherwise, care must be taken to evaluate an appropriate storm frequency for establishing the water surface elevation of the receiving water that is assumed to be coincident with the culvert discharge. Assuming that both the culvert watershed and the statistically independent receiving water body are at peak simultaneously is probably overly conservative due to timing issues. Statistically independent watersheds normally occur when a tributary watershed is significantly smaller than the watershed for the primary or receiving
watercourse. Although tributary, the smaller watershed generally does not have a statistical significance on the peak discharges within the receiving river or water body.

- If the design flood events for the two watersheds being analyzed are statistically independent, then the tailwater (and subsequent culvert hydraulics) should be evaluated for two conditions:
  - The design peak flow of the culvert is concurrent with a 10-year peak flow in the primary watercourse, or
  - The 10-year peak flow in the culvert is concurrent with the design peak flow in the main watercourse.

Outlet Velocity

For inlet control, the outlet velocity should be based on the actual flow depth at the culvert outlet. If the HY-8 computer program is used, this velocity will be computed by the program and included in the program output. For outlet control, the outlet velocity is based on the flow area at critical depth, the tailwater depth, or full flow depending on the tailwater conditions:

- Critical depth is used when the tailwater is less than critical depth.
- Tailwater depth is used when tailwater is greater than critical depth but below the top of the barrel.
- Total barrel area is used when the tailwater exceeds the top of the barrel.

Outlet Protection

The type of outlet protection required is based on the ratio of culvert outlet velocity to natural stream velocity as follows:

- No protection is generally required in the natural stream if the outlet velocity is less than 1.5 times the natural stream velocity.
- Loose riprap is generally sufficient for ratios between 1.5 and 2.0 with an outlet velocity less than 10 fps.
- Wire-tied rock riprap should generally be used where the ratio is 1.5 to 2.5 with an outlet velocity between 10 and 15 fps.
- Energy dissipators are required when the ratio between outlet and natural stream velocities is greater than 2.5 or the outlet velocity is greater than 15 fps.

4.2.2 Computer Applications

The FHWA HY-8 computer program (HY-8, 2014) has been developed to automate the culvert design process using the procedures in HDS 5, Hydraulic Design of Highway Culverts (HDS 5, 2012) and HEC 14, Hydraulic Design of Energy Dissipators for Culverts and Channels (HEC 14, 2006). The program is public domain and is available for free download from the following website:

http://www.fhwa.dot.gov/engineering/hydraulics/software/hy8/

The culvert and energy dissipator design for an entire roadway design project can be evaluated within a single HY-8 computer data file (.hy8). The project can consist of an unlimited number of
discrete culvert crossings and each crossing can have up to 6 different culverts. Each of the 6 available culverts may have multiple barrels. For example, a 3 barrel pipe culvert crossing where all three barrels have the same properties is one of the 6 available culverts for analyzing that crossing. A general process for building the .hy8 file is:

1. Prepare a plan view graphic of the entire project in a bitmap (.bmp) format. It is good to place a symbol at each culvert location to be included in the project. This way the culverts can be located in their correct geographic location in the HY-8 map viewer.

2. Create a new HY-8 project and import the bitmap graphic into the map viewer using the “Open Image” button within the map tools toolbar.

3. Create each crossing by selecting the “Add Crossing” button within the map tools toolbar, then clicking on the symbol that was created in the project graphic at the crossing location. The crossing data screen will open.

4. Enter the culvert crossing data for that crossing and click “OK” to save the data. A unique name can be created for each crossing and for each culvert within the crossing.

5. Enter the crossing data in a logical, sequential order so that the listing of crossings in the Project Explorer is as desired.

6. An appropriate report should be generated from HY-8 to be included in the culvert calculations appendix of the Drainage Report. The electronic .hy8 file should be included in the project data submittal.

4.3 CULVERT DESIGN

The methodology for designing culverts, including all design equations, nomographs, charts, design examples and calculations, and descriptions of the various hydraulic parameters and characteristics of culvert flow are summarized in HDS 5, and will not be repeated herein. The following sections provide ADOT specific input, as appropriate or needed, to expand or clarify culvert design procedures and parameters for Arizona.

4.3.1 Site Assessment

An important first step in culvert design is a careful assessment of the site conditions where the culvert would be installed. Important conditions to consider include culvert overtopping controls, stream characteristics and soil properties. The site conditions will influence the location and layout of the culvert crossing.

4.3.1.1 Location and Layout

For new or existing project alignments, candidate cross drainage locations should first be identified to establish potential sites that may require culvert installation. These locations should include natural or man-made watercourses, and may include culvert crossings for known proposed channels as well as existing culverts.
All cross drainage locations should be initially identified to present a complete picture of potential culvert locations for the project. This is accomplished by plotting the proposed roadway alignment over topographic mapping for the project area and noting the watercourse intersects. For projects involving modification or expansion of existing roadways, existing culverts are also to be plotted and noted regarding type, size, and general condition.

ADOT preference is for all cross drainage to be passed in a manner that most closely represents the pre-project condition flow patterns whenever possible and practical. Combining flows from small watersheds to save on culverts is discouraged.

In some cases, flooding can occur as broad areas of sheet flow with no well-defined watercourse or incised conveyance. Care must be exercised to identify these areas early, as they may require special consideration regarding interception, conveyance, and release at highway crossing locations.

Culvert geometry design reflects consideration of the natural topography, the proposed roadway horizontal alignment, improvement width (lanes, medians, shoulders, rights of way, etc.), vertical profile, as well as culvert skew, length, slope, and barrel configuration. Many of these design aspects are discussed in detail in HDS 5. The following are ADOT specific guidance that should be considered by the design engineer when designing culverts for Arizona highways:

**Culvert Length and Slope**

The culvert length and slope should be chosen to approximate existing topography, and to the degree practicable: 1) the culvert invert should be aligned with the channel bottom at both the inlet and outlet locations; 2) the culvert location in both plan and profile should be investigated to minimize the potential for sediment buildup in culvert barrels, and 3) the culvert inlet should be selected to match the geometry of the roadway embankment. Culvert layout considerations and recommended orientation are illustrated on Figure 4–4.

**Multiple Barrels**

The total span width of multiple barrel culverts should fit within the main channel top width whenever possible to avoid conveyance loss through sediment deposition in some of the barrels. The main channel is the primary conveyance area of a watercourse wherein flood discharges of frequent events (2-5 year) are normally conveyed. The main channel can usually be identified by an incised channel with clearly defined banks. In ephemeral washes, the bottom of the main channel is usually comprised of sand or sandy gravel. Multiple barrel culverts should be avoided where the approach flow has a high velocity, and particularly in a supercritical flow regime, as these sites require either a single barrel or special inlet treatment to avoid adverse effects from unintended hydraulic jumps forming in the barrel or right at the throat.
4.3.1.2 Allowable Headwater Conditions

Allowable headwater is the depth of water that can be ponded at the upstream end of the culvert during the design flood. Determining the allowable headwater depth or elevation is subject to the design policies set forth in DDM Volume 1 - Policy & Guidelines. Determination of allowable
headwater for typical culvert site conditions is illustrated on Figure 4–5. Design considerations to assigning the allowable headwater depth for a given culvert include:

- The headwater should be a minimum of 3-inches below the edge of pavement (Case 1)
- Check to ensure that the depth assigned is non-damaging to upstream property, and especially if existing buildings or habitable structures are present. (Case 2)
- Elevations that allow the unintended diversion of flows around the culvert and into another watercourse should be avoided. (Case 3)
- In a system where culverts are present in series, the allowable headwater for the downstream culvert should not create a backwater that would adversely affect the performance of upstream culverts, unless the impact is accounted for in the upstream culvert design.

**Overtopping Analysis Considerations**

For discharges in excess of the design discharge, an overtopping analysis may be required to evaluate the impacts of the 100-year or larger discharge. In cases where overtopping is modeled, the design engineer should give careful consideration to identifying the profile used to model the overtopping flows. When considering a roadway, the obstructing elements that will ultimately define the overtopping profile must be carefully considered to ensure a proper hydraulic analysis is performed to develop the overtopping rating curve. For instance, the roadway at a certain culvert crossing may include:

- A median barrier, which would be the controlling elevation for an overtopping analysis.
- The high side curb may control the overtopping profile if the culvert is located at a superelevated section of the roadway.
- The roadside ditch elevations on the inlet side of the road should also be carefully evaluated with respect to the overtopping profile, as there may be instances where the ditch will provide significant diversion of flow away from the roadway overtopping discharge.

In fact, the roadside ditch may dictate the overland flow profile and become the primary flow path for flows exceeding the design culvert capacity. Collectively, these conditions may have a significant impact on the overtopping hydraulics of the roadway.

When evaluating overtopping, care must also be exercised to ensure that the proper hydraulic equations are used. For instance, a simple roadway overtopping of a low spot on a paved roadway would typically be evaluated as a broad-crested weir. Alternatively, flow over the top of a jersey barrier might better be approximated using a sharp-crested weir analysis.
4.3.1.3 Soil Properties

As part of the site assessment at the beginning of the project, the geotechnical scope of work should include sampling and testing to provide pertinent soil properties to support the drainage design. Important properties may include streambed samples for sediment transport or scour.
protection analysis and design as well as properties for culvert material selection and strength design, such as sulfate levels, pH and resistivity of the soil.

ADOT Materials Group is responsible for determining the site conditions that impact a pipe's service life. The roadway designer should contact ADOT Materials Group early in the design process to coordinate the locations of all potential culvert locations to avoid delays in obtaining soil test results.

4.3.1.4 Stream Characteristics

As part of the overall site assessment for a culvert, the condition of the existing stream should be carefully assessed to identify potential stream instabilities that may impact the culvert performance or may be exacerbated by construction of the culvert if not mitigated. Typical indicators of stream instability include very steep cut banks with obvious signs of recent movement and erosion, large deposits of sand, gravel, cobble, and boulder mixtures within the stream bed indicating significant bed-load transport, headcuts progressing upstream towards the potential culvert location, root mass exposure of old-growth trees and especially saguaro cacti, significant activity in the watershed (grazing, development, sand and gravel operations, etc.), and/or recent wildfire activity.

4.3.2 Culvert Selection

Selection of the appropriate culvert type is an essential element to the design process. In many cases, the type and material selection may involve an iterative solution to obtain the most effective and cost efficient design.

When considering culvert types, the design engineer should generally consider the site elements previously discussed, as well as environmental impacts (changes in flow velocity, distribution and alignment), risk, and cost. The selected configuration should best integrate engineering and economic considerations, and regulatory requirements, such as:

- Construction and maintenance costs,
- Risk of failure or property damage,
- Traffic safety,
- Environmental or regulatory requirements,
- Legal considerations, and
- Land use requirements.

4.3.2.1 Culvert Size and Shape

The culvert size and shape selected is dependent on engineering and economic criteria related to site conditions. The available sizes and shapes for use on ADOT projects are contained within the _Pipe Selection Guidelines and Procedures_ (ADOT, 1996) and the Bridge Group Structure Detail (SD) Drawings. Other considerations regarding culvert size and shape include:
A large differential of flow line elevation from the upstream to downstream culvert face, the presence of rock outcrop, or culvert extensions may require or result in a broken-back culvert profile. Use of broken-back culverts requires the approval of the Chief Drainage Engineer.

Land-use or maintenance requirements (e.g., need for an equipment or cattle pass) may dictate a larger or different barrel geometry than required for hydraulic considerations. See DDM Volume 1 - Policy & Guidelines for further guidance on the selection of these types of structures.

Use of arch or oval shapes is only allowed if required by hydraulic limitations, site characteristics, structural criteria or regulatory requirements.

### 4.3.2.2 Culvert Material

The selection of culvert materials is based on ADOT’s *Pipe Selection Guidelines and Procedures* (ADOT, 1996). Factors generally influencing the selection of pipe material include durability, soil corrosion potential, height of fill cover, life-cycle cost, difficulty of construction and traffic delay, hydraulic roughness, bedding conditions, and water-tightness requirements.

The most widely used types of prefabricated culverts are Corrugated Steel Pipe (CSP) and Reinforced Concrete Pipe (RCP). CSP sections are connected with bands that encircle the joints and the RCP sections have tongue and groove joints typically fitted with a rubber gasket to prevent leakage.

Unless site conditions require the installation of a particular type of pipe, alternate pipe materials are allowed as long as all requirements given in the plans and specifications are met. This allows the contractor to supply the most economical culvert that meets the project requirements. Allowable pipe materials are specified in the project construction plans on the “New Pipe Summary” sheet. The allowable pipe options are those which meet the minimum service life requirements. Currently, the following pipe materials are included for potential use on the New Pipe Summary sheet:

- Corrugated Steel Pipe (CSP)
- Corrugated Aluminum Pipe (CAP)
- Reinforced Concrete Pipe (RCP)
- Non-Reinforced Concrete Pipe (NRCP)
- Non-Reinforced Cast in Place Concrete Pipe (NRCIPCP)
- Corrugated High-Density Polyethylene Plastic Pipe (CHDPEPP)

### 4.3.2.3 End Treatments

Scour potential at culvert inlets and outlets is increased due to the concentration, and often, acceleration of flows in the boundary regions of the culvert. Inlet scour can occur with culverts that have steep barrels and a tendency to accelerate the flows as they contract and enter the barrel. The most effective method of scour protection for culvert inlets is to construct a headwall with a sufficient toe-down of the wall footer. For extreme cases where flows are accelerating
through a transition zone from the wash into the culvert, riprap or gabion mattress may be required to stabilize the transition.

Scour at culvert outlets is the subject of Chapter 5 which discusses computation of the downstream scour hole and the selection and design of energy dissipators to protect against scour at the culvert outlet.

### 4.3.3 Special Considerations

#### 4.3.3.1 Culverts as Bridges

Per DDM Volume 1 - Policy & Guidelines, all natural or earthen bottom structures such as arch culverts or structural plate culverts without bottom lining, shall be designed and evaluated for scour as bridges (See Chapter 6 of this manual).

Box culverts with a total span of twenty feet or greater are also classified as bridges and are required to meet operation and maintenance requirements under the National Bridge Inventory System. Even though they are classified as bridges for administrative purposes box culverts are still analyzed and designed using culvert methodologies. The total span includes the intermediate supporting walls of multi-barrel box culverts and is measured from the inside face to inside face of the end barrels.

#### 4.3.3.2 Right-of-Way Requirements

In culvert locations that require energy dissipators, or other special inlet or outlet structures, the standard right of way width obtained for the highway may not be adequate. Energy dissipator designs that are truncated or modified to fit within the standard right of way are often ineffective and become long term maintenance problems that accumulate high maintenance costs over the service life of the culvert installation. In these situations it is prudent and cost effective to identify and obtain the required additional right of way to allow construction of suitable outlet protection structures.

Ponding areas upstream of culverts may also require additional right-of-way or easements. Additional right of way requirements for culverts and appurtenant features should be identified early in the project so that special right of way requirements can be included in the overall right of way acquisition process. Right of way requirements can be identified based on preliminary design and initial layout estimates with adequate additional area to accommodate future design refinements. An example of a special right of way requirement is illustrated on Figure 4–6.
4.3.3.3 Animal Passage

In addition to conveyance of stormwater runoff, culverts may also be used for non-hydraulic purposes such as passage of small roads under the highway and animal passage. These uses may be included in the project to meet the requirements of other agencies or jurisdictions. Requirements for these non-hydraulic uses will typically be provided by the requesting agency.

Where animal passage is required at a multi barrel location only one barrel needs to meet the passage size requirements. It may be appropriate in some instances to construct a separate; non-hydraulic structure for animal passage.

4.3.3.4 Junctions

As a rule, the joining of two culverts at an enclosed junction should be avoided due to maintenance access limitations and additional structure costs. For most ADOT projects it is preferable to outlet the two culverts at a common headwall rather than constructing a junction structure. In situations where the combining of two culverts is unavoidable, prior approval will be required from the ADOT Drainage Section. Design of junction structures should include the consideration of appropriate head losses and should account for the effects of momentum in the analysis of the junction structure. Section 5.2.4 of HDS 5 (HDS 5, 2012) addresses the design of...
junctions, with reference to HEC 22, *Urban Drainage Design Manual* (HEC 22, 2009) for detailed loss calculations. Provision should be made during design for maintenance access to any junction structure.

### 4.3.3.5 Median Drainage

Median drainage is sometimes vertically dropped into a cross drainage culvert by including a precast or prefabricated tee section or by providing an opening while forming a cast-in-place structure. The effect of this flow introduction is usually minimal as long as the culvert capacity is significantly greater than the flow introduced by the median structure. As with junctions, it may be preferable to carry those median flows in a separate conduit and outlet to a shared headwall. Otherwise, the culvert hydraulics should account for the losses due to the entry of the median flows plus any increase in design discharge downstream of the median inlet. Section 5.2.7 of HDS 5 addresses median drainage and provides design guidelines for inlet and outlet controlled culverts.

### 4.3.3.6 Drop Inlets

Drop inlets can be an effective way to introduce flows into a culvert at locations with limited right of way or extremely steep cut slopes. Drawing C-15.75 of the ADOT Standard Drawings shows a drop inlet for use on ADOT projects. Use of drop inlets should be limited to areas with small drainage areas to avoid construction of overly large inlet structures. When using a drop inlet structure, the design engineer should be sure to calculate the capacity of the drop inlet itself, to make sure there is sufficient opening or weir perimeter, to allow the flows to enter the structure itself. Often the available headwater for the culvert provides a culvert capacity that exceeds the drop inlet capacity. Careful design of the inlet structure will ensure a compatible system. Provision for maintenance access should be provided to allow removal of sediment and debris from within the drop inlet structure.

### 4.3.4 Maintenance Considerations

Debris control structures shall be designed using HEC 9, *Debris Control Structures – Evaluation and Countermeasures*, (HEC 9, 2005). Assessment for the need of debris control shall consider situations where:

- Experience or physical evidence indicates the watercourse will transport a heavy volume of controllable debris.
- Culverts are to be located in mountainous or steep regions.
- Culverts are to be constructed under high fills.
- Access to the culvert barrel entrance is restricted during a flood event, but sufficiently available to clean out the debris-control device on a routine maintenance basis during dry periods.
4.3.5 Culvert Design for New Construction Using HY-8 (Example)

An example problem is contained in Appendix 4A to illustrate the use of HY-8 for culvert design for new construction. The example includes the following computations:

Size a 6-foot high box culvert for a new ADOT roadway design based on the data provided in Appendix 4A using the HY-8 computer program. Report the following with the solution:

1. Design Discharge
2. Allowable Headwater depth (elevation)
3. Culvert size, slope, length
4. Overtopping discharge at Q_{100}
5. Culvert outlet velocity and downstream channel velocity for the design discharge
6. The type of erosion treatment that will be required at the culvert outlet

4.4 CULVERT EXTENSIONS AND RETROFITS

As Arizona highways age, ADOT is continually faced with the need for repair or modification of roadway elements. Population growth and increased traffic counts also create demand for additional lanes on many ADOT roadways. With each repair, replacement or widening project, ADOT must evaluate the existing culverts for potential replacement or modification to fit the new roadway limits. Considerations for culvert extensions and retrofits are contained in the following sections.

4.4.1 Culvert Extensions

Culvert extensions require a field inspection and a hydraulic analysis to determine if they will still perform as originally intended upon extension and would not cause any new drainage problems to the travelling public or adjacent property. ADOT’s highways programmed for widening and retrofit projects are typically those that were constructed 30 to 60 years ago and so it is expected that design information such as flow values may not be available for use, or may not be good enough because of changes in hydrology methods and hydraulic analysis procedures over time. ADOT has prioritized the choice to be made when faced with a culvert extension situation, but consideration should also be given on a case-by-case basis commensurate with the historical performance of the culvert in place. ADOT’s policy calls for a drainage report to be prepared for any projects involving culvert extensions and pipe lining so that a historical record is kept of all changes made to the culvert.

4.4.1.1 Hydrology

Maintenance records and previous project reports provide the background information needed for evaluating the extent of treatment necessary at a culvert during extension. A new hydrology study is typically not required for every culvert extension unless a history of overtopping has been reported, or some other drainage issues are present that need to be corrected during the
extension. Existing flow values from previous projects should be the first source of information to use before an entirely new hydrology study is conducted.

### 4.4.1.2 Capacity Check Analysis

If there are no historical overtopping issues and no other drainage repair is needed, a capacity check should be performed to ascertain that the modification will not leave the culvert with diminished performance. The following steps may be followed:

1. No hydrologic analysis is performed.
2. Evaluate the capacity of the existing culvert using the HY-8 computer program.
3. Determine the discharge that the existing culvert will pass based upon a headwater elevation 3-inches below the existing edge of pavement.
4. Extend the culvert as necessary for the roadway widening.
5. Utilizing the same methodology and discharge, determine the new headwater elevation for the culvert extensions. The new Allowable Headwater Elevation should be the elevation which is 3-inches below the new edge of pavement. Most culverts on Arizona highways are expected to be inlet controlled due to tailwater conditions and short lengths. As such the headwater elevations are expected to increase by an amount equal to the culvert slope times the length of the upstream extension, neglecting any upstream flow breakouts. Extensions that are on the downstream side of the culvert only, may not have impact on the upstream side for a culvert performing in inlet control.
6. For higher roadway embankments where the possibility of flow overtopping the edge of road is remote, HW/D (Headwater/Culvert Height ratio) should be limited to 1.5 for checking whether there is additional impact to adjacent property due to the culvert extension.
7. The culverts will also be evaluated for capacity reduction based upon the same headwater elevation established in step 3 above. The change in culvert capacity will be determined.
8. If there is capacity reduction leading to a higher headwater relative to the new edge of roadway elevation, measures should be taken to improve the inlet configuration of the culvert to enhance performance. The hydraulic analysis for improved inlets is covered in HDS 5 manual and analysis can be performed with HY-8.

### 4.4.2 Broken-Back Culverts

Use of broken back culverts on ADOT projects is very limited and installations are usually the result of special circumstances such as the presence of rock outcrop in the culvert profile or vertical deflections required due to roadway widening. Broken back culvert configurations may also provide solutions to avoiding utility conflicts and can even be used as a means of dissipating energy internal to the culvert in steep hillside environments.
Broken back culverts require the approval of the ADOT Drainage Section and should be designed and hydraulically evaluated per HDS 5, Section 5.6 (HDS 5, 2012).

### 4.4.3 Horizontal Bends

Bends or changes in horizontal direction of culverts can be acceptable for ADOT projects as long as the deflections are gradual and the losses associated with the change in direction are accounted for in the culvert hydraulics. Friction losses due to bends can be calculated as a function of the velocity head in the barrel using coefficients presented in Section 5.2.3 of HDS 5 (HDS 5, 2012). Further design considerations are discussed in the HEC 22 Urban Drainage Design Manual (HEC 22, 2009).

### 4.4.4 Pipe Liners

FHWA provides guidance for selection and design of pipe liners in Culvert Pipe Liner Guide and Specifications, (FHWA, 2005). The guide identifies alternative pipe lining approaches and materials and provides a multi-criteria decision analysis tool to aid in identifying the best method for a given culvert application. The following five approaches to lining are included in the guide:

- Sliplining
- Close-fit Lining
- Spirally Wound Lining
- Cured-in-place pipe Lining
- Spray-on lining
Limitations on the application of each lining alternative are tabulated in Table 7–4.

### Table 4–1  Culvert Lining Alternative Limitations (FHWA, 2005)

<table>
<thead>
<tr>
<th>Input</th>
<th>Alternative</th>
<th>Slip lining</th>
<th>Close-fit lining</th>
<th>Spirally wound lining</th>
<th>Cured-in-place lining</th>
<th>Spray-on lining</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Segmental Method</td>
<td>Continuous Method</td>
<td>Deformed/Reformed Method</td>
<td>Fold and Form Method</td>
<td>Inversion Method</td>
</tr>
<tr>
<td>Applicable Length</td>
<td>&lt; 985 ft</td>
<td>&lt; 985 ft</td>
<td>&lt; 2,625 ft</td>
<td>&lt; 689 ft</td>
<td>&lt; 985 ft</td>
<td>&lt; 2,955 ft</td>
</tr>
<tr>
<td>Diameter Limitation</td>
<td>3 - 48 in</td>
<td>4 - 48 in</td>
<td>4 - 16 in</td>
<td>4 - 24 in</td>
<td>4 - 48 in</td>
<td>4 - 48 in</td>
</tr>
<tr>
<td>Diameter Change/Discontinuity</td>
<td>Severe Prohibits¹</td>
<td>Severe Prohibits¹</td>
<td>Allowable</td>
<td>Allowable</td>
<td>Allowable</td>
<td>Allowable</td>
</tr>
<tr>
<td>Structural Integrity</td>
<td>RI²</td>
<td>RI²</td>
<td>NA¹</td>
<td>NA¹</td>
<td>RI²</td>
<td>RI²</td>
</tr>
</tbody>
</table>

¹Prohibits - Existence of prohibits use,
²RI - Restores structural integrity,
³NA - Not applicable to structurally deteriorated culverts,
⁴DE - Does not enhance structural integrity

#### 4.4.5 Example 2

An example problem is contained in Appendix 4B to illustrate the use of HY-8 for culvert design for an extension of an existing culvert. The example includes the following computations:

Extend the 6-foot high box culvert from Example 4-1 for a roadway widening design based on the data provided in Appendix 4B using the HY-8 computer program. Report the following with the solution:

1. Allowable Headwater depth (elevation)
2. Headwater depth at design discharge
3. Overtopping discharge at $Q_{100}$
4.5 REFERENCES


APPENDIX 4A
CULVERT DESIGN
EXAMPLE 4-1
Example No. 4-1  Culvert Design for New Construction Using Computer Program HY-8

Problem:
Size a 6-foot high box culvert for a new ADOT roadway design based on the data provided below using the HY-8 computer program. Report the following with the solution:

- Design Discharge
- Allowable Headwater depth (elevation)
- Culvert size, slope, length
- Overtopping discharge at $Q_{100}$
- Culvert outlet velocity and downstream channel velocity for the design discharge
- The type of erosion treatment that will be required at the culvert outlet

Given:
Roadway Drainage Frequency Class = 2
Discharge Frequency Data:

<table>
<thead>
<tr>
<th>Return Period (years)</th>
<th>Discharge (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>380</td>
</tr>
<tr>
<td>25</td>
<td>460</td>
</tr>
<tr>
<td>50</td>
<td>600</td>
</tr>
<tr>
<td>100</td>
<td>750</td>
</tr>
</tbody>
</table>

Solution:
1. Determine design criteria
   a. The Design Frequency for new construction on Roadway Drainage Frequency Class 2 = 50-year. Therefore, the design discharge is 600 cfs.
   b. The allowable headwater is the lower of:
      - 3-inches below the pavement surface \((188.23 - (3/12)) = 187.98\), or
      - 1.5 times the culvert height. \((178.60 + (1.5 * 6)) = 187.60\).
2. Using the HY-8 computer program, complete the “Crossing Data” Form:
   a. Complete the Discharge Data by entering the Minimum, Design, and Maximum discharges from the Discharge Frequency Data table provided. \(0,600,750 \text{ cfs}\).
   b. Complete the Tailwater Data by entering the Irregular Channel data provided.
   c. Complete the Roadway Data by entering the Irregular roadway profile data provided.

   ![Tailwater/Channel Cross Section](image)
   ![Roadway profile](image)
d. Complete the Site Data by entering the Embankment Toe Data from the roadway embankment section provided. HY-8 will then compute the length and invert elevations of the culvert to fit the embankment.

![Diagram of embankment profile]

e. With the foregoing data entered, various culvert design configurations may be evaluated using the Culvert Data section of the Crossing Data window. Use standard ADOT culvert sizes. Begin by selecting a 6 x 6 foot concrete box culvert with a 1:1 bevel and 45° wingwalls.

3. With the Crossing Data form completed, select “Analyze Crossing” and review the results presented in the “Summary of Flows at Crossing” window. From the Crossing Summary Table Display note that at the design discharge of 600 cfs the headwater elevation is 189.07 ft. The culvert is passing 531 cfs and 68 cfs is overtopping the road.

![Summary of Flows at Crossing - Example 4-1]
4. Looking at the Geometry section of the window, the 6 x 6 foot concrete box culvert slope and length are shown:

```
<table>
<thead>
<tr>
<th>Geometry</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inlet Elevation:</td>
</tr>
<tr>
<td>Outlet Elevation:</td>
</tr>
<tr>
<td>Culvert Length:</td>
</tr>
<tr>
<td>Culvert Slope:</td>
</tr>
<tr>
<td>Inlet Crest:</td>
</tr>
<tr>
<td>Inlet Throat:</td>
</tr>
</tbody>
</table>
```

Note that the length, inlet and outlet elevations are computed to fit within the specified embankment section. With the shortened culvert and lowered inlet elevation, the allowable headwater criteria for 1.5 times the culvert height needs to be recomputed. The revised allowable headwater elevation is lowered by 1.28 feet from the allowable headwater elevation computed using the embankment toe data \((177.32 + (1.5 \times 6 \text{ ft}) = 186.32)\).

5. Since the allowable headwater criteria is not met for the design discharge, a new culvert configuration is needed.

6. Go back to the Crossing Data window (“Edit Input Data” button) and try a larger box culvert size.

   a. The next larger ADOT standard size culvert with a 6-foot height is an 8 x 6 foot box culvert.

   b. Modify the Span in the Culvert Data section from 6 to 8-feet and select “Analyze Crossing”.

   c. Reviewing the Crossing Summary Table, note that at the design discharge of 600 cfs the Headwater Elevation is 187.09 and there is no roadway overtopping. The headwater elevation is not below the revised allowable headwater elevation of 186.32 based on the computed culvert invert data.

   d. At the 100-year discharge of 750 cfs there is 45 cfs flowing over the road.

7. Since the allowable headwater criteria is still not met for the design discharge, a new culvert configuration is needed.

8. Go back to the Crossing Data window (“Edit Input Data” button) and try a larger box culvert size.

   a. The next larger ADOT standard size culvert with a 6-foot height is a 10 x 6 foot box culvert.

   b. Modify the Span in the Culvert Data section from 8 to 10 feet and select “Analyze Crossing”.

   c. Reviewing the Crossing Summary Table, note that at the design discharge of 600 cfs the Headwater Elevation is 185.19 and there is no roadway overtopping. The headwater elevation is below the revised allowable headwater elevation of 186.32.
based on the computed culvert invert data. The design meets the allowable headwater criteria.

d. At the 100-year discharge of 750 cfs there is no roadway overtopping.

9. Once the design is finalized the Site Data should be updated to replace embankment toe data with culvert invert data so the design computations will match the construction plans.

10. The Culvert Summary Table shows that for the 600 cfs design discharge the culvert outlet velocity is 24 ft/s and the downstream (Tailwater) channel velocity is 13 ft/s. Per ADOT criteria, since the culvert outlet velocity is over 15 fps; an energy dissipator structure will be required.
APPENDIX 4B
CULVERT DESIGN
EXAMPLE 4-2
Example No. 4-2  Culvert Extension Design for Road Widening Project Using Computer Program HY-8

Problem:

Extend the 6-foot high box culvert from Culvert Design Example 4-1 for a roadway widening design based on the data provided below using the HY-8 computer program. Report the following with the solution:

- Allowable Headwater depth (elevation).
- Headwater depth at design discharge.
- Overtopping discharge at Q_{100}.

Given:

An auxiliary lane is being added on the upstream side of the culvert such that the culvert must be extended 12-feet in length.

Revised roadway embankment:

![Diagram of revised roadway embankment with Sta. 9+63 Edge of Pavement Elev. 187.99, Sta. 9+42.60 Elev. 177.56 So=0.02 ft/ft, Sta. 9+54.60 Elev. 177.32, Sta. 10+00 Elev. 175.05 So=0.05 ft/ft, Sta. 10+58.2 Elev. 172.24, 50' between stations, and Crest Elev. 188.73.]
Solution:

1. The allowable headwater is the lower of:
   a. 3-inches below the pavement surface \(188.23 - (12*0.02) - (3/12) = 187.74\), or
   b. 1.5 times the culvert height. \(177.56 + (1.5 * 6 \text{ ft}) = 186.56\)

2. Using the HY-8 file from Culvert Design Example 4-1, modify the “Crossing Data” form to add the culvert extension. Since the culvert extension is at a different slope than the existing culvert, the culvert will be analyzed as a broken-back culvert.
   a. Using the drop down menu for Culvert Type in the Culvert Data section, select “Single Broken-back”.
   b. In the Site Data section input the elevation and station for the inlet, break, and outlet locations. The break location will be the inlet from Culvert Design Example 4-1. In the current version of HY-8, the inlet station should be set at zero and the other stations computed as distance from the inlet station for the output graphics to display correctly. The input data is shown below:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Name</td>
<td>Example 4-2</td>
<td></td>
</tr>
<tr>
<td>Shape</td>
<td>Concrete Box</td>
<td></td>
</tr>
<tr>
<td>Upper Section Material</td>
<td>Concrete</td>
<td></td>
</tr>
<tr>
<td>Lower Section Material</td>
<td>Concrete</td>
<td></td>
</tr>
<tr>
<td>Span</td>
<td>10.00</td>
<td>ft</td>
</tr>
<tr>
<td>Rise</td>
<td>6.00</td>
<td>ft</td>
</tr>
<tr>
<td>Upper Section Manning's n</td>
<td>0.0120</td>
<td></td>
</tr>
<tr>
<td>Lower Section Manning's n</td>
<td>0.0120</td>
<td></td>
</tr>
<tr>
<td>Culvert Type</td>
<td>Single Broken-back</td>
<td></td>
</tr>
<tr>
<td>Inlet Configuration</td>
<td>1:1 Bevel (45' flare) Wingwall</td>
<td></td>
</tr>
<tr>
<td>Inlet Depression?</td>
<td>No</td>
<td></td>
</tr>
<tr>
<td>Inlet Station</td>
<td>0.00</td>
<td>ft</td>
</tr>
<tr>
<td>Inlet Elevation</td>
<td>177.56</td>
<td>ft</td>
</tr>
<tr>
<td>Break Station</td>
<td>12.00</td>
<td>ft</td>
</tr>
<tr>
<td>Break Elevation</td>
<td>177.32</td>
<td>ft</td>
</tr>
<tr>
<td>Outlet Station</td>
<td>113.60</td>
<td>ft</td>
</tr>
<tr>
<td>Outlet Elevation</td>
<td>172.24</td>
<td>ft</td>
</tr>
<tr>
<td>Number of Barrels</td>
<td>1</td>
<td></td>
</tr>
</tbody>
</table>

3. Select “Analyze Crossing” and review the results presented in the “Summary of Flows at Crossing” window. Note that the headwater elevation at the design discharge of 600 cfs is 185.44 which is below the allowable headwater elevation. There is no overtopping during the 100-year discharge.
Chapter 5
ENERGY DISSIPATORS

This chapter contains details on the following:

- Energy dissipator selection and design
- Energy dissipator types and preferences

5.1 INTRODUCTION

The concentration of flow at culverts often results in increased velocities with a corresponding increase in erosion potential. If unchecked, the increased erosion potential can cause damage or failure of structures and the highway. To protect the culvert and adjacent areas, it is often desirable to use an energy dissipator. Energy dissipators are also used in channels to dissipate energy as part of drop structure design.

5.1.1 Design Resources

The design of energy dissipators is described in FHWA’s Hydraulic Engineering Circular No. 14, Hydraulic Design of Energy Dissipators for Culverts and Channels (HEC 14, 2006). HEC 14 includes the applicable charts, equations, and theory needed for energy dissipator analysis and design. It is anticipated that designers will utilize HEC 14 as a primary resource; therefore, equations, procedures, and figures from HEC 14 are not included in this chapter. The procedures contained in HEC 14 have been incorporated into the HY-8 Computer program (HY-8, 2014). These resources are recommended for use on ADOT projects and are summarized below:

- **HY-8 Computer Program** (HY-8, 2014) – HY-8 offers steady-state hydraulic analysis of the energy dissipator types discussed in HEC 14. HY-8 also provides a direct computational interface for energy dissipator design at culvert outlets with culvert structures discussed in Chapter 4.

5.1.2 General Category and Type Definitions

Dissipator type selection for a site must be appropriate to the location and use. The primary focus of this section is energy dissipators associated with culverts, however, many of the methods and dissipator types may be easily adapted for use at the terminus of high velocity channels as well as other applications not covered elsewhere in the Hydraulics Manual.
The suitability of an energy dissipator for a particular application depends on many factors:

- Maintenance requirements
- Availability of R/W
- Long term performance
- Cost
- Availability of Material
- Soil Conditions
- Wildlife access
- Safety

HEC 14 discusses multiple types of energy dissipators, with the majority being some form of structural measure at the culvert outlet. Other types include structural modification of the culvert barrel and naturally developed scour holes. Dissipator types discussed in HEC 14 are listed in Table 5-1 along with their acceptability for application on ADOT projects.

<table>
<thead>
<tr>
<th>Table 5-1</th>
<th>HEC 14 Dissipator Types and Uses</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Chapter in HEC 14</strong></td>
<td><strong>Dissipator Type</strong></td>
</tr>
<tr>
<td>4</td>
<td>Flow Transitions</td>
</tr>
<tr>
<td>5</td>
<td>Natural Scour Hole</td>
</tr>
<tr>
<td>6</td>
<td>Hydraulic Jump</td>
</tr>
<tr>
<td>7</td>
<td>Tumbling Flow</td>
</tr>
<tr>
<td>7</td>
<td>Increased Resistance</td>
</tr>
<tr>
<td>Chapter in HEC 14</td>
<td>Dissipator Type</td>
</tr>
<tr>
<td>------------------</td>
<td>-----------------------------------------</td>
</tr>
<tr>
<td>7</td>
<td>USBR Type IX Baffled Apron</td>
</tr>
<tr>
<td>7</td>
<td>Broken-Back Culvert</td>
</tr>
<tr>
<td>7</td>
<td>Outlet Weir</td>
</tr>
<tr>
<td>7</td>
<td>Outlet Drop/Weir</td>
</tr>
<tr>
<td>8</td>
<td>USBR Type III Stilling Basin</td>
</tr>
<tr>
<td>8</td>
<td>USBR Type IV Stilling Basin</td>
</tr>
<tr>
<td>8</td>
<td>SAF Stilling Basin</td>
</tr>
<tr>
<td>9</td>
<td>CSU Rigid Boundary Basin</td>
</tr>
<tr>
<td>9</td>
<td>Contra Costa Basin</td>
</tr>
<tr>
<td>9</td>
<td>Hook Basin</td>
</tr>
<tr>
<td>9</td>
<td>USBR Type VI Impact Basin</td>
</tr>
<tr>
<td>10</td>
<td>Riprap Basin</td>
</tr>
<tr>
<td>10</td>
<td>Riprap Apron</td>
</tr>
</tbody>
</table>
Table 5–1  HEC 14 Dissipator Types and Uses

<table>
<thead>
<tr>
<th>Chapter in HEC 14</th>
<th>Dissipator Type</th>
<th>Acceptability for ADOT Use</th>
<th>NOTES</th>
</tr>
</thead>
<tbody>
<tr>
<td>11</td>
<td>Straight Drop Structure</td>
<td>Acceptable</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>Box Inlet Drop Structure</td>
<td>Acceptable</td>
<td>Design should ensure adequate maintenance access for removal of sediment.</td>
</tr>
<tr>
<td>12</td>
<td>USACE Stilling Well</td>
<td>Approval Required</td>
<td>Use of this device should only be used in applications that are relatively free of sediment transport.</td>
</tr>
</tbody>
</table>

In this chapter, the terms internal and external are used to indicate the location of the dissipator in relationship to the culvert. An external dissipator is located outside of the culvert, and an internal dissipator is located within the culvert barrel.

Limitations for the ADOT “Acceptable” and “Approval Required” dissipator types listed in Table 5–1 are provided in Table 5–2, which can be used as an aid to identify candidate dissipator types.

Table 5–2  Dissipator Application and Limitations (HEC 14, 2006)

<table>
<thead>
<tr>
<th>Dissipator Location</th>
<th>Dissipator Type</th>
<th>Fr Froude Number(^7) (Fr)</th>
<th>Allowable Debris(^1)</th>
<th>Tailwater (TW)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Internal and/or External</td>
<td>Hydraulic Jump</td>
<td>&gt; 1</td>
<td>H</td>
<td>H</td>
</tr>
<tr>
<td>Internal</td>
<td>Tumbling Flow(^2)</td>
<td>&gt; 1</td>
<td>M</td>
<td>L</td>
</tr>
<tr>
<td>Internal</td>
<td>Increased Resistance(^3)</td>
<td>na</td>
<td>M</td>
<td>L</td>
</tr>
<tr>
<td>External</td>
<td>USBR Type IX Baffled Apron</td>
<td>&lt; 1</td>
<td>M</td>
<td>L</td>
</tr>
<tr>
<td>Internal and/or External</td>
<td>Broken-Back Culvert</td>
<td>&gt; 1</td>
<td>M</td>
<td>L</td>
</tr>
<tr>
<td>Internal and/or External</td>
<td>Outlet Weir</td>
<td>2 to 7</td>
<td>M</td>
<td>L</td>
</tr>
</tbody>
</table>

\(^1\) Allowable Debris: Silt/Sand, Boulder, Floating

\(^2\) Tumbling Flow is used when the Froude Number is greater than 1.

\(^3\) Increased Resistance is used when the Froude Number is less than 1.

\(^7\) Froude Number is calculated using the formula: \(F = \frac{V}{\sqrt{2gD}}\), where \(V\) is the average velocity, \(g\) is the acceleration due to gravity, and \(D\) is the diameter of the culvert.

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### Table 5–2 Dissipator Application and Limitations (HEC 14, 2006)

<table>
<thead>
<tr>
<th>Dissipator Location</th>
<th>Dissipator Type</th>
<th>Froude Number(^7) (Fr)</th>
<th>Allowable Debris(^1)</th>
<th>Tailwater (TW)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Internal and/or External</td>
<td>Outlet Drop/Weir</td>
<td>3.5 to 6</td>
<td>M L M</td>
<td>Not needed</td>
</tr>
<tr>
<td>External</td>
<td>USBR Type III Stilling Basin</td>
<td>4.5 to 17</td>
<td>M L M</td>
<td>Required</td>
</tr>
<tr>
<td>External</td>
<td>USBR Type IV Stilling Basin</td>
<td>2.5 to 4.5</td>
<td>M L M</td>
<td>Required</td>
</tr>
<tr>
<td>External</td>
<td>SAF Stilling Basin</td>
<td>1.7 to 17</td>
<td>M L M</td>
<td>Required</td>
</tr>
<tr>
<td>External</td>
<td>CSU Rigid Boundary Basin</td>
<td>&lt; 3</td>
<td>M L M</td>
<td>Not needed</td>
</tr>
<tr>
<td>External</td>
<td>Contra Costa Basin</td>
<td>&lt; 3</td>
<td>H M M</td>
<td>&lt; 0.5D</td>
</tr>
<tr>
<td>External</td>
<td>USBR Type VI Impact Basin(^4)</td>
<td>na</td>
<td>M L L</td>
<td>Desirable</td>
</tr>
<tr>
<td>External</td>
<td>Riprap Basin</td>
<td>&lt; 3</td>
<td>H H H</td>
<td>Not needed</td>
</tr>
<tr>
<td>External</td>
<td>Riprap Apron(^8)</td>
<td>na</td>
<td>H H H</td>
<td>Not needed</td>
</tr>
<tr>
<td>External</td>
<td>Straight Drop Structure(^5)</td>
<td>&lt; 1</td>
<td>H L M</td>
<td>Required</td>
</tr>
<tr>
<td>External</td>
<td>Box Inlet Drop Structure(^6)</td>
<td>&lt; 1</td>
<td>H L M</td>
<td>Required</td>
</tr>
<tr>
<td>External</td>
<td>USACE Stilling Well</td>
<td>na</td>
<td>M L N</td>
<td>Desirable</td>
</tr>
</tbody>
</table>

---

\(^1\) Debris notes: N = none, L = low, M = moderate, H = heavy
\(^2\) Bed slope must be in the range \(4\% < S_o < 25\%\)
\(^3\) Check headwater for outlet control
\(^4\) Discharge, \(Q < 400\) cfs and Velocity, \(V < 50\) fps
\(^5\) Drop < 15 ft
\(^6\) Drop < 12 ft
\(^7\) At release point from culvert or channel
\(^8\) Culvert rise less than or equal to 60 in
na = not applicable

### 5.1.3 Typical Applications

Energy dissipators are typically used at culvert outlets and at the terminus of an improved channel or chute to reduce velocities to near natural channel levels of the receiving channel to
prevent downstream erosion. The need for an energy dissipator at the culvert outlet is determined by comparison of the culvert outlet velocity with the downstream channel velocity as directed in Section 4.2.1 of the Culverts chapter.

5.1.4 General Design Procedure

The general design procedure for energy dissipators as presented in HEC 14 is illustrated on Figure 5–1. The remaining sections in this chapter are organized around this general design procedure. When used at culvert outlets, energy dissipators can be evaluated and designed in conjunction with the culvert design using the HY-8 computer program.

5.2 DESIGN DATA COLLECTION

5.2.1 Hydrology

The flow frequency used in the design of the energy dissipator should be the same flow frequency that is used for the culvert design. The required flow frequency for culvert design is contained in DDM Volume 1 - Policy & Guidelines.

5.2.2 Inflow Culvert/Channel

The culvert design establishes the outlet flow conditions for the culvert, which in turn are used as the approach flow condition for the energy dissipator. The culvert outlet flow conditions or the channel flow characteristics upstream of the energy dissipator are required for design.

5.2.3 Transition Data

When an apron is included with wingwalls at the culvert outlet, there is a flow transition between the face of the culvert outlet and the downstream apron edge in an abrupt expansion condition. The flow characteristics at the downstream end of the transition are used for the energy dissipator design. When an apron is not provided, the conditions at the face of the culvert outlet may be used for energy dissipator design. Chapter 4 of HEC 14 addresses several types of flow transitions that are commonly used at culvert outlets.

5.2.4 Tailwater Data

The outlet channel downstream from the energy dissipator will dictate the tailwater conditions at the energy dissipator. The level of tailwater will influence the energy dissipator selection and its performance. In addition to the design flow rate, the energy dissipator performance and related tailwater elevation should be evaluated for a range of discharges to ensure adequate
dissipation during frequent low flows. Determination of tailwater depth is discussed in Section 4.2.1 of the Culverts chapter.

5.2.5 Scour/Stability Assessment

Visual observations of conditions that would limit or exacerbate scour should be noted in the field. Existing rock at the culvert outlet should be noted along with soil parameter data including grain size, shear strength, and plasticity index to be used for scour estimates.

5.2.6 Debris Load

Selection of an appropriate dissipator type should include consideration of the allowable debris load as directed in Table 5–2. Assessment of potential for heavy debris or large bed material transport should be made based on field observations to determine if debris loads should be a factor in the design. Debris typically reaching roadway crossings consists of floating debris of varying sizes and transported debris such as sand, gravel, and larger bed material sizes. Due to the extended periods with no rainfall in Arizona, significant amounts of vegetative debris accumulates within the watershed and often becomes transported to the culvert crossing during storm runoff events. This occurs in both rural and urban areas. The best way to determine debris potential is to observe evidence from past flow events. Floating debris potential can be indicated by deposits of vegetative and landscape materials in backwater areas such as overbank areas and ponding areas upstream of obstructions and culverts. Transported debris can be indicated by the size of material on the channel bed and evidence of recent bed movement. Armored channel beds may indicate relative stability, whereas evidence of recent bed change indicates active debris movement. The size of material being transported indicates the size of debris that will ultimately reach the culvert crossing and energy dissipator structure.

5.3 EVALUATE VELOCITIES

The relationship of culvert outlet velocity to natural downstream channel velocity will dictate the requirement for providing an energy dissipator according to criteria in DDM Volume 1 - Policy & Guidelines. The culvert design should consider the potential for maintaining near natural channel velocities to minimize the need for costly downstream energy dissipators. As discussed in Chapter 4, the design of the culvert and energy dissipator should be performed together as a system to identify the most cost-effective combination.

5.4 SCOUR HOLE EVALUATION

Most culverts discharging to a natural watercourse will cause some measure of local scour immediately downstream of the culvert. The magnitude of scour is dependent on many factors including discharge, culvert shape, soil type, armoring capability, duration of flow, culvert slope, culvert height above the bed, and tailwater depth. There are also occasions where drop structures may be located in channels to serve grade control or energy dissipation needs.
5.4.1.1 Culvert Outlet Scour

Chapter 5 of HEC 14 outlines a procedure for estimating scour hole geometry based on soil type, flow data and culvert geometry. Example calculations for both cohesive and non-cohesive soils are provided. When appropriate, scour prediction by this procedure is to be used and is intended to serve together with the maintenance history and site reconnaissance information for determining energy dissipator needs.

5.4.1.2 Channel Drop Scour

Scour below channel drops, such as culvert outlets and grade control structures, is a special case of local scour. Scour at Sill Structures (Laursen and Flick, 1983) shall be used to estimate the scour downstream of channel drops.

5.5 ALTERNATIVES EVALUATION

The culvert, energy dissipator, and downstream channel protection designs function as an integrated system. Energy dissipators can change culvert performance and channel protection requirements. Velocity can be increased or reduced by changes in the culvert design. Downstream channel conditions (velocity, depth, and channel stability) are important considerations in energy dissipator design. The approach in developing alternative energy dissipation designs should be based on identifying the most cost-effective alternative considering the integrated system just described.

The first step should be to consider alternative culvert designs that would preclude the need for an external energy dissipator. This may be accomplished by selecting a wider culvert configuration with multiple pipe barrels, by flattening the culvert slope or incorporating a broken-back culvert profile. If an economical culvert configuration cannot be identified that would not require any energy dissipation, then proceed to a review of the available energy dissipators in Table 5–1 and Table 5–2.

An initial screening of the available types should be made to determine which energy dissipation types have potential application in the culvert setting. The screening should identify dissipator types acceptable for ADOT use and appropriate for the following site conditions:

- Froude number
- Debris load
- Tailwater conditions

The available energy dissipator types appropriate for the site conditions should be reviewed in more detail to determine the most cost effective alternative. It may be useful to use HY-8 to develop preliminary designs for each alternative and prepare a preliminary layout to determine right-of-way requirements and construction costs. Available right-of-way is often a design constraint and may dictate the selection of a dissipator type. The preliminary layout and estimated cost should provide the information needed to select a preferred energy dissipator. At this point it may be beneficial or necessary to go back and modify the culvert design based on
the results of the preliminary design to improve the performance of the selected dissipator. Once the preferred energy dissipator is identified, it may be advanced to final design.

5.6 FINAL DESIGN

Procedures for final design of energy dissipators are contained within HEC 14. Final design of the selected energy dissipator should be fully documented in the drainage report. The dissipator generalized design procedure relies upon proper layout to function as intended. Deviations from the standardized design dimensions should be avoided during layout of the dissipator.

5.7 SPECIAL CONSIDERATIONS

5.7.1 Riprap Apron Design

Chapter 10 of HEC 14 presents procedures for the design of basins or aprons lined with riprap. It should be noted that only the Riprap Basin is truly an energy dissipation device. The Riprap Apron design procedures are provided for installations where energy dissipation may not be needed, but armoring of the outlet of the culvert is required. The advantage of Riprap Basins and Aprons is that riprap material is normally available for most ADOT projects.

5.7.2 Animal Passage

If animal passage is a key component of the design, then the screening of acceptable alternatives should eliminate energy dissipators that would not be suitable for animal passage. For example, energy dissipators with riprap, vertical drops, or closed systems would not be suitable for animal passage. The selected dissipator should be approved by ADOT for applications requiring animal passage.

The selection of culverts and energy dissipators to satisfy animal passage requirements often requires coordination with stakeholder agencies such as:

- US Fish and Wildlife (USFW)
- US Forest Service (USFS)
- US Bureau of Land Management (BLM)

Requirements and stipulations for animal passage design should be coordinated with stakeholder agencies during design.

5.7.3 Spreader Channels

Spreader channels are used to return concentrated culvert outflow to the natural downstream flow conditions in areas of unconfined flow. Spreader channels are not energy dissipators. Due to the difficulty of changing concentrated flow to sheet flow, spreader channels should only be used when there is no other viable alternative. The design of spreader channels should include consideration of the invert slope along the channel length to avoid inadvertently concentrating flows at one end. Erosion protection may be required along the downstream edge to prevent
erosion of the sill, and immediately downstream of the culvert to prevent erosion from high velocity culvert outlet conditions.

5.7.4 Material Selection

Selection of materials for energy dissipators should include consideration of long term maintenance as well as aesthetic and site considerations. Wire enclosed rock, wire-tied riprap, or gabions may not be appropriate where abrasion or vandalism is a concern.

5.8 REFERENCES


Chapter 6
BRIDGES

This chapter contains details on the following:

- Bridge hydraulics design concepts, process and procedures
- Bridge types and hydraulics

6.1 INTRODUCTION

This chapter provides guidance for design of bridge crossing rivers. The chapter includes an overview of bridge hydraulics to aid in understanding the conditions that control or affect bridge performance. The chapter also describes the bridge design process to be used on ADOT projects. Over and above new bridge construction, many of ADOT’s projects involve the widening or modification of existing bridge crossings.

6.1.1 Design Resources

FHWA and ADOT guidance documents for the analysis of bridge hydraulics and scour are summarized below:

- **Evaluating Scour at Bridges, HEC 18**, (HEC 18, 2012) – This manual presents the state of knowledge and practice for the design, evaluation and inspection of bridges for scour.
- **Stream Stability at Highway Structures, HEC 20**, (HEC 20, 2012) – This manual provides guidelines for identifying stream instability problems at highway stream crossings.
- **Bridge Structural Details** (SD), ADOT - Structure details are intended to supplement contract structure drawings for bridges and appurtenant highway structures. These drawings standardize the details for construction of recurring features used in Arizona highway structures.

The hydraulic design engineer should seek input from other disciplines as cited in relevant federal publications. For scour related issues, a consensus among geotechnical, structural and bridge hydraulics team members is necessary.
6.1.2 Bridge Hydraulics Report

A Bridge Hydraulics Report should be prepared for all new and reconstruction projects. The Bridge Hydraulics Report should address flooding history of the site, general scour at piers and abutments, and erosion caused by roadway and bridge deck drainage systems. For a new alignment, the Bridge Hydraulic Report should include the location assessment of the bridge structure and the hydrology/hydraulic analysis. The Bridge Hydraulics Report should be a stand-alone document and not be combined with other documents.

Bridge Hydraulics Report requirements and general outline are included in Appendix 6C. In addition to the Bridge Hydraulics Reports, scour design elevations need to be shown on bridge design plans.

6.2 BRIDGE DESIGN

The hydraulic design of bridges requires estimates of peak discharge for the identified bridge location. Hydrologic calculation methods are presented in DDM Volume 2 - Hydrology. The required design storm frequency is set forth in DDM Volume 1 - Policy & Guidelines and is based on the Operational Drainage Frequency Class of the project highway.

6.2.1 Hydrology

In general, each bridge design should consider hydraulic evaluation flood flow frequencies defined as follows:

- **Hydraulic Design Flood Frequency**, \( Q_D \) - is the flood frequency consistent with the class of highway. See DDM Volume 1 - Policy & Guidelines for the specific Operational Drainage Frequency Class of the relevant design corridor.
- **Scour Design Flood Frequency**, \( Q_S \) - is the 100-year storm \( (Q_{100}) \).
- **Scour Design Check Flood Frequency**, \( Q_C \) - is the flood frequency equivalent to the 500-year flow (or the Overtopping Flood Frequency if less than the 500-year flood).
- **Overtopping Flood Frequency**, \( Q_{OT} \) - is described as the frequency associated with the discharge at which flow begins to spill over the roadway embankment, adjacent watershed divide or through offset structures not normally experience flow during the design storm.

When design flood values are not available from an acceptable published source (per DDM Volume 1 - Policy & Guidelines) they are to be estimated using the methodology outlined in DDM Volume 2 - Hydrology.

6.2.2 Hydraulics

The water surface profile through the bridge may be determined by the use of mathematical one- or two-dimensional models or by physical models. Where flow is essentially two-dimensional in the horizontal plane, a one-dimension model may not provide an adequate analysis of the cross-stream water surface elevations, flow velocities, or flow distribution. Two-dimensional finite
element modeling software, Finite-Element Surface Water Modeling System, FESWMS, (FESWMS, 2003) is available for the analysis of flows at bridge crossings where unusually complicated hydraulic conditions exist. For exceedingly complex sites that defy accurate or extreme practicable mathematical modeling, physical modeling may be appropriate. The constraints on physical modeling are size, cost, and time. Modeling methods other than HEC-RAS shall be used only with the prior approval of the relevant ADOT Section and Group.

One-Dimensional Modeling

In practice, the water surface profile and velocities in a section of river are often predicted using the standard step method. The United States Army Corps of Engineers (USACE) computer program Hydrologic Engineering Center – River Analysis System (HEC-RAS, 2010a) is the acceptable method for performing step backwater water surface profile computations of bridges and channels in non-prismatic channels where a one-dimensional model is acceptable.

Specific hydraulic elements for HEC-RAS modeling at bridge locations is presented in Appendix 6A and examples in HEC-RAS Reference Manuals.

Two-Dimensional Modeling

Two-dimensional modeling may be necessary for many types of steady and unsteady flow problems including multiple opening bridge crossings, spur dikes, floodplain encroachments, multiple channels, and flow around islands. Two-dimensional models are more complex and require more time to calibrate. They require essentially the same type of field data but at a greater density of information than a one-dimensional model. FESWMS is a finite element modeling software that has been developed to analyze flow at bridge crossings where complicated hydraulic conditions exist. Two-dimensional models should be used where the flow is essentially two-dimensional in the horizontal plane where a one-dimensional analysis may lead to costly over-design or possibly improper design of hydraulic structures and improvements.

Further discussion on the appropriateness of 1D- vs. 2D-models is discussed in Chapters 4, 5 and 6 in Hydraulic Design of Safe Bridges (HDS 7, 2012).

6.2.3 Scour

The following sections discuss various bridge scour prediction methods and provides recommendations and guidance that should be considered for ADOT projects. In general, design policy and criteria are not included in this chapter, but may be found in DDM Volume 1 – Policy & Guidelines.

6.2.3.1 Contraction Scour

Contraction scour is evaluated using the principle of conservation of sediment transport. Live-bed contraction scour occurs at a bridge when there is transport of bed material in the upstream reach into the bridge cross-section. In live-bed scour, the area of the contracted section increases until the fully developed scour in the bridge cross-section reaches equilibrium when sediment
transported into the contracted section equals sediment transported out. As scour develops, the shear stress in the contracted section decreases as a result of a larger flow area and decreasing average velocity. For live-bed scour, maximum scour occurs when the shear stress reduces to the point that sediment transported in equals the bed sediment transported out and the conditions for sediment continuity are in balance.

Clear-water contraction scour occurs when there is no bed material transport from the upstream reach into the downstream reach, or the material being transported in the upstream reach is transported through the downstream reach mostly in suspension and at less than the capacity of the flow. For clear-water scour, the transport into the contracted section is essentially zero and maximum scour occurs when the shear stress reduces to the critical shear stress of the bed material in the section. With clear-water contraction scour the area of the contracted section increases until, in the limit, the velocity of the flow \( v \) or the shear stress \( \tau \) on the bed is equal to the critical velocity \( V_c \) or the critical shear stress \( \tau_c \), respectively, for the representative particle size \( D_{50} \), in the bed material.

There are four cases of contraction scour at bridge sites depending on the type of contraction, and whether there is overbank flow or relief bridges. The two most common occurrences of contraction scour for bridges in Arizona are presented in Section 6.4.1.3. For any condition, it is only necessary to determine if the approach flow is transporting bed material (live-bed) or is not (clear-water), and then apply the equation appropriate for the case with the variables defined according to the location of contraction scour (channel or overbank).

### 6.2.3.2 Abutment Scour

Scour occurs at bridge abutments when the abutments and roadway embankments obstruct the flow. There are many procedures available for predicting abutment scour. Froehlich, HIRE and the NCHRP 24-20 methods for calculating abutment scour are discussed in detail in HEC 18. In general:

- Froehlich’s Abutment Scour Equation is applicable when the ration of projected abutment length \( L \) to the flow depth \( y_i \) is less than 25.
- HIRE Abutment Scour Equation is applicable when the ration of projected abutment length \( L \) to the flow depth \( y_i \) is greater than 25.
- NCHRP developed abutment scour equations considers a range of abutment types, abutment locations, flow conditions, and sediment transport conditions.

For further discussion and application of the Froehlich Equation, HIRE Equation and the NCHRP 24-20 Abutment Scour Approach refer to Chapter 8 in (HEC 18, 2012) and the example presented in Appendix 6B.

### 6.2.3.3 Pier Scour

At bridge piers, the flow that is obstructed changes into a “horseshoe” pattern around the pier. There is also a component of flow downward along the face of the pier. This flow continues
downward until either the resistance of the water consumes the energy or another obstruction is encountered. If the obstruction is the bed of the stream, the flow will, depending on the grain size, loosen the bed material creating a hole. The flow will carry the displaced material out of the hole until the combination of grain size, stream force, and depth of hole result in an equilibrium condition with no additional material being carried out of the hole for clear-water scour, or a balance of sediment inflow and outflow transport for live-bed scour.

ADOT recommends using the HEC 18 pier scour equation.

In the absence of additional site information, for all piers, a debris width of 4-feet should be added to the normal width of pier when calculating pier scour. The debris should be assumed to extend to a depth of 12-feet from the water surface or to the flow depth, whichever is less. The Drainage Engineer may use HEC 18 and knowledge of the site for guidance. Less frequent events may lead to higher scour depths.

For piers with multiple columns, if the clear opening width between columns is greater than five diameters of the column, they should be evaluated as single columns. If the clear opening width between columns is less than five diameters of the column, the overall width over the exterior columns should be used as the obstructed width, “a”. Debris width should be included in the width determined above.

The top width of a local scour hole on each side of a pier ranges from one to three times the depth of local pier scour. A top width value of two times the depth of local scour on each side of a pier is suggested for practical applications.

See Appendix 6B for an example of pier scour computation.

6.3 DESIGN CONSIDERATIONS

The following sections summarize various bridge design recommendations and guidance that should be considered for ADOT projects.

6.3.1 General Design Objectives

The process to identify the recommended stream crossing system is based on the evaluation of alternatives for site-specific criteria including capital costs, traffic service, environmental and property impacts, hazard to human life, and hydraulic performance.

The hydraulic performance of structure alternatives are based on the success in meeting the objectives outlined below.

6.3.1.1 Backwater Effects

Backwater should not significantly increase flood impact to property. The interest of adjacent property owners must be considered in the design of a proposed stream-crossing system. Not all stream crossing systems can be designed to economically pass all possible flows without backwater effects, therefore the impacts of the project on the stream should be evaluated.
Overtopping of the roadway may be used to control backwater levels for flows greater than the Roadway Operational Frequency Flow. Consider the impacts of the overtopping flow in both the bridge structure and the adjacent land.

In FEMA regulated floodplains, whenever practicable, the stream crossing should avoid encroachment on the floodway within a floodplain. When this is not feasible, modification of the floodway itself should be considered. If neither of these alternatives is feasible, FEMA regulations for "floodway encroachment" where demonstrably appropriate, should be met.

6.3.1.2 Backwater/Increases over Existing Conditions

The effects of the bridge backwater water surface profile changes should be evaluated at the right-of-way line. Consider the impact on adjacent properties during the passage of the 100-year flood for sites not covered by National Flood Insurance Program (NFIP) and Conform to FEMA regulations for sites covered by the NFIP as outlined in DDM Volume 1 – Policy & Guidelines.

There are situations where the proposed roadway and structural designs regarding the vertical positioning of a bridge result in a small vertical clearance between the low chord and the ground. Significant increases in span length provide small increases in effective waterway opening in these cases. Although it may seem possible to increase the effective area by excavating a flood channel through the reach, the desired long-term hydraulic performance does not usually result. The design usually fails to address the issue of a stable channel in regards to erosion, scour and/or aggradation. The use of waterway enlargement is discouraged. Use of waterway enlargement will require approval from the ADOT Drainage Group.

6.3.1.3 Existing Flow Distribution

Existing flow distribution should be maintained to the extent practicable. The conveyance of the proposed stream crossing location should be calculated to determine the flow distribution and to establish the location of bridge opening(s). The proposed facility should not cause any significant change in the existing flow distribution.

6.3.1.4 Velocities

Increased velocities should not damage either the structure or significantly increase impact to adjacent property. Velocity changes associated with a bridge design should be minimal. Velocities that result in predicted bed or bank erosion should be addressed. Erosion protection in the form of bed or bank protection should be included in the stream crossing system. Whenever possible, clearing of vegetation upstream and downstream of the toe of the embankment slope should be avoided. For the design storm frequency, wire-tied riprap, railbank, or soil-cement abutment protection is used to protect the roadway embankment or abutment fill slopes. Abutment protection design information is presented in Chapter 3.

Guide banks should be used, as necessary, to align flows, protect roadway embankments, and mitigate the effects of changes in the stream hydraulic behavior. However, reducing the design scour at the abutment by the use of guide banks is only acceptable if the guide bank is designed
to withstand the Scour Design Flood Frequency with the appropriate factor of safety. Guide banks are recommended to align the approach flow with the bridge opening. They are usually elliptical shaped with a major to minor axis ratio of 2.5 to 1. Their length can be determined according to Design Guideline 15 in *Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance*, (HEC 23, 2009). Guide banks and embankments should be protected by revetment for flows up to the Hydraulic Design Flood Frequency.

**6.3.1.5 Bridge Freeboard Clearance**

For new bridges on a new alignment, minimum clearance of 3 ft. for Class 1 roadways or 1-foot for Class 2, 3 or 4 roadways is required. The minimum clearance is defined as the difference between the design approach water surface elevation (upstream cross-section located at the toe of the upstream bridge embankment) and the lowest point of the bridge superstructure (bridge soffit or bottom of girder) for the selected design alternative.

For bridge widening, replacement or retrofit projects, minimum clearance less than described above needs to be discussed and approved by the ADOT Bridge Group.

Caution needs to be exercised when the profile of existing bridges are raised for hydraulic reasons to avoid adverse effects to the access of adjacent property owners from bridge and roadway approaches.

**6.3.2 Site Assessment**

Site assessments should be evaluated using the procedures and guidelines outlined in *Stream Stability at Highway Structures* (HEC 20, 2012). Arizona specific considerations for assessment categories are discussed in the following subsections.

**6.3.2.1 Location of Bridge Crossing**

The location of the bridge is driven by the roadway alignment. The site assessment is to identify any site characteristics that suggest a “minor” adjustment at the bridge site and to identify elements that need to be addressed in the design of the crossing. When a suitable crossing location has been established, specific crossing components can then be determined.

When necessary, these include:

- Probable type and approximate location of the abutments,
- Probable number and approximate location of the piers,
- Estimated depth of foundation supporting the piers (to protect against local scour),
- The location of the longitudinal encroachment in the floodplain,
- The amount of allowable longitudinal encroachment into the main channel,
- The required river training structures to ensure that river flows approach the crossing or the encroachment in a satisfactory way, and
- The profile of an existing bridge should not be raised to increase hydraulic capacity if the approach roadway is overtopped.
6.3.2.2 Stream Morphology and Stream Stability

A stream is a dynamic natural system that, as a result of the encroachment caused by elements of a stream crossing system, will respond in ways that may be unexpected. Among the many hydraulic factors that affect, and need to be considered in, the design of a stream crossing system are: floodplain width and roughness, flow distribution and direction, bed slope, stream type (braided, straight, or meandering), stream regime (aggrading, degrading, or equilibrium), and stream controls. The hydraulics of a proposed location may affect environmental considerations such as aquatic life, wetlands, sedimentation, and stream stability. The history of the stream must be considered, including assessment of long-term trends in aggradation or degradation.

The inherent complexities of stream stability, further complicated by highway stream crossings, requires a multilevel procedure as described in HEC 20. At a minimum, fluvial watercourses exhibiting potential stability issues should have a Level 1 assessment performed. The evaluation and design of a highway stream crossing or encroachment should begin with a qualitative assessment of stream stability. This involves application of geomorphic concepts to identify potential problems and alternative solutions. This analysis should be followed with quantitative analysis using basic hydrologic, hydraulic, and sediment transport engineering concepts. Such analyses could include evaluation of flood history, channel hydraulic conditions (up to and including, for example, water surface profile analysis) and basic sediment transport analyses such as evaluation of watershed sediment yield, incipient motion analysis and scour calculations.

Typical indicators of stream instability that a design engineer should look for include near vertical cut banks with obvious signs of recent movement and erosion, large deposits of sand, gravel, cobble, and boulder mixtures within the stream bed indicating significant bed-load transport, headcuts progressing upstream towards the potential bridge location, root mass exposure of old-growth trees and especially saguaro cacti, significant activity in the watershed such as grazing, development, sand and gravel operations, etc. and recent wildfire activity.

The analysis can be considered adequate for many locations if the problems are resolved and the relationships between different factors affecting stability are adequately explained. If not, a more complex quantitative analysis based on detailed mathematical modeling and/or physical hydraulic models should be considered. This multilevel approach is presented in HEC 20.

6.3.3 Water Surface Profile Modeling

The hydraulic analysis of bridges includes the computation of the flow, water surface profile, and velocity distribution. Divided or split flows have to be clearly identified.

Hydraulic computations performed for other agencies should be carefully evaluated for appropriateness in bridge hydraulic analysis. The evaluation should consider the completeness and level of detail used in the waterway model.
6.3.4 Unique Bridge Conditions and Suggested Approaches for Hydraulic Modeling

6.3.4.1 Perched Bridge

A perched bridge is one for which the road approaching the bridge is at the floodplain ground level and in the immediate vicinity of the bridge the road rises above the ground level to span the watercourse. A typical low-flow situation is low flow under the bridge and overbank flow around the bridge. If the road approaching the bridge is not much higher than the surrounding ground, the assumption of weir flow is not justified. A solution based on the energy method is a better solution, especially when a large percentage of the total discharge is in the overbank areas.

6.3.4.2 Bridges on a Skew

Skewed bridge crossings are generally handled by making adjustments to the bridge dimensions to define an equivalent cross-section perpendicular to the flow lines. It is noted in HDS 7 that skewed crossing with angles up to 20 degrees show no objectionable flow patterns. However for skew angles greater than 20 degrees, flow tended to concentrate at the abutments producing eddies, reducing the efficiency of the bridge opening and increasing the possibility of scour.

6.3.4.3 Multiple/Parallel Bridges

The hydraulic loss through closely spaced parallel bridges is between one and two times the loss for one bridge. If the bridges are far enough apart, the loss for the multiple bridges is equal to the sum of the losses for each bridge. If the bridges are very close together and the flow is not able to expand between the bridges, the contraction should occur only at the upstream bridge with only pier and friction losses at the downstream bridges, the bridges can be modeled as a single wide bridge. If there is sufficient distance between the bridges in which the flow has room to expand and contract, the bridges should be modeled as separate bridges. If separate bridges are modeled, the expansion and contraction rates should be based on the same procedure as for a single bridge. For additional 1-D modeling guidance refer to HEC-RAS, 2010 a, b, and c.

6.3.5 Roadway and Bridge Deck Drainage

The drainage engineer should coordinate to ensure that the runoff from the roadway and roadside ditches are properly discharged without causing erosion and/or environmental concerns near the bridge abutment.

6.3.5.1 Roadway Drainage

Protecting bridge abutment fill against roadway drainage-induced erosion may be achieved through providing spillways or down drains. These appurtenances should not affect the integrity of the abutment or that of the approach roadway. They should be appropriately designed to drain away from the ends of the bridge.
6.3.5.2 Deck Drainage and Hydroplaning Requirements

The methods described in Chapter 7 and in Design of Bridge Deck Drainage (HEC 21, 1993) should be used as guidelines. The hydraulic design engineer may extend the pavement drainage calculations to estimate the spread. However, pavement drainage should be intercepted prior to running onto the bridge and should be discharged away from the bridge abutments and approach roadways. Potential for hydroplaning should be minimized using special rough road surfaces. Follow the guidelines in HEC 21 for hydroplaning issues.

For deck drainage design, the frequency and the spread criteria is as specified in DDM Volume 1 - Policy & Guidelines and should be consistent with the approach roadways pavement drainage. See Chapter 7 for further discussion. For Rational Method calculations, ADOT uses a minimum time of concentration (T_c) of 10 minutes for deck drainage and a T_c of 5 minutes for hydroplaning effects.

6.4 SCOUR ASSESSMENT AND PROCEDURES

Reasonable and prudent hydraulic analysis of a bridge design requires an assessment of the proposed bridge's vulnerability to undermining due to scour.

6.4.1 Scour Types

The following bridge scour elements are considered:

- Long term profile changes (aggradation/degradation),
- Plan form change (lateral channel movement, bank widening),
- Contraction, and
- Local scour (pier and abutment).

Procedures below are the generally recommended procedures for use in most situations. In concert with Section 6.3.1 objectives presented above, the designer is encouraged to apply all the information available.

6.4.1.1 Long Term Profile Changes

Aggradation or degradation long-term profile changes can result from streambed gradient or sediment transport changes.

- Aggradation is the deposition of sediment due to a decrease in the local stream sediment transport capacity. This may be due to a decrease in the local energy gradient.
- Degradation is the scouring of bed material due to an increase in the local stream sediment transport capacity. This may result from an increase in the energy gradient or from removal of sediment at an upstream section due to gravel mining or a reservoir.
- For minor events, the wash may show aggradation but may show otherwise for major flows.
Where gravel mining is an ongoing or an expected activity, the impacts of the change in the sediment transported may result in long-term profile changes:

- Gravel mining can cause an increase in the energy gradient at the upstream end of the operation that may result in degradation of the streambed profile.
- Gravel mining can reduce the sediment transported to downstream reaches that may result in degradation of the streambed profile.

### 6.4.1.2 Plan Form Changes

Plan form changes are morphological changes such as meander migration or bank widening. The lateral movement of meanders can threaten bridge approaches as well as increase scour by changing flow patterns approaching a bridge opening. Bank widening can cause significant changes in the flow distribution and thus the bridge's flow contraction ratio.

### 6.4.1.3 Contraction and Expansion

Contraction scour results from a constriction of the flow area that may, in part, be caused by bridge piers in the waterway in addition to the encroaching embankment/abutment. Deposition results from an expansion of the channel or the bridge site being positioned at the beginning of a flatter reach of stream.

Highways, bridges, and natural channel constrictions are the most commonly encountered cause of contraction scour. The scour is considered as either live-bed or clear-water contraction scour. The two most common occurrences of contraction scour for bridges in Arizona are:

**Case I.** Overbank flow on a floodplain being forced back into the main channel. Case I occurrences include the following conditions:

- The river channel width becomes narrower either due to the bridge abutments projecting into the channel or the bridge being located at a narrowing reach of the river.
- Overbank flow area is obstructed either partially or completely by the road embankment/abutment with no constriction of the main channel.

**Case II.** Flow is confined to the main channel (i.e. there is no overbank flow). The normal river channel becomes narrower due to the bridge itself or the bridge site is located at a narrowing reach of the river.

### 6.4.1.4 Local Scour (Pier and Abutment)

Exacerbating the potential scour hazard at a bridge site are any abutments or piers located within the flood flow prism. The amount of potential scour caused by these features is termed local scour. Local scour is a function of the geometry of these features as they relate to the flow geometry.
6.4.2 Special Scour Conditions

6.4.2.1 Armoring

Armoring occurs because a stream or river is unable to move the more coarse material during a particular flood event comprising either the bed or if some bed scour occurs its underlying material. A review of the armored material may reveal well-rounded material that has been transported; not a coarse resident bed material. Scour may occur initially but later becomes arrested by armoring before the full scour potential is reached again for a given flood magnitude. When armoring does occur, the coarser bed material will tend to remain in place or quickly redeposit so as to form a layer of riprap-like armor on the streambed or in the scour holes and thus limit further scour for a particular discharge. When a larger flood occurs than the flood that created the armoring, scour will probably penetrate deeper until armoring again occurs at some deeper threshold.

Armoring may result in the stream being unable to satisfy its desired sediment transport capacity, possibly resulting in bank widening. Bank widening encourages rivers or streams to seek a more unstable, braided regime. Such instabilities may pose serious problems for bridges as they encourage further, difficult to assess plan form changes. Bank widening also spreads the approach flow distribution that in turn results in a more severe bridge opening contraction.

6.4.2.2 Scour Resistant Materials

Caution is necessary in determining the scour resistance of bed materials and the underlying strata. Serious scour has been observed to occur in materials commonly perceived to be scour resistant such as consolidated soils and glacial till, as well as so-called bedrock streams and streams with gravel and boulder beds. With sand size material, the passage of a single flood may result in the predicted scour depths. Conversely, in scour resistant material the maximum predicted depth of scour might not be realized during the passage of a particular flood; however, some scour resistant material may be lost. Commonly, the stream will replace this material with transported material that is more easily scoured. Thus, at some later date another flood may reach the predicted scour depth. The methods described in Chapter 4 in Evaluating Scour at Bridges (HEC 18, 2012) should be used as guidelines.

6.4.2.3 Pressure Scour

Pressure flow results from a build-up of water on the upstream bridge face, and a plunging of the flow downward and under the bridge. This occurs when the water surface elevation at the upstream face of the bridge is greater than or equal to the low chord of the bridge superstructure. At higher approach flow depths, the bridge can be entirely submerged with the resulting flow being a complex combination of the plunging flow under the bridge and the flow over the bridge.

With pressure flow, the local scour depths at a pier or abutment are larger than for free surface flow with similar depths and approach velocities. The increase in local scour at a pier subject to pressure flow results from the flow being directed downwards toward the bed by the superstructure and by increasing the intensity of the horseshoe vortex. The vertical contraction...
of the flow is a more significant cause of the increase in scour depth. However, in many cases, when a bridge becomes submerged, the average velocity under it is reduced due to a combination of additional backwater caused by the bridge superstructure impeding the flow and a reduction of discharge passing under the bridge due to weir flow over the bridge and approach embankments. As a consequence of this, increases in local scour attributed to pressure flow scour at a particular site, may be offset to a degree by lesser velocities through the bridge opening due to increased backwater and a reduction in discharge under the bridge due to overtopping.

HEC-RAS can be used to determine the discharge through the bridge and the velocity of approach and depth upstream of the piers when flow impacts the bridge superstructure. These values should be used to calculate local pier scour. Engineering judgment will then be exercised to determine the appropriate multiplier to be applied times the calculated pier scour depth for the pressure flow scour depth. This multiplier ranges from 1.0 for a low approach Froude numbers (Fr = 0.1) to 1.6 for high approach Froude numbers (Fr = 0.6). If the bridge is overtopped, the depth to be used in the pier scour equations and for computing the Froude number is the depth to the top of the bridge deck or guardrail obstructing the flow.

6.4.3 Scour Assessment Procedures

The following scour assessment procedure should be used for all ADOT bridge construction and rehabilitation projects.

6.4.3.1 Collect Site Data

Geometry

Obtain existing stream and floodplain cross-sections and profile, site plan and the stream's present, and where possible, historic geomorphic plan and profile form. Also, locate the bridge site with respect to such things as other bridges in the area, tributaries to the stream or close to the site, bedrock controls, man-made controls (dams, old check structures, river training works, etc.), and downstream confluence with other streams. Locate (distance and height) any "headcuts" due to natural causes or activities such as gravel mining operations. Data related to plan form changes such as meander migration and the rate at which they may be occurring are useful.

When gravel mining is an ongoing activity or should be expected, the impacts of gravel mining on the stream should be evaluated. Upstream gravel mining operations may “capture” the bed material discharge resulting in the more adverse clear-water scour case discussed later. Current practice is to include an allowance for future degradation at the bridge site where extensive mining is occurring, such as the Salt River in Phoenix or the Santa Cruz River in Tucson. For isolated gravel mining, an estimate of the degradation depth may be made using the procedures in Effects of In-Stream Mining on Channel Stability (ADOT, 1989).
Bed Material

The bed material should be observed. Look for evidence of bedrock outcrops, grain size for determination of bed forms and type of scour, and other indicators of the morphology of the stream. ADOT practice is to consider all material scour susceptible unless proven otherwise. Grain size is not considered for the purpose of decreasing the predicted scour (i.e. stream bed armoring).

Geomorphology

Classify the geomorphology of the site; i.e., such things as whether it is a floodplain stream or crosses an alluvial fan; youthful, mature or old age, presence of headcuts, and meanders.

Historic Scour

Review available information such as as-buils, bridge inspection reports, old contour mapping, and aerial photographs to evaluate scour data on other bridges or similar facilities along the stream.

Debris

A buildup of debris on the pier should be considered. In the absence of site specific information, the debris should be assumed to extend 2 feet on each side of the pier and have a depth of 12 feet from the water surface. The use of debris determination procedures in HEC 18 should only be used with approval from ADOT Bridge Group.

Hydrology

Identify the character of the stream hydrology; i.e., perennial, ephemeral, intermittent as well as whether it is "flashy" or subject to broad hydrograph peaks resulting from gradual flow increases such as occur with general thunderstorms or snowmelt and dam releases. The hydraulic design flood, scour design flood, and the scour design check flood frequencies will be required.

6.4.3.2 Bridge Scour Determination Procedure

Step 1  Determine the magnitude of the hydraulic design flood, scour design flood, and scour design check flood and the magnitude of the incipient overtopping flood. Accomplish steps 2 through 8 using the discharges computed.

Review the site to identify the discharge that places the greatest stress on the bed material in the bridge opening. Where there are relief structures on the floodplain or overtopping occurs, some flood other than the base flood or 500-year event may cause the worst case bridge opening scour. This situation occurs where the bridge opening will pass the greatest discharge just prior to incurring a discharge relief from overtopping or a floodplain relief opening. Conversely care must be exercised in that a discharge relief at the bridge due to overtopping or relief openings might not result in reduction in the bridge opening discharge. Should a reduction in the bridge
opening discharge occur, the incipient overtopping flood or the overtopping flood corresponding to the base flood is to be used to evaluate the bridge scour. If the overtopping flood discharge is less than the 500-year flood, then the overtopping discharge is considered the scour design check flood for bridge design.

Step 2  
Assess the bridge crossing reach of the stream for **profile** bed scour changes to be expected from degradation or aggradation. Take into account past, present and future conditions of the stream and watershed in order to forecast what the elevation of the bed might be in the future. Certain plan form changes such as migrating meanders causing channel cutoffs would be important in assessing future streambed profile elevations. The possibility of downstream mining operations inducing "headcuts" should be considered. The quickest way to assess streambed elevation changes due to "headcuts" (degradation) is by obtaining a vertical measurement of the downstream "headcut(s)" and projecting that measurement(s) to the bridge site using the existing stream slopes if it is acceptable to assume the stream is in regime conditions; if it is not, then it may be necessary to estimate the regime slope. A more time consuming way to assess elevation changes would be to use a sediment routing practice in conjunction with a synthetic flood history.

Step 3  
Assess the bridge crossing reach of the stream for **plan form** scour changes. Attempt to forecast whether an encroaching meander will cause future problems within the expected life of the bridge. Take into account past, present and expected future conditions of the stream and watershed in order to forecast how such meanders might influence the approach flow direction in the future. This forensic analysis on a site’s past geomorphic history may prove useful to forecast the future. Otherwise this assessment has to be largely subjective in nature.

Step 4  
Develop a water surface profile for the discharges to be considered through the site's reach for fixed bed conditions using HEC-RAS.

Step 5  
Assess the magnitude of contraction scour based on the fixed bed hydraulics.

Step 6  
Estimate local scour using the channel and bridge hydraulics assuming no bed armoring. Abutment scour is estimated using abutment methods outlined in Section 6.2.3.2 method and as demonstrated in Appendix 6B. Pier scour is estimated using the HEC 18 pier scour equation as also demonstrated in Appendix 6B.

Step 7  
For each discharge under consideration, plot the scour depths from Step 2 through 6 on a cross-section of the stream channel and floodplain at the bridge site. Using judgment, enlarge any overlapping scour holes.

Step 8  
The resultant predicted scour needs to be discussed with the geotechnical and bridge designers to confirm the assumptions made and to verify the erodibility of the bed material. As the goal is for the bridge not to fail, the bed material must be proven to be non-erodible.
6.5 REFERENCES

ADOT, Bridge Structure Details (SD)

ADOT, Bridge Design Guidelines


APPENDIX 6A
HEC-RAS EXAMPLE
Example No. 6-1  HEC-RAS Model

The following data from a HEC-RAS Model will be used in the bridge scour example presented in Appendix 6B. Typical model set up for bridges are discussed in detail in the HEC-RAS User’s Manual.

1. HEC-RAS Cross-section Layout encroached conditions (with bridge).
2. Encroached Conditions (Bridge Section).

The distance from the bridge to the upstream section is 30 feet. Using a 1:1 contraction ratio, at River Station 14.50, the ineffective flow is set at 820 and 1530. At section 21.0, the encroachment stations are 170 and 2180. For the downstream section, use an expansion coefficient of 2:1, this is from the expansion coefficient table (in HEC-RAS Manuals) for \( b/B = 0.25 \), slope = 10 feet/mile and \( N_{ob}/N_{mc} = 1.3 \). Range of expansion coefficients is typically 1.3 to 2.0.

Expansion and contraction components are set per HEC-RAS User’s Manual at 0.1/0.3 respectively for a minimal contraction/expansion ration and 0.3/0.5 at typical bridge encroachment sections.

For the purposes of the scour evaluation in Appendix 6B, the upstream face of the bridge is cross-section 14.5 the fully expanded upstream cross-section (approach) is cross-section 21.
3. HEC-RAS Cross-sections and model results.
Detailed output for Cross-section 21

Flow Distribution for Cross-section 21
Detailed output for Cross-section 14.5

Flow Distribution for Cross-section 14.5
Detailed output for upstream internal Bridge section 14.0

Flow Distribution for upstream internal Bridge section 14.0
APPENDIX 6B
BRIDGE SCOUR EXAMPLE
Example No. 6-2  Bridge Scour Example

The hydraulic model summarized in Appendix 6A was used to determine the scour characteristics at the bridge crossing. The model was setup using guidance outlined in HEC-RAS Users Manuals for cross-section placement, expansion/contraction and other bridge hydraulic characteristics. Three cross-sections were used for this analysis and are denoted as cross-section 21 or “Approach” section, cross-section 14.5 or the “Upstream face of the bridge” and 14.0 for the Internal Bridge Section. See Appendix 6A for cross-section layout and overview.

The following three elements are evaluated at bridge structures in Arizona for the Scour Design Flood and Scour Design Check Flood.

1.  Long term aggradation/degradation (evaluation may not be flood event specific)
2.  Contraction Scour
3.  Local Scour at Piers and Abutments

The following example did not consider long-term degradation/aggradation and contraction scour components. Example of the application of these elements can be found in HEC 18 and HEC 20. The focus of the following example will be the evaluation of local scour at piers and abutments at bridges in Arizona.

Problem:

Determine the pier and abutment scour at a bridge crossing:

The hydraulic data was taken for the HEC-RAS model summarized in Appendix 6A.

Given:

- Discharge (Scour Design Flood): $Q_s = 30,000 \text{ cfs}.$
- The bridge is 650 feet long between the faces of abutments.
- The left abutment is set at station 850, 200 feet left of the bank.
- Right abutment station 1515.
- Right bank is at station 1500.
- The bridge has 6 stem wall piers that are 5-feet thick and 40-feet long. Center of the piers are at the following stations: 957.9, 1050.7, 1143.6, 1236.4, 1329.3, and 1422.1.
- For abutment scour prediction for foundation design, assume embankment in front of abutment is scoured; therefore, the abutment scour is calculated for a vertical face.
- Bed material is sand with a $D_{50}$ of 0.0066 ft.

Solution:

1.  Determine Pier Scour using the HEC 18 Pier Scour Equation

Since the depth of flow is less than 12-feet the scour is predicted for only the case with debris. The scour elevation is measured from bottom of debris, the streambed.
Pier Geometry: Six round nose stem wall piers that are 5-feet thick and 40-feet long

Debris: ADOT assumes 2-feet of debris on each side of the pier that extends from the water surface elevation down 12-feet.

Effective width: \( a = 5.0 \text{ ft} + 4.0 \text{ ft} = 9.0 \text{ ft}. \) (includes 4-feet for debris)

Angle of attack: ADOT minimum = 15 degrees.

Max Velocity (v): 8.97 ft/sec (flow distribution at cross-section 14.5 in Appendix 6A)

Max Depth \((y_1)\): 10.89 ft (15.34 – 4.45, detailed output table at cross-section 14.5 in Appendix 6A)

Froude number: \( F_r = \frac{v}{\sqrt{g y}} \)

\( F_r = \frac{8.97}{\sqrt{32.2(10.89)}} = 0.48 \)

\( K_1 = 1.0 \) (Per HEC 18, Table 7.1 for round nose)

\( K_2 = 1.55 \), Interpolated from HEC 18, Table 7.2 assuming debris \((L/a = 4.4)\) and an angle of attack 15 degrees

\( K_3 = 1.1 \) (Per HEC 18, Table 7.3)

Therefore:

\[
\frac{y_s}{y_1} = 2.0K_1K_2K_3\left(\frac{a}{y_1}\right)^{0.65}F_r^{0.43}
\]

\[
y_s = 10.89 \left[ 2.0 \times 1.0 \times 1.55 \times 1.1 \left(\frac{9}{10.89}\right)^{0.65} \times 0.48^{0.43} \right]
\]

\[
y_s = 23.9 \text{ ft}
\]

2. Determine Abutment Scour using Froehlich’s Method

\[
\frac{y_s}{y_a} = 2.27K_1K_2\left(\frac{L'}{y_a}\right)^{0.43}F_r^{0.61} + 1
\]

where:

\( y_s = \) Scour depth (ft),

\( K_1 = \) Coefficient for abutment shape, See HEC 18, Table 8.1,

\( K_2 = \) \((\theta/90)^{0.13}\), coefficient for angle of embankment to flow, See HEC 18, Figure 8.5 for definition of \( \theta \),

\( L' = \) Length of active flow obstructed by the embankment (ft),

\( y_a = \) Average depth of flow on the floodplain (ft), and

\( F_r = \) Froude number of approach flow upstream of the abutment
From cross-section 21, flow distribution with bridge in place:

<table>
<thead>
<tr>
<th>Pos Flow</th>
<th>Pos Flow</th>
<th>Flow (cfs)</th>
<th>Area (sq ft)</th>
<th>W.P (ft)</th>
<th>Convey %</th>
<th>Hydr Depth (ft)</th>
<th>Vel. (ft/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 LOB</td>
<td>0</td>
<td>185</td>
<td>0.00</td>
<td>66.26</td>
<td>57.61</td>
<td>0.00</td>
<td>1.15</td>
</tr>
<tr>
<td>2 LOB</td>
<td>185</td>
<td>490</td>
<td>1877.35</td>
<td>915.78</td>
<td>305.01</td>
<td>6.26</td>
<td>3.00</td>
</tr>
<tr>
<td>3 LOB</td>
<td>490</td>
<td>795</td>
<td>2333.77</td>
<td>1043.50</td>
<td>305.00</td>
<td>7.78</td>
<td>3.42</td>
</tr>
<tr>
<td>14 LOB</td>
<td>795</td>
<td>850</td>
<td>591.68</td>
<td>230.85</td>
<td>55.00</td>
<td>1.97</td>
<td>4.20</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td></td>
<td></td>
<td>4803</td>
<td>2256</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table values prorated for abutment station at 850.

For the left abutment:

The obstructed flow, \( Q_{obl} \) = 4803 cfs

The obstructed area, \( A_{obl} \) = 2256 ft²

\[
K_1 = 1, \text{ for vertical face abutment (See HEC 18, Table 8.1)},
\]

\[
K_2 = 1, \text{ for embankment perpendicular to flow (See HEC 18, Figure 8.5 for definition } \theta,)
\]

\[
L' = \text{Length of active flow obstructed by the embankment,
}= 850 - 185 = 665 \text{ ft}
\]

\[
y_a = \text{Average depth of flow on the floodplain, } A_{obl}/L' = 2256/665= 3.39 \text{ ft}
\]

\[
L'/y_a = 665/3.39 = 196, \text{ Greater than 25, use } L'/y_a = 25
\]

\[
\text{Fr} = \text{Froude number of approach flow upstream of the abutment:}
= \frac{v}{(gy_a)^{0.5}} = \frac{2.13}{((32.2)(3.41))^{0.5}} = 0.204
\]

\[
\frac{y_s}{y_a} = 2.27K_1K_2(L'/y_a)^{0.43}\text{Fr}^{0.61} + 1 \quad \text{HEC 18}
\]

\[
\frac{y_s}{y_a} = 2.27(1)(1)(25)^{0.43}0.204^{0.61} + 1
\]

\[
\frac{y_s}{y_a} = 4.43
\]

\[
y_s = 4.43 (3.39) = 15.0 \text{ ft}
\]
Table 6–2  Flow Distribution from Cross-section 21, Right Overbank

<table>
<thead>
<tr>
<th>Pos Flow</th>
<th>Left Sta (ft)</th>
<th>Right Sta (ft)</th>
<th>Flow (cfs)</th>
<th>Area (sq ft)</th>
<th>W.P (ft)</th>
<th>Convey %</th>
<th>Hydr Depth (ft)</th>
<th>Vel. (ft/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1) 10 ROB</td>
<td>1515</td>
<td>1731.8</td>
<td>2479.31</td>
<td>973.75</td>
<td>216.80</td>
<td>8.27</td>
<td>4.20</td>
<td>2.38</td>
</tr>
<tr>
<td>11 ROB</td>
<td>1731.8</td>
<td>1963.6</td>
<td>1994.99</td>
<td>877.88</td>
<td>231.80</td>
<td>6.65</td>
<td>3.79</td>
<td>2.27</td>
</tr>
<tr>
<td>12 ROB</td>
<td>1963.6</td>
<td>2195.4</td>
<td>1644.79</td>
<td>781.87</td>
<td>231.80</td>
<td>5.48</td>
<td>3.37</td>
<td>2.10</td>
</tr>
<tr>
<td>13 ROB</td>
<td>2195.4</td>
<td>2600</td>
<td>0.00</td>
<td>761.73</td>
<td>281.27</td>
<td>0.00</td>
<td>2.71</td>
<td>0.00</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td></td>
<td>6119.09</td>
<td>3395.23</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1Table values prorated for abutment station at 1515.

For the right abutment:

The obstructed flow, $Q_{obr} = 6119$ cfs
The obstructed area, $A_{obl} = 3395$ ft$^2$

\[
K_1 = 1, \text{ for vertical face abutment (See HEC 18, Table 8.1)},
\]

\[
K_2 = 1, \text{ for embankment perpendicular to flow (See HEC 18, Figure 8.5 for definition } \theta),
\]

\[
L' = \text{ Length of active flow obstructed by the embankment, } 2195 - 1515 = 680 \text{ ft}
\]

\[
y_a = \text{ Average depth of flow on the floodplain, } A_{obl}/L' = 3395/680 = 4.99 \text{ ft}
\]

\[
L'/y_a = 695/4.99 = 136, \text{ Greater than 25, use } L'/y_a = 25
\]

\[
Fr = \text{ Froude number of approach flow upstream of the abutment: } v = Q/A = 6119/3395 = 1.80 \text{ ft/sec.}
\]

\[
Fr = \frac{v}{(gy_a)^{0.5}} = \frac{2.13}{(32.2)(4.99)^{0.5}} = 0.142
\]

\[
\frac{y_s}{y_a} = 2.27K_1K_2(L'/y_a)^{0.43}Fr^{0.61} + 1 \quad \text{HEC 18 Equation 8.1}
\]

\[
\frac{y_s}{y_a} = 2.27(1)(1)(25)^{0.43}0.142^{0.61} + 1
\]

\[
\frac{y_s}{y_a} = 3.76
\]

\[
y_s = 3.76 (4.99) = 18.7 \text{ ft}
\]
3. Determine Abutment Scour using HIRE Method

For HIRE, scour is a function of the depth and Froude number for the flow adjacent to the abutment.

\[
\frac{y_s}{y_1} = 4.0Fr^{0.33} \frac{K_1}{0.55} K_2
\]

where:
- \(y_s\) = Scour depth (ft),
- \(K_1\) = Coefficient for abutment shape, See HEC 18, Table 8.1
- \(K_2\) = \((\theta /90)^{0.13}\), coefficient for angle of embankment to flow, See HEC 18, Figure 8.5 for definition of \(\theta\),
- \(y_1\) = Depth of flow at the abutment on the overbank or in the main channel (ft), and
- \(Fr\) = Froude Number based on the velocity and depth adjacent to and upstream of the abutment

**For the left abutment:**

At left abutment: \(y_1 = 4.30, V = 3.93. Fr = 0.337\) (Flow distribution at cross-section 14.5)

\[
y_s = 4.0Fr^{0.33} \frac{K_1}{0.55} K_2 y_1 = 4.0 (0.337)^{0.33} \frac{1}{0.55} (1)(4.30)
\]

\[
y_s = 21.8 \text{ ft}
\]

**For the right abutment:**

At right abutment: \(y_1 = 6.23, V = 6.47. Fr = 0.429\) (Flow distribution at cross-section 14.5)

\[
y_s = 4.0Fr^{0.33} \frac{K_1}{0.55} K_2 y_1 = 4.0 (0.429)^{0.33} \frac{1}{0.55} (1)(6.23)
\]

\[
y_s = 36.3 \text{ ft}
\]
4. Determine Abutment Scour using NCHRP 24-20 Abutment Scour Approach

The following scour conditions will be evaluated:

(a) Scour occurring when the abutment is in or close to the main channel,
(b) Scour occurring when the abutment is set back from the main channel, and
(c) Scour occurring when the embankment breaches and the abutment foundation acts as a pier.

a. Evaluate bridge site for scour conditions (a) and/or (b)

If the projected length of the embankment, \( L \), is 75 percent or greater than the width of the floodplain, \( B_f \), scour condition (a) occurs and the contraction scour calculation is performed using a live-bed scour calculation.

or

If the projected length of the embankment, \( L \), is less than 75 percent of the floodplain, \( B_f \), scour condition (b) occurs and the contraction scour calculation is performed using a clear-water scour calculation.

For the left abutment:

Where \( B_f \) is the width of the floodplain upstream of the bridge (Cross-section 14.5)

\[ L = \text{toe of abutment station} - \text{left edge of water} = 879 \text{ ft} - 119 \text{ ft} = 760 \text{ ft} \]

\[ B_f = \text{left bank station} - \text{left edge of water} = 1100 \text{ ft} - 119 \text{ ft} = 981 \text{ ft} \]
L/Bf = 760 ft/981 ft = 0.77 > 0.75 live-bed scour calculation (Close so clear-water should be checked too)

**For the right abutment:**

L = Right edge of water – toe of abutment station = 2481 ft – 1500 ft = 981 ft
Bf = Right edge of water – right bank station = 2481 ft – 1500 ft = 981 ft
L/Bf = 981 ft/981 ft = 1.0 > 0.75 live-bed scour calculation

∴ Check left abutment for live bed and clear-water conditions and right abutment for live bed condition.

b. Calculate \( y_c \) for left abutment (Live Bed)

\[
y_c = y_1 \left( \frac{q_{2c}}{q_1} \right)^{6/7}
\]

where:
- \( y_c \) = Flow depth including live-bed contraction scour (ft),
- \( y_1 \) = Upstream flow depth (ft),
- \( q_{2c} \) = Unit discharge in the constricted opening accounting for non-uniform flow distribution (ft\(^2\)/s), and
- \( q_1 \) = Upstream unit discharge (ft\(^2\)/s).

Use SBR guidance in Chapter 8, HEC 18 to determine unit discharges used in HEC 18, Equation 8.5.

Left abutment SBR > 5

q\(_1\) = upstream overbank unit discharge (Section 21)

\[
v = \frac{Q}{A} \text{ for the left overbank} = \frac{7492.26}{3305.74} = 2.27 \text{ ft/sec.}
\]

q\(_1\) = (velocity)(overbank floodplain depth) = (2.27)(3.54) = 8.02 ft\(^3\)/sec

q\(_{2c}\) = bridge overbank unit discharge (Section 14 BU)

\[
v = \frac{Q}{A} \text{ for the left overbank} = \frac{4026.20}{853.25} = 4.72 \text{ ft/sec.}
\]

q\(_{2c}\) = (velocity)(overbank floodplain depth) = (4.72)(4.25) = 20.05 ft\(^3\)/sec
\[ y_c = y_1 \left( \frac{q_{2c}}{q_1} \right)^{6/7} \]
\[ y_c = 3.54 \left( \frac{20.05}{8.02} \right)^{6/7} \]
\[ y_c = 7.8 \text{ ft} \]

c. Calculate \( y_c \) for left abutment (Clear-water)

\[ y_c = \left( \frac{q_{2f}}{K_u D_{50}^{1/3}} \right)^{6/7} \]

Equation 8.6

where:

- \( y_c \) = Flow depth including clear-water contraction scour (ft),
- \( q_{2f} \) = Unit discharge in the constricted opening accounting for non-uniform flow distribution (ft²/s),
- \( K_u \) = 11.17 English units, and
- \( D_{50} \) = Particle size with 50 percent finer (ft).

Use SBR guidance in Chapter 8, HEC 18 to determine unit discharges used in HEC 18, Equation 8.6.

Left abutment SBR > 5

\[ q_{2f} = \text{left overbank unit discharge (Section 14 BU)} \]
\[ v = \frac{Q}{A} \text{ for the left overbank} = \frac{4026.20}{853.25} = 4.72 \text{ ft/sec.} \]
\[ q_{2f} = \text{(velocity)(left overbank floodplain depth)} = (4.72)(4.25) \]
\[ = 20.05 \text{ ft}^3/\text{sec} \]

\[ y_c = \left( \frac{q_{2f}}{K_u D_{50}^{1/3}} \right)^{6/7} \]
\[ y_c = \left( \frac{20.05}{11.17(0.0066)^{1/3}} \right)^{6/7} \]
\[ y_c = 6.9 \text{ ft} \]
d. Calculate \( y_c \) for left abutment (Live bed)

\[
y_c = y_1 \left( \frac{q_{2c}}{q_1} \right)^{6/7}
\]

where:
- \( y_c \) = Flow depth including live-bed contraction scour (ft),
- \( y_1 \) = Upstream flow depth (ft),
- \( q_{2c} \) = Unit discharge in the constricted opening accounting for non-uniform flow distribution (ft²/s), and
- \( q_1 \) = Upstream unit discharge (ft²/s),

Use SBR guidance in Chapter 8, HEC 18 to determine unit discharges used in HEC 18, Equation 8.5.

Right abutment determined using SBR < 5 and SBR > 5 since left abutment SBR > 5 and right abutment SBR < 5. See HEC 18, Figure 8.16.

\[
q_1 = \text{upstream overbank unit discharge (Section 21)}
\]
\[
v_1 = \frac{Q}{A} \text{ for the right overbank} = \frac{(6149.39)/(3294.99)} = 1.87 \text{ ft/sec.}
\]
\[
q_1 = \text{(velocity)(overbank floodplain depth)} = (1.87)(3.71) = 6.92 \text{ ft}^2/\text{sec}
\]
\[
q_{2c} = \text{main channel and overbank unit discharge (Section 14 BU)}
\]
\[
v_2 = \frac{Q}{A} \text{ for the main channel and right overbank} = \frac{(25912.24 + 61.56)/(2853.17 + 19.58)} = 9.04 \text{ ft/sec.}
\]
\[
q_{2c} = \text{(velocity)(main channel and right overbank floodplain depth)} = (9.04)((2853.17 + 19.58)/(380 + 8.85)) = 66.79 \text{ ft}^2/\text{sec}
\]
\[
y_c = y_1 \left( \frac{q_{2c}}{q_1} \right)^{6/7}
\]
\[
y_c = 3.71 \left( \frac{66.79}{6.92} \right)^{6/7}
\]
\[
y_c = 25.9 \text{ ft}
\]
e. Determine $\alpha_A$ and/or $\alpha_B$ for each abutment and $y_{max}$

$$y_{max} = \alpha_A y_c \text{ or } y_{max} = \alpha_B y_c$$

$$y_s = y_{max} - y_0$$

where: $y_{max} =$ Maximum flow depth resulting from abutment scour (ft),
$y_c =$ Flow depth including live-bed or clear-water contraction scour (ft),
$\alpha_A =$ Amplification factor for live-bed conditions,
$\alpha_B =$ Amplification factor for clear-water conditions,
$y_s =$ Abutment scour depth (ft), and
$y_0 =$ Flow depth prior to scour (ft).

i. Wingwalls and Live-bed conditions (Left Abutment)

Using HEC 18 Figure 8.10 and $q_2/q_1 = 20.05/8.02 = 2.5$

$$\alpha_A = \frac{y_{max}}{y_c} = 1.14$$

$$y_{max} = \alpha_A y_c = 1.14 \times 7.8$$

$$y_{max} = 8.8 \text{ ft}$$

ii. Wingwalls and clear-water conditions (Left Abutment)

Using HEC 18 Figure 8.12 and $q_2/q_f = 20.05/8.02 = 2.5$

$$\alpha_B = \frac{y_{max}}{y_c} = 1.7$$

$$y_{max} = \alpha_B y_c = 1.7 \times 6.9$$

$$y_{max} = 11.8 \text{ ft}$$

iii. Wingwalls and Live-bed conditions (Right Abutment)

Using HEC 18 Figure 8.10 and $q_2/q_1 = 66.79/6.92 = 9.65$ use 3.0

$$\alpha_A = \frac{y_{max}}{y_c} = 1.1$$
$y_{max} = \alpha_A y_c = 1.1 \times 25.9$

$y_{max} = 28.5\text{ ft}$

f. Determine $y_s$ for each abutment

i. Wingwalls and live-bed conditions (Left Abutment)

$y_o = \text{flow depth prior to scour at Section 14 BU}$

$y_o = \text{water surface} - \text{ground elevation}$

$y_o = 14.92 - 11.5 = 3.42\text{ ft}$

$y_s = y_{max} - y_0 = 8.8 - 3.4$

$y_s = 5.4\text{ ft}$

ii. Wingwalls and clear-water conditions (Left Abutment)

$y_s = y_{max} - y_0 = 11.4 - 3.4$

$y_s = 8.0\text{ ft}$

iii. Wingwalls and live-bed conditions (Right Abutment)

$y_o = 14.92 - 4.45 = 10.47\text{ ft}$

$y_s = y_{max} - y_0 = 28.5 - 10.5$

$y_s = 18.0\text{ ft}$

f. Determine scour condition (c) for the right abutment.

As the bank erodes, the abutment may be exposed to flow as a pier, scour is also computed for this condition.
Assume the right abutment is supported on two shafts that act independently. The shafts have a 5 ft diameter at 25 foot spacing. Use the hydraulic parameters for flow through the bridge (Station 14 BR U). Using the maximum hydraulic data from the flow distribution, max channel hydraulic depth = 10.13 ft. and velocity = 9.87 ft/sec. Channel low point elevation 4.35.

Given:

- \( a = 5 \) ft (no debris added to pier width)
- \( Y_1 = 10.12 \) ft
- \( V_1 = 9.77 \) ft/sec
- \( Fr_1 = \frac{V_1}{(gY_1)^{1/2}} = \frac{9.77}{((32.2)(10.12)^{1/2})} = 0.55 \)
- \( K_1 = 1 \)
- \( K_2 = 1 \)
- \( K_3 = 1 \)

Using HEC 18 Pier scour Equation:

\[
\frac{y_s}{y_1} = 2.0K_1K_2K_3 \left( \frac{a}{y_1} \right)^{0.65} Fr_1^{0.43}
\]

HEC 18

Equation 7.1

\[
\frac{y_s}{y_1} = 2.0(1)(1)(1) \left( \frac{5}{10.13} \right)^{0.65} 0.55^{0.43}
\]

\( y_s = 10.12(1.068) \)

\( y_s = 10.8 \) ft
5. Total Bridge Scour Summary

When using the Froehlich/HIRE method for abutment scour the composite scour results are applied to the bridge scour profile as follows;

**Abutments:** Long Term Degradation + Abutment Scour + Contraction Scour

**Pier:** Long Term Degradation + Contraction Scour + Pier Scour

When using the NCHRP 24-20 Abutment Scour Approach the composite scour results are applied to the bridge scour profile as follows;

**Abutments:** Long Term Degradation + Abutment Scour Method (a), (b) or (c).

**Pier:** Long Term Degradation + Contraction Scour + Pier Scour

<table>
<thead>
<tr>
<th>Table 6-3 Bridge Scour Summary</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Left Abutment</strong></td>
</tr>
<tr>
<td>Ground Elevation</td>
</tr>
<tr>
<td><strong>Method</strong></td>
</tr>
<tr>
<td>Froehlich¹</td>
</tr>
<tr>
<td>HIRE</td>
</tr>
<tr>
<td>NCHRP (a)</td>
</tr>
<tr>
<td>NCHRP (b)</td>
</tr>
<tr>
<td>NCHRP (c)</td>
</tr>
<tr>
<td>Pier Scour</td>
</tr>
</tbody>
</table>

¹Froehlich’s with L'/ya limited to 25.

²Elevations are calculated by subtracting scour depth from thalweg elevation.

For this example, use lower values from HIRE for design scour elevation.
APPENDIX 6C
BRIDGE HYDRAULICS
REPORT REQUIREMENTS
DEVELOPMENT OF BRIDGE HYDRAULICS REPORTS

A stand-alone Bridge Hydraulics Report shall be prepared for all new construction and reconstruction projects. When traffic capacity is increased, the project may be considered as a reconstruction project. The Bridge Hydraulics Report should address flooding history of the site, erosion of the abutments from roadway and bridge deck drainage. For a new alignment, the location of the drainage structures and the hydrology analysis should be finalized during the scoping stage. The construction plans shall clearly indicate:

- The design discharge value and the water surface elevation, and
- The 100-year design discharge elevations if in FEMA floodplain, and
- Either the 500-year or overtopping discharge.

The above values shall be marked on the elevation view. This requirement is applicable for all new waterway bridges and for any bridge project that may affect the existing drainage pattern. Typical Table of Contents of a Bridge Hydraulics Report and a Project Assessment Report for a bridge scour retrofit project are shown below. The Bridge Hydraulics Report shall be a stand-alone document and not be combined with other documents. The final Bridge Hydraulics Report shall be distributed during Stage III project submittal. The document shall contain details of the location of the piers and scour depth estimates. These reports and their supporting documents shall be submitted electronically, in Acrobat® format, to the Bridge Hydraulics Section.

Example Outline for Bridge Hydraulics Reports

Section 1 Purpose
Section 2 Background
Section 3 Hydraulic Design Considerations
  Section 3.1 Hydrology
  Section 3.1.1 USGS Gage Information
  Section 3.2 Bridge Hydraulics
  Section 3.2.1 Existing Bridge Hydraulics
  Section 3.2.2 Proposed Bridge Hydraulics
  Section 3.2.3 Comparison of Existing vs. Proposed Model
  Section 3.2.4 Deck Drains
  Section 3.2.5 Scour Analysis
  Section 3.2.6 Scour Countermeasure
Section 4 Related Development Considerations
  Section 4.1 Stormwater Pollution Prevention Plan (SWPPP)
Section 4.2 Geotechnical
Section 4.3 Schedule
Section 5 Conclusion and Recommendations
Appendix A FEMA FIRM panel
Appendix B Reach Cross-Section Locations
Appendix C Proposed Bridge Plans
Appendix D Scour Calculations
Appendix E Survey Details

Example Outline of Project Assessment Report for Bridge Retrofit Projects

Section 1 Purpose
Section 2 Background
Section 3 Study of Alternatives
Section 4 Development Considerations
Section 5 Staging, Stockpiling and Access Details
Section 6 Project Schedule
Figure 1 Location of the Project on a quad map
Appendix A Photos
Appendix B Field Review Minutes
Appendix C Preliminary Plans
Chapter 7

STORM DRAINAGE SYSTEMS

This chapter contains details on the following:
- Pavement drainage analysis
- Stormwater inlet design
- Hydraulic design of storm drains

7.1 INTRODUCTION

This chapter provides guidance for design of storm drainage systems. Storm drainage system design includes pavement drainage analysis, stormwater inlet design, and design and analysis of underground storm drain pipe systems.

7.1.1 Design Resources

The design of storm drainage systems is described in FHWA’s Hydraulic Engineering Circular No. 22, Urban Drainage Design Manual (HEC 22, 2009). The following design resources are primary resources for use in design of storm drainage systems:

- **Pipe Selection Guidelines and Procedures**, (ADOT, 1996)
- **Construction Standard Drawings**, (C- Stds) (ADOT, 2012)

Flow in gutters and drainage inlet design are covered in Chapter 4 and storm drain design is covered in Chapter 7 of HEC 22. It is anticipated that designers of storm drainage systems will utilize HEC 22 as a primary resource. Therefore, equations, procedures, and figures from HEC 22 are not be included in this manual.

The procedures for analyzing flow in gutters and computing inlet interception and flow-by contained in HEC 22 have been incorporated into the Hydraulic Toolbox Computer program prepared by FHWA (FHWA, 2014). The Hydraulic Toolbox program does not incorporate methods to evaluate all ADOT standard gutters and inlets. In particular, the Hydraulic Toolbox does not include the capability to compute flows in the urban freeway curb and gutter section and inlet interception for the associated freeway inlet; both of which include a “back-slope” on the curb in lieu of the vertical curb used on many streets. The Hydraulic Toolbox does not have the capability to account for different roughness factors (n-values) for concrete curb and gutter and pavement portions of the roadway section. There is also no provision to account for the reduction in flow area resulting from placement of the 1-inch rubberized asphalt overlay provided on many...
urban freeways. The user is therefore cautioned to apply the Hydraulic Toolbox program with care and only for computations that provide accurate results for ADOT inlets. This is also true for any software application that performs pavement and inlet calculations. The software must be capable of evaluating the details just described.

The Pipe Selection Guidelines and Procedures are to be used to determine pipe materials included in construction plans. A methodology for achieving minimum service life requirements for currently approved pipe materials is provided. The intent is to include the appropriate pipe alternates as a bid condition.

Dimensions and construction details for curb, gutter, and all ADOT standard inlets are contained in the Construction Standard Drawings.

### 7.1.2 System Planning

A storm drainage system for a street or highway is a collection of structures to collect and convey stormwater runoff from land surface areas to a discharge location in a manner that adequately drains the roadway and minimizes the potential for flooding and erosion of adjacent properties.

The system begins with flow concentration elements such as gutters and channels which drain to a series of inlets that collect runoff and pass it to conveyance structures such as pipes or channels. The collected flows are ultimately conveyed to an outfall. The outfall may be a pump station, storage facility, or a means of conveyance, such as a constructed channel or a natural stream or river.

Planning and design of storm drainage systems is an iterative process that begins with establishing a preliminary system layout, locating and designing stormwater inlets, and designing the storm drain pipe system. The general design process is summarized as follows:

#### 7.1.2.1 Preliminary Layout

A useful description of drainage system planning concepts for highway drainage in general and preliminary layout in particular is contained in Chapter 2 of HEC 22. Preliminary layout begins with identification of system outfalls and the drainage areas that drain to them using the project base map. Each outfall and tributary area is designed as a hydraulically independent system.

Once the outfalls are identified, an initial inlet layout is prepared. Inlets will typically be provided at roadway sags and at the curb return of intersections to prevent runoff from crossing intersecting streets. The location of the starting upstream inlet and typical inlet spacing is computed based on pavement hydraulics and spread criteria. The initial inlet layout can then be used to delineate subbasins draining to each inlet. For each subbasin, the component runoff coefficient “C” values and flow paths used to compute the time of concentration can be delineated as well for use in developing the Rational Method hydrology.

Consideration should be given to the ultimate roadway condition when preparing the preliminary layout. Future widening projects or median infill projects can be more easily accommodated if preliminary layout is completed with future improvements in mind. Thoughtful location and
profile of the trunk storm drain and connector pipes may save cost and inconvenience during future improvement projects.

Although the sizing of inlets and pipes will typically be based on the design storm frequency, consideration must also be given to less frequent storm events, sometimes referred to as the “major” storm. Particularly when runoff is diverted into a storm drain system or redirected in any way from its historic flow direction, the consequences of a storm event greater than the storm drain capacity must be considered. Flow paths should be identified for runoff up to the 100-year storm event to ensure that an acceptable flow path is available that avoids damage to the highway or adjacent property.

7.1.2.2 Inlet Design

The initial inlet layout is used as the starting point for inlet design. As the inlet design computations progress, the initial inlet layout will likely change based on the need for additional inlets or changes in inlet spacing as required to meet the pavement spread criteria. To avoid unnecessary redesign, the inlet designs should be refined in this way prior to beginning storm drain pipe layout or design.

The first step in inlet design is preparation of the inlet hydrology using the Rational Method based on the subbasin delineations completed in the preliminary system layout. The Rational Method computations are described in DDM Volume 2 - Hydrology. If the subbasins and supporting linework and shapes are prepared in GIS or CAD software, refinements can be made with minimal effort.

With the hydrology results, the gutter or median flow computations and inlet interception computations can be completed. Pavement drainage and inlet interception computation procedures are presented in Chapter 4 of HEC 22. Inlet design begins with the selection of the inlet type appropriate for the location. The curb and gutter and available inlet types are detailed in the C-Stds. Gutter flow and inlet interception computations should be based on the specific dimensions and details from the C-Stds. Several commercially available computer software programs include gutter and inlet capacity calculation routines. Commercially available software packages should be verified to ensure that the computations are completed correctly for the pavement drainage analysis.

The design progresses from the upstream end of the system in a downstream fashion. For each inlet, the intercepted flow, \( Q_i \), and bypass flow, \( Q_b \), are calculated. The bypass flow is added to the next downstream inlet for its pavement capacity and inlet interception calculation. Inlets can be moved or added as the computations progress to meet the pavement spread/depth criteria. Once the inlet layout has been refined in this way and the pavement and inlet hydraulic computations have been completed such that the spread/depth criteria are met, the layout and design of the underground storm drain pipe system may begin.
7.1.2.3 Storm Drain Design

The storm drain system layout is prepared by identifying the alignment of the trunkline and connector pipes to reach each of the inlets and extending to the system outfall. Selection of the pipe alignments should include consideration of the ultimate roadway condition and future improvements that may be made which could be impacted by the pipe location. For example, future addition of service lanes may require that inlets be moved to a new, relocated gutter location. Thoughtful layout of the pipe system could potentially avoid the need to relocate or replace a trunk line or connector pipe in the future that may be under existing pavement. From the pipe layout, junction locations are identified. Manholes are then positioned at required access locations according to the manhole spacing criteria. Inlets can be connected to the trunkline using tees, wyes, or at manholes.

The storm drain design flows are based on the Rational Method and are determined by accumulating the drainage area, composite C-factor, and time of concentration from each inlet starting at the upstream inlet and proceeding in a downstream fashion. For each reach of storm drain pipe, a single tributary time of concentration, area, and composite C-factor are used to compute the design discharge.

With a design discharge for each reach of pipe, a preliminary pipe size and profile can be developed. This is initially accomplished in the upstream to downstream direction. The system hydraulic grade line (HGL) is then computed. The HGL computations begin at the downstream end of the system at the outlet and proceed in an upstream direction. The starting HGL is based on the outlet conditions and the system energy losses are added in the upstream direction. Losses to be included in the HGL computation are:

- Exit losses,
- Pipe friction losses,
- Bend and transition losses,
- Junction and manhole losses, and
- Inlet losses.

As the HGL computations proceed, adjustments in pipe size or profile may be needed to meet the criteria for HGL location and/or velocity. Once these criteria are met, the initial storm drain design is complete. Further refinements may be required as the roadway design progresses. The final design will likely be the result of an iterative process between the roadway designers and the drainage designer. A comprehensive storm drain design example is contained in Section 7.3.3.

The storm drain design is typically prepared based on a reinforced concrete pipe (RCP) material. The final step in the design process is to determine the pipe structural requirements, specify the required pipe class or D-Load, and identify allowable alternate pipe materials using the ADOT Pipe Selection Guidelines and Procedures. The final storm drain design and allowable alternate materials are incorporated into the project construction plans by completing the “New Pipe
Summary” sheet which can be found under CADD Resources in the Roadway Engineering section of the ADOT website.

7.2 PAVEMENT DRAINAGE

Effective drainage of highway pavements is essential to the maintenance of highway service level and for traffic safety. Water on the pavement can interrupt traffic, reduce skid resistance, increase potential for hydroplaning, limit visibility due to splash and spray, and cause difficulty in steering a vehicle when the front wheels encounter puddles.

Pavement drainage requires consideration of surface drainage, gutter flow, and inlet capacity. The design of these elements is dependent on storm frequency and the allowable spread of stormwater on the pavement surface. This section is based on Chapter 4 in HEC 22 and presents design guidance for the design of pavement drainage systems.

7.2.1 Flow in Gutters

The roadway typical section includes a cross-slope to facilitate quickly clearing rainfall from the pavement surface. The curb and gutter as well as a portion of the available travel lanes are used for stormwater conveyance. However, the inundation areas are limited during the design storm event to maintain safe travel conditions. The design of storm drainage systems is based on limiting flow in the gutter section by clearing stormwater from the roadway using inlets as needed to meet the allowable pavement spread/depth criteria for the design storm.

Gutter flow calculations are necessary to determine the spread of water on the pavement section. A modification of the Manning's equation is used for computing flow in the pavement section. The modification is necessary because the hydraulic radius in the equation does not adequately describe the gutter cross-section, particularly where the top width of the water surface may be more than 40 times the depth at the curb. To compute gutter flow, the Manning's equation is integrated for an increment of width across the section. The resulting equation is presented in Section 4.3 of HEC 22. Procedures are included in HEC 22 for computing the gutter flow for standard and composite gutter sections for a single Manning’s n-value.

In applying the gutter flow equations, ADOT requires the use of separate n-values for the pavement and concrete portions of the cross-section. A concrete n-value of 0.013 and pavement n-value of 0.016 are used. Most ADOT standard roadway sections use composite gutter sections which means the roadway cross-slope is different from the gutter cross-slope.

As part of ADOT’s program to include rubberized asphalt on roadways using an overlay process, a portion of the roadway conveyance area is lost to the rubberized asphalt layer. It may be necessary for gutter flow calculations for existing and new freeway sections to reduce the conveyance area resulting from a rubberized asphalt overlay in spread/depth calculations. A review of the project typical section should define the thickness and extent of any overlay. A typical urban freeway curb and gutter section with asphalt overlay is shown on Figure 7–1. Software applications used for gutter analysis must be capable of evaluating these features.
Figure 7–1  Typical Freeway Curb and Gutter Section With Asphalt Overlay
7.2.2 Pavement Drainage Inlets

The term "inlets" refers to all types of inlets such as grate inlets, curb opening inlets and slotted drain inlets. Grate inlets and depression of curb opening inlets should be located outside the through traffic lanes to minimize the shifting of vehicles attempting to avoid them. All grate inlets shall be bicycle safe when used on roadways that allow bicycle travel. Curb opening inlets are preferred to grate inlets at major sag locations because of their debris handling capabilities. In sag vertical curve locations where significant ponding may occur such as at underpasses or in depressed sections, it is recommended practice to place flanking inlets on each side of the inlet at the low point in the sag. Due to difficulties handling large debris, curb opening inlets shall not be used when the collected stormwater will drain to a pump station.

7.2.2.1 Inlet Locations

Inlets are required at locations needed to collect runoff within the design controls specified in the design criteria. In addition, there are locations where inlets may be necessary due to geometric considerations. These locations should be identified as part of the preliminary layout. Examples of such locations are as follows:

- Sag points in the gutter grade,
- Flanking inlets at sag points or where clogging may result in safety concerns,
- Upstream of median breaks, entrance/exit ramp gores, cross walks and street intersections,
- Upstream of bridges,
- Upstream of cross slope reversals,
- On side streets at intersections, and
- Behind curbs, shoulders, or sidewalks to drain low areas.

Inlets should not be located in the path pedestrians are likely to walk.

7.2.2.2 Inlet Types

Various types of inlets are in use; grates, curb openings and slotted drain. Curb opening inlets and slotted drain inlets are also used in combination with grate inlets. Grate inlets are used at the downstream end of curb opening and slotted drain inlets to provide maintenance access and increase the interception capacity. Combination inlets are desirable in sags because they can provide relief capacity in the event of plugging of the grate inlet. Figure 3–2 shows typical inlet types.

Grate Inlets

These inlets consist of an opening in the gutter covered by one or more grates. On continuous grades they are most efficient in capturing frontal flow. In sag locations, they are more susceptible to clogging with debris. The use of standard grate inlets at sag points should be limited to minor sag point locations without debris potential. Special design (oversize) grate inlets or grate with curb opening can be utilized at major sag points if sufficient capacity is provided for...
clogging. In this case, flanking inlets are recommended. Grates shall be bicycle safe unless bike traffic is specifically excluded.

Curb-Opening Inlets

These inlets are vertical openings in the curb. They are best suited for use at sag points since they can convey large quantities of water and debris. They are a viable alternative to grates in many locations where grates are not desired due to pedestrian or bicycle traffic. They are not very efficient on steep continuous grades. They shall not be used with pump station collection systems.

Slotted Drain Inlets

These inlets consist of a vertical slot opening in the gutter with bars perpendicular to the slot opening. For shallow flow slotted inlets function as weirs with flow entering from the side. They are used to intercept sheet flow and collect gutter flow with or without curbs. Grate inlets must be used at the downstream end of slotted drain inlets to provide access and reduce the overall length of slotted drain. Slotted drain inlets shall not be used for offsite collection nor where snow and ice are anticipated (above elevation 4000 ft. +/-).

Figure 7–2 Typical Inlet Types (HEC 22, 2009)
7.2.2.3 Inlet Clogging with Debris

To account for a potential reduction of inflow capacity due to inlet clogging with debris that is frequently carried in the gutter section, the design length, perimeter or area, as appropriate, of inlets shall be calculated using the effective area ratios shown in DDM Volume 1 - Policies and Guidelines. Effective area ratio is the ratio of the unclogged opening to the total opening. An example application of inlet sizing considering clogging with an effective area ratio of 0.5 for a grate inlet in a sag condition is illustrated in Example 4-11 of HEC 22.

7.2.3 Gutter Flow and Inlet Design Example

An example of gutter flow and inlet design calculation are contained in Appendix 7A. The two example problems are to be completed in order and include the following computations:

- Consideration of different n-values for pavement and concrete,
- Spread calculation upstream of inlet for a compound gutter section with rubberized asphalt overlay,
- Application of clogging factor (capture ratio), and
- Consideration of the grate area in the curb section (back slope).

7.3 STORM DRAIN DESIGN

Key considerations in the design of storm drains are:

- Computation of design discharge using the Rational Method to account for inlet flows,
- Properly accounting for all energy losses in the system,
- Ensuring that the hydraulic grade line remains sufficiently below ground, and
- Ensuring that structural requirements are met through ground cover and pipe strength.

The Rational Method for computing design discharges is described in DDM Volume 2 - Hydrology. The specific application of the Rational Method for storm drain design is described in Section 7.3.2 and illustrated in the design example in Section 7.3.3. Computation of the energy losses and hydraulic grade line is introduced in Section 7.3.1 below and described in detail in Section 7.1 of HEC 22 and illustrated in the Section 7.3.3 example. Structural requirements for storm drains are addressed in Pipe Selection Guidelines and Procedures.

7.3.1 Hydraulics of Storm Drain Systems

The hydraulic design of storm drains is based on conveying the design discharge through the pipe system while keeping the hydraulic grade line sufficiently below manhole rims and inlets to avoid stormwater spilling out of the system or backing up into an inlet which would restrict the free-flowing capacity of the inlet. The hydraulic grade line computation is simply an accounting of energy that begins at the system outlet at the controlling downstream condition and adding the various energy loss components in an upstream direction. The friction loss within the pipe is commonly thought of as the primary means of energy loss with “minor” losses from manholes,
curves, junctions, and others being additive to the friction losses. It is important to understand that in many cases these “minor” losses are not really minor and can be quite large. Careful analysis of these losses and attention to detail in design of the system may result in meaningful cost savings.

7.3.1.1 Inlet and Manhole Losses

HEC 22 provides methods for estimating inlet and manhole losses. The method is referred to as the “FHWA Access Hole Method” and should be used as described in Section 7.1.6.7 of HEC 22.

7.3.2 Design Procedure

The storm drain design and analysis procedure is completed in two phases. The system is first designed from upstream to downstream to establish design discharges and preliminary pipe sizes. The system is then analyzed from downstream to upstream to establish the hydraulic grade line profile and finalize the design. The steps required to establish design discharges and preliminary pipe sizes are summarized below. A Preliminary Storm Drain Computation Sheet is provided in HEC 22 for use in documenting manual computations. A similar format may be created in a spreadsheet to automate the computations.

7.3.2.1 Initial Design Process

**Step 1** Determine inlet location and spacing as outlined in Section 7.2.

**Step 2** Prepare plan layout of the storm drainage system establishing the following design data:
- Location of storm drains.
- Direction of flow.
- Location of access holes.
- Location of existing utilities such as water, gas, or underground cables.

**Step 3** Set a preliminary slope between the outfall and the upstream end of the system. On a profile, plot the outfall elevations, and the invert elevation of any inlet that would control the invert elevations of the storm drain.

**Step 4** From the inlet design get the drainage areas and runoff coefficient. For the first inlet in the system get the time of concentration to the first inlet. Using an Intensity-Duration-Frequency (IDF) curve, determine the rainfall intensity. Calculate the discharge by $Q = CiA$.

**Step 5** Size the pipe to convey the discharge assuming full flow by varying the slope and pipe size as necessary. The full flow velocity is computed as $Q/A$.

**Step 6** Calculate travel time in the pipe to the next downstream inlet or access hole by dividing pipe length by the velocity. This travel time is added to the time of
concentration for a new time of concentration and a new rainfall intensity at the next entry point.

**Step 7** At the next inflow point, calculate the additional CA, the sub area (A) multiplied by the subarea runoff coefficient (C), then add to the previous CA. Multiply the total CA by the rainfall intensity at the computed time of concentration to determine the new discharge. If the local CA is large, compare this discharge with the CA for the subbasin and the rainfall intensity using the inlet time of concentration. Use the larger of the two discharges. Determine the size of pipe and slope necessary to convey the discharge.

**Step 8** Continue this process to the storm drain outlet. Utilize the equations and/or nomographs to accomplish the design effort.

**Step 9** Complete the design by calculating the hydraulic grade line as described in HEC 22.

### 7.3.2.2 HGL Calculation Procedure

A step-by-step procedure to manually calculate the location of the hydraulic grade line, HGL, is presented in Section 7.5 of HEC 22. The procedure is tied to two provided computation sheets; the Table A sheet is used to tabulate the EGL and HGL computations for the system, and the Table B sheet is used to tabulate the exit loss, pipe losses, and structure losses. The resulting loss values from Table B are transferred to Table A as total loss values to be used in the Table A computations.

### 7.3.3 Storm Drain Design Example: Inundation Boulevard

The storm drain design methods are best illustrated using an example problem as presented in Appendix 7B. The example problem illustrates the following:

- Determine the design discharge and initial size for each pipe.
- Determine the junction losses for the initial system design.
- Determine the hydraulic grade line for the initial system design.

### 7.4 REFERENCES


APPENDIX 7A

GUTTER FLOW AND INLET DESIGN EXAMPLES
Example No. 7-1  Compound Gutter Flow

Example 7-1 illustrates the computation of gutter flow in the urban freeway curb and gutter section with consideration of different $n$-values for asphalt pavement and concrete gutter for the following conditions:

- Roadway capacity without a rubberized asphalt overlay.
- Roadway capacity with a 1-inch rubberized asphalt overlay.
- Spread calculation with a 1-inch rubberized asphalt overlay.

The variable terms and applicable equations are summarized first, followed by the problem statement and completed solution for each condition.

![Gutter Cross-section 1](image1)

**Figure 7–3  Gutter Cross-section 1**

![Gutter Cross-section 2](image2)

**Figure 7–4  Gutter Cross-section 2**
List of Equations

\[ Q_A = \frac{0.56 \sqrt{S_L}}{n_G} \left( \frac{d_1^{8/3}}{S_{x1}} \right) \] 7.1

\[ Q_B = \frac{0.56 \sqrt{S_L}}{n_G} \left( \frac{d_1^{8/3} - (d_1 - d_2)^{8/3}}{S_{x2}} \right) \] 7.2

\[ Q_C = \frac{0.56 \sqrt{S_L}}{n_R} \left( \frac{(d_1 - d_2 - d_3)^{8/3}}{S_{x3}} \right) \] 7.3

\[ Q_D = \frac{0.56 \sqrt{S_L} d_1^{8/3}}{n_G} \left( \frac{1}{S_{x1}} + \frac{1}{S_{x2}} \right) \] 7.4

where:

- \( A_A \) Flow area as defined in Figure 7–3 (ft²),
- \( A_B \) Flow area as defined in Figure 7–3 (ft²),
- \( A_C \) Flow area as defined in Figure 7–3 (ft²),
- \( A_D \) Flow area as defined in Figure 7–4 (ft²),
- \( Q_A \) Flow in area \( A_A \) (cfs),
- \( Q_B \) Flow in area \( A_B \) (cfs),
- \( Q_C \) Flow in area \( A_C \) (cfs),
- \( Q_D \) Flow in area \( A_D \) (cfs),
- \( Q_{Total} \) Total flow (cfs),
- \( V_R \) Velocity in area \( A_A \) (ft/sec),
- \( H_C \) Curb height (ft),
- \( W_C \) Curb width (ft),
- \( S_{x1} \) Curb backslope (ft/ft),
- \( S_{x2} \) Gutter foreslope (ft/ft),
- \( S_{x3} \) Roadway cross-slope (ft/ft),
- \( S_L \) Roadway longitudinal slope (ft/ft),
- \( n_G \) Roughness coefficient for gutter
- \( n_R \) Roughness coefficient for roadway
- \( d_1 \) Flow depth measured from the gutter invert (ft),
- \( d_2 \) Gutter depression (ft),
- \( d_3 \) Roadway overlay thickness (ft), and
- \( T \) Spread (ft).
Three Possible Cases

**Case 1**

\[ d_1 \leq d_2 \text{ (with or without overlay)} \]

\[ Q_{\text{Total}} = Q_D \]

\[ T = \frac{d_1}{S_{x2}} \]

**Case 2**

\[ d_1 > (d_2 + d_3) \text{ (with or without overlay)} \]

\[ Q_{\text{Total}} = Q_A + Q_B + Q_C \]

\[ T = \frac{d_2}{S_{x2}} + \frac{d_1 - d_2 - d_3}{S_{x3}} \]

**Case 3**

\[ d_2 < d_1 \leq d_2 + d_3 \text{ (with overlay)} \]

\[ Q_{\text{Total}} = Q_A + Q_B \]

\[ T = \frac{d_2}{S_{x2}} \]
Problem:
For the compound gutter without overlay, determine the discharge in an ADOT Type E Urban Freeway Curb and Gutter assuming a maximum allowable spread of 10 feet.

Given:
Use the following parameters:
- Curb height \((H_C) = 4\) in
- Overlay thickness \((d_3) = 0\)
- Curb Width \((W_C) = 2\) ft
- Roadway slope \((S_x3) = 0.02\) ft/ft
- Curb Slope \((S_x1) = 6:1 = 0.1667\)
- Longitudinal slope \((S_L) = 0.018\) ft/ft
- Gutter Depression \((d_2) = 5/8\) in
- Gutter roughness \((n_G) = 0.013\)
- Roadway roughness \((n_R) = 0.016\)
- C-05.10 Type E Gutter Width = 2.5 ft

Solution:
1. Calculate gutter foreslope \((S_x2)\).
   \[
   S_{x2} = \frac{d_2}{2.5\text{ ft}} = \frac{0.05208\text{ ft}}{2.5\text{ ft}} = 0.02083\text{ ft/ft}
   \]
   Assume Case 1, and calculate \(d_1\). Compare calculated depth to gutter depression to check result.
   \[
   T = \frac{d_1}{S_{x2}} \rightarrow d_1 = S_{x2}T = 0.02083(10) = 0.2083\text{ ft}
   \]
   Compare depth to gutter and overlay depth, \(d_2\) and \(d_3\):
   \[0.2083\text{ ft} > (0.05208 + 0) \text{ ft}\]
   \[0.2083\text{ ft} > 0.05208\text{ ft}\]
   Therefore problem satisfies Case 2.

2. Calculate maximum allowable depth based on maximum spread criteria.
   \[
   T = \frac{d_2}{S_{x2}} + \frac{d_1 - d_2 - d_3}{S_{x3}} \rightarrow \\
   d_1 = d_2 + d_3 + S_{x3}\left(T - \frac{d_2}{S_{x2}}\right) = 0.05208\text{ft} + 0 + 0.02\left(10\text{ft} - \frac{0.05208\text{ft}}{0.02083}\right) \\
   d_1 = 0.202\text{ ft}
   \]
   Problem satisfies Case 2, so total discharge \((Q_{\text{Total}})\) will be calculated using Equations 7.1, 7.2 and 7.3.
   \[
   Q_A = \frac{0.56\sqrt{S_L}}{n_G}\left(\frac{d_1^{8/3}}{S_{x1}}\right) = \frac{0.56\sqrt{0.018}}{0.013}\left(\frac{0.202^{8/3}}{0.1667}\right)
   \]
   \[Q_A = 0.49\text{ cfs}\]
\[ Q_B = \frac{0.56\sqrt{S_L}}{n_G} \left( \frac{d_1^{8/3} - (d_1 - d_2)^{8/3}}{S_{x_2}} \right) \]

\[ Q_B = \frac{0.56\sqrt{0.018}}{0.013} \left( \frac{0.202^{8/3} - (0.202 - 0.0528)^{8/3}}{0.02083} \right) \]

\[ Q_B = 2.14 \text{ cfs} \]

\[ Q_C = \frac{0.56\sqrt{S_L}}{n_R} \left( \frac{(d_1 - d_2 - d_3)^{8/3}}{S_{x_3}} \right) = \frac{0.56\sqrt{0.018}}{0.016} \left( \frac{(0.202 - 0.0528 - 0)^{8/3}}{0.02} \right) \]

\[ Q_C = 1.49 \text{ cfs} \]

\[ Q_{\text{Total}} = Q_A + Q_B + Q_C = 0.49 + 2.14 + 1.49 = 4.12 \text{ cfs} \]
Problem:
For compound gutter with overlay, determine the discharge in an ADOT Type E Urban Freeway Curb and Gutter assuming a maximum allowable spread of 10 feet.

Given:
Use the following parameters:
- Curb height \( H_C \) = 4 in
- Overlay thickness \( d_3 \) = 1 in
- Curb Width \( W_C \) = 2 ft
- Roadway slope \( S_{x3} \) = 0.02 ft/ft
- Curb Slope \( S_{x1} \) = 6:1 = 0.1667
- Longitudinal slope \( S_L \) = 0.018 ft/ft
- Gutter Depression \( d_2 \) = 5/8 in
- Gutter roughness \( n_G \) = 0.013
- Roadway roughness \( n_R \) = 0.016
- C-05.10 Type E Gutter Width = 2.5 ft

Solution:
1. Calculate gutter foreslope \( S_{x2} \).
   \[
   S_{x2} = \frac{d_2}{2.5 \text{ ft}} = \frac{0.05208 \text{ ft}}{2.5 \text{ ft}} = 0.02083 \text{ ft/ft}
   \]
   Assume Case 1, and calculate \( d_1 \). Compare calculated depth to gutter depth to check result.
   \[
   T = \frac{d_1}{S_{x2}} \rightarrow d_1 = S_{x2} T = 0.02083(10) = 0.2083 \text{ ft}
   \]
   Compare depth to gutter and overlay depth, \( d_2 \) and \( d_3 \):
   \[
   0.2083 \text{ ft} > (0.05208 + 0.0833) \text{ ft}
   \]
   \[
   0.2083 \text{ ft} > 0.135 \text{ ft}
   \]
   Therefore problem satisfies Case 2.

   Calculate maximum allowable depth based on maximum spread criteria.
   \[
   T = \frac{d_2}{S_{x2}} + \frac{d_1 - d_2 - d_3}{S_{x3}} \rightarrow \\
   d_1 = d_2 + d_3 + S_{x3} \left( T - \frac{d_2}{S_{x2}} \right) = 0.05208\text{ft} + 0.0833\text{ft} + 0.02 \left( 10\text{ft} - \frac{0.05208\text{ft}}{0.02083} \right) \\
   d_1 = 0.285 \text{ ft}
   \]

   Problem satisfies Case 2, so discharge will be calculated using Equations 7.1, 7.2 and 7.3.
   \[
   Q_A = \frac{0.56\sqrt{S_L}}{n_G} \left( \frac{d_1^{8/3}}{S_{x1}} \right) = \frac{0.56\sqrt{0.018}}{0.013} \left( \frac{0.285^{8/3}}{0.1667} \right)
   \]
   \[
   Q_A = 1.21 \text{ cfs}
   \]
\[ Q_B = \frac{0.56\sqrt{S_L}}{n_G} \left( \frac{d_1^{8/3} - (d_1 - d_2)^{8/3}}{S_{x2}} \right) \]

\[ Q_B = \frac{0.56\sqrt{0.018}}{0.013} \left( \frac{0.285^{8/3} - (0.285 - 0.05208)^{8/3}}{0.02083} \right) \]

\[ Q_B = 4.06 \text{ cfs} \]

\[ Q_C = \frac{0.56\sqrt{S_L}}{n_R} \left( \frac{(d_1 - d_2 - d_3)^{8/3}}{S_{x3}} \right) \]

\[ Q_C = \frac{0.56\sqrt{0.018}}{0.016} \left( \frac{(0.285 - 0.05208 - 0.0833)^{8/3}}{0.02} \right) \]

\[ Q_C = 1.48 \text{ cfs} \]

\[ Q_{Total} = Q_A + Q_B + Q_C = 1.21 + 4.06 + 1.48 = 6.76 \text{ cfs} \]
Problem:
For compound gutter with overlay, determine the spread associated with 8 cfs in an ADOT Type E Urban Freeway Curb and Gutter.

Given:
Use the following parameters:

- Curb height (H_C) = 4 in
- Curb width (W_C) = 2 ft
- Curb slope (S_{x1}) = 6:1 = 0.1667
- Gutter depression (d_2) = 5/8 in
- Gutter roughness (n_G) = 0.013
- Roadway roughness (n_R) = 0.016
- Roadway slope (S_{x3}) = 0.02 ft/ft
- Longitudinal slope (S_L) = 0.018 ft/ft
- Overlay thickness (d_3) = 1 in
- C-05.10 Type E Gutter Width = 2.5 ft

Solution:
1. Calculate gutter foreslope (S_{x2}).
   \[
   S_{x2} = \frac{d_2}{2.5 \text{ ft}} = \frac{0.05208 \text{ ft}}{2.5 \text{ ft}} = 0.02083 \text{ ft/ft}
   \]
   Assume Case 1, and solve for depth
   \[
   Q_{Total} = Q_D = \frac{0.56\sqrt{S_L d_1^{8/3}}}{n_G} \left( \frac{1}{S_{x1}} + \frac{1}{S_{x2}} \right) = 8.0
   \]
   \[
   \frac{0.56\sqrt{0.018d_1^{8/3}}}{0.013} \left( \frac{1}{0.1667} + \frac{1}{0.02083} \right)
   \]
   \[
   d_1 = 0.253 \text{ ft}
   \]
   Compare depth to gutter and overlay depth, d_2 and d_3:
   \[
   0.253 \text{ ft} > (0.05208 + 0.0833) \text{ ft}
   \]
   \[
   0.253 \text{ ft} > 0.135 \text{ ft}
   \]
   Since d_1 > (d_2 + d_3), problem satisfies Case 2.

2. Set up series of equations relating discharge to depth.
   \[
   Q_{Total} = Q_A + Q_B + Q_C = 8.0 \text{ cfs}
   \]
   \[
   Q_A = \frac{0.56\sqrt{S_L d_1^{8/3}}}{n_G} \left( \frac{d_1^{8/3}}{S_{x1}} \right) = 0.56\sqrt{0.018} \left( \frac{d_1^{8/3}}{0.1667} \right)
   \]
   \[
   Q_B = \frac{0.56\sqrt{S_L}}{n_G} \left( \frac{d_1^{8/3} - (d_1 - d_2)^{8/3}}{S_{x2}} \right) = 0.56\sqrt{0.018} \left( \frac{d_1^{8/3} - (d_1 - 0.05208)^{8/3}}{0.02083} \right)
   \]
\[ Q_C = \frac{0.56 \sqrt{S_L}}{n_R} \left( \frac{(d_1 - d_2 - d_3)^{6/3}}{S_{x3}} \right) = \frac{0.56 \sqrt{0.018}}{0.016} \left( \frac{(d_1 - 0.05208 - 0.0833)^{6/3}}{0.02} \right) \]

Solve for \( d_1 \) using successive approximations of \( d_1 \) above to satisfy the above series of equations.

\( d_1 = 0.303 \text{ ft} \)

Calculate spread based using \( d_1 \).

\[ T = \frac{d_2}{S_{x2}} + \frac{d_1 - d_2 - d_3}{S_{x3}} = \frac{0.05208}{0.02083} + \frac{0.303 - 0.05208 - 0.0833}{0.02} = 10.88 \text{ ft} \]
Example No. 7-2  Inlet Interception Example

Example 7-2 illustrates the computation of inlet interception in the urban freeway curb and gutter section with a grated inlet including consideration of the additional capacity in the curb section (back slope) of the grate and application of a clogging factor.

The variable terms are summarized first, followed by the problem statement and completed solution.

![Diagram of Gutter Cross-section 3]

**Figure 7–5 Gutter Cross-section 3**
List of Variables:

- \( A_1 \): Flow area as defined in Figure 7–5 (ft\(^2\))
- \( A_2 \): Flow area as defined in Figure 7–5 (ft\(^2\))
- \( A_3 \): Flow area as defined in Figure 7–5 (ft\(^2\))
- \( A_{2,3} \): Flow area for \( A_2 \) and \( A_3 \) (ft\(^2\))
- \( A_{G,L} \): Flow area above left side of inlet grate (ft\(^2\))
- \( A_{G,R} \): Flow area above right side of inlet grate (ft\(^2\))
- \( A_G \): Flow area above the grate, equal to \( A_{G,L} + A_{G,R} \) (ft\(^2\))
- \( Q_1 \): Flow in area \( A_1 \) (cfs)
- \( Q_2 \): Flow in area \( A_2 \) (cfs)
- \( Q_3 \): Flow in area \( A_3 \) (cfs)
- \( Q_{2,3} \): Flow in areas \( A_2 \) and \( A_3 \) (cfs)
- \( Q_{G,L} \): Flow above the left side of inlet grate (cfs)
- \( Q_{G,R} \): Flow above the right side of inlet grate (cfs)
- \( Q_G \): Flow in area \( A_G \) (cfs)
- \( Q_i \): Interception flow (cfs)
- \( Q_r \): By-pass flow (cfs)
- \( Q_{\text{Total}} \): Total flow (cfs)
- \( V_1 \): Velocity in area \( A_1 \) (ft/sec)
- \( V_{2,3} \): Average velocity in areas \( A_2 \) and \( A_3 \) (ft/sec)
- \( V_G \): Velocity in area \( A_G \) (ft/sec)
- \( V_o \): Splash over velocity (ft/sec)
- \( E_0 \): Ratio of frontal flow to total gutter flow
- \( E_1 \): Ratio of \( Q_1 \) to total gutter flow
- \( E_2 \): Ratio of \( Q_2 \) to total gutter flow
- \( E_3 \): Ratio of \( Q_3 \) to total gutter flow
- \( E_{2,3} \): Ratio of \( Q_{2,3} \) to total gutter flow
- \( R_f \): Ratio of intercepted frontal flow to total frontal flow
- \( R_1 \): Ratio of \( Q_1 \) that is intercepted as side flow
- \( R_2 \): Ratio of \( Q_2 \) that is intercepted as side flow
- \( R_3 \): Ratio of \( Q_3 \) that is intercepted as side flow
- \( R_{2,3} \): Ratio of \( Q_{2,3} \) that is intercepted as side flow
- \( S_e \): Effective cross-slope (ft/ft)
- \( W_{G,L} \): Grate left width (ft)
- \( W_{G,R} \): Grate right width (ft)
- \( L \): Grate length (ft)
- \( L_e \): Effective grate length based on clogging factor
- \( CF \): Clogging Factor, where a value of 1 represents no clogging, and a value of 0 represents full obstruction
Problem:

Determine the amount of flow intercepted by ADOT C-15.91 grate and the amount of flow-by in an ADOT Type E Urban Freeway Curb and Gutter.

Given:

Use the following properties:

- Curb height \( H_C \) = 4 in
- Curb Width \( W_C \) = 2 ft
- Curb Slope \( S_{x1} \) = 6:1 = 0.1667
- Gutter Depresssion \( d_2 \) = 5/8 in
- Roadway roughness \( n_R \) = 0.016
- Overlay thickness \( d_3 \) = 1 in
- Roadway slope \( S_{x3} \) = 0.02 ft/ft
- Longitudinal slope \( S_L \) = 0.018 ft/ft
- Gutter roughness \( n_G \) = 0.013

Grate Properties:

- Grate left width \( W_{G,L} \) = 12 in
- Grate Length \( L \) = 42 in
- Grate right width \( W_{G,R} \) = 24 in
- Clogging Factor \( CF \) = 0.5

Calculated from Gutter Flow Problem:

- Gutter foreslope \( S_{x2} \) = 0.02083 ft/ft
- Flow depth \( d_1 \) = 0.285 ft

\[
Q_{total} = 6.76 \text{ cfs} \\
Q_A = 1.21 \text{ cfs} \\
Q_B = 4.06 \text{ cfs} \\
Q_C = 1.48 \text{ cfs}
\]

Solution:

1. Calculate flow area for sections \( A_A \) and \( A_B + A_C \) as defined in Figure 7-3.

\[
A_A = \frac{d_1^2}{2S_{x1}} = \frac{0.285^2}{2(0.1667)} = 0.244 \text{ ft}^2
\]

\[
A_B = \frac{d_2^2}{2S_{x2}} + \frac{(d_1 - d_2)d_2}{S_{x2}} = \frac{0.05208^2}{2(0.02083)} + \frac{(0.285 - 0.05208)0.05208}{0.02083} = 0.647 \text{ ft}^2
\]

\[
A_C = \frac{(d_1 - d_2 - d_3)^2}{S_{x3}} = \frac{(0.285 - 0.05208 - 0.0833)^2}{0.02} = 1.12 \text{ ft}^2
\]
2. Calculate flow areas directly above and beside inlet grate.

\[ A_{G,L} = W_{G,L}(d_1 - W_{G,L}S_{x1} + 0.5W_{G,L}S_{x1}) = 1(0.285 - 1(0.1667) + 0.5(1)(0.1667)) \]

\[ A_{G,L} = 0.202 \text{ ft}^2 \]

\[ A_{G,R} = W_{G,R}(d_1 - W_{G,R}S_{x2} + 0.5W_{G,R}S_{x2}) \]

\[ A_{G,R} = 2(0.285 - 2(0.02083) + 0.5(2)(0.02083)) \]

\[ A_{G,R} = 0.528 \text{ ft}^2 \]

\[ A_1 = A_A - A_{G,L} = 0.244 - 0.202 = 0.0420 \text{ ft}^2 \]

\[ A_2 = A_B - A_{G,R} = 0.647 - 0.528 = 0.119 \text{ ft}^2 \]

\[ A_3 = A_C = 1.12 \text{ ft}^2 \]

3. Calculate flow \((Q_G)\) and the ratio of flow \((E_0)\) to total gutter flow directly above inlet grate.

Using **Equation 7.2**, where \(d_2 = W_{G,L}S_{x1}\)

\[ Q_{G,L} = \frac{0.56\sqrt{S_L}}{n_G} \left[ d_1^{8/3} - \left( d_1 - W_{G,L}S_{x1} \right)^{8/3} \right] \frac{S_{x1}}{S_{x1}} \]

\[ Q_{G,L} = 0.56\sqrt{0.018} \left( 0.285^{8/3} - (0.285 - 1 \times 0.1667)^{8/3} \right) \]

\[ Q_{G,L} = 1.10 \text{ cfs} \]

Using **Equation 7.2**, where \(d_2 = W_{G,R}S_{x2}\)

\[ Q_{G,R} = \frac{0.56\sqrt{S_L}}{n_G} \left[ d_1^{8/3} - \left( d_1 - W_{G,R}S_{x2} \right)^{8/3} \right] \frac{S_{x2}}{S_{x2}} \]

\[ Q_{G,R} = 0.56\sqrt{0.018} \left( 0.285^{8/3} - (0.285 - 2 \times 0.02083)^{8/3} \right) \frac{0.1667}{0.208} \]

\[ Q_{G,R} = 3.36 \text{ cfs} \]
\[ Q_G = Q_{G,L} + Q_{G,R} = 1.10 + 3.36 = 4.46 \text{ cfs} \]
\[ E_0 = \frac{Q_G}{Q_{\text{Total}}} = \frac{4.46 \text{ cfs}}{6.76 \text{ cfs}} = 0.660 \]

4. Calculate flow in bounding areas, \( A_1 \), \( A_2 \), and \( A_3 \), as well as the ratio of both flows (i.e., \( Q_1 \) and \( Q_{2,3} \)) to the total flow.

\[ Q_1 = Q_A - Q_{G,L} = 1.21 - 1.10 = 0.107 \text{ cfs} \]
\[ Q_2 = Q_B - Q_{G,R} = 4.06 - 3.36 = 0.704 \text{ cfs} \]
\[ Q_3 = Q_C = 1.48 \text{ cfs} \]

\[ E_1 = \frac{Q_1}{Q_{\text{Total}}} = \frac{0.107 \text{ cfs}}{6.76 \text{ cfs}} = 0.0159 \]
\[ E_{2,3} = \frac{Q_{2,3}}{Q_{\text{Total}}} = \frac{(0.704 + 1.48) \text{ cfs}}{6.76 \text{ cfs}} = 0.323 \]

5. Calculate the flow velocity in the gutter as well as in \( A_1 \) and \( A_{2,3} \) (as defined in Figure 7–5).

\[ V_G = \frac{Q_G}{A_{G,L} + A_{G,R}} = \frac{4.46 \text{ cfs}}{(0.202 + 0.528) \text{ ft}^2} = 6.11 \text{ ft/s} \]
\[ V_1 = \frac{Q_1}{A_1} = \frac{0.107 \text{ cfs}}{0.0420 \text{ ft}^2} = 2.55 \text{ ft/s} \]
\[ V_{2,3} = \frac{Q_{2,3}}{A_{2,3}} = \frac{(0.704 + 1.48) \text{ cfs}}{(0.119 + 1.12) \text{ ft}^2} = 1.76 \text{ ft/s} \]

Use Figure 7–6 to determine the splash over velocity of the C-15.91 grate, and compare to the gutter velocity to determine the ratio of frontal flow intercepted to total frontal flow, \( R_f \).

\[ L_e = CF \times L = 0.5 \times 3.5 \text{ ft} = 1.75 \text{ ft} \]

Based on Figure 7–6, assuming a P – 1 7/8 grate, \( V_0 = 7.6 \text{ ft/s} \).

\[ R_f = 1 - 0.09(V_G - V_0) = 1 - 0.09(6.11 - 7.6) = 1.15 \rightarrow R_f = 1.0 \]

\( R_f \) cannot be greater than 1.0

6. Calculate the ratio of side flow intercepted by the inlet grate, \( R_1 \) and \( R_{2,3} \).
\[ R_1 = \left( 1 + \frac{0.15V_1^{1.8}}{S_{x1}(L \times CF)^{2.3}} \right)^{-1} = \left( 1 + \frac{0.15(2.55)^{1.8}}{0.1667(3.5 \times 0.5)^{2.3}} \right)^{-1} = 0.427 \]

The ratio of side flow intercepted from the roadway and right gutter, \( R_{2,3} \), is a function of the equivalent cross slope, \( S_e \) (modified from Equation 4-24 in HEC-22). This value is a function of both the immediate cross slope adjacent to the inlet, \( S_{x2} \), the cross slope of the roadway, \( S_{x3} \) and the ratio of flows conveyed by both sections, \( Q_2 \) and \( Q_3 \).

\[ S_e = S_{x3} + (S_{x2} - S_{x3}) \left( \frac{Q_2}{Q_2 + Q_3} \right) \]
\[ S_e = 0.02 + (0.02083 - 0.02) \left( \frac{0.704}{0.704 + 1.48} \right) \]
\[ S_e = 0.0203 \]

\[ R_{2,3} = \left( 1 + \frac{0.15V_{2,3}^{1.8}}{S_e(L \times CF)^{2.3}} \right)^{-1} = \left( 1 + \frac{0.15(1.76)^{1.8}}{0.0203(3.5 \times 0.5)^{2.3}} \right)^{-1} = 0.150 \]

7. Calculate the interception capacity of the grate.

\[ Q_i = Q_{\text{Total}}(R_{f}E_0 + R_1E_1 + R_{2,3}E_{2,3}) \]
\[ Q_i = 6.76 \left( 1.0(0.660) + 0.427(0.0159) + 0.150(0.323) \right) = 4.83 \text{ cfs} \]

8. Calculate the flow-by discharge.

\[ Q_r = Q_{\text{Total}} - Q_i = (6.76 - 4.83) \text{ cfs} = 1.93 \text{ cfs} \]
Figure 7–6 Grate Inlet Frontal Flow Interception Efficiency (HEC 22, 2009)
APPENDIX 7B
STORM DRAIN DESIGN EXAMPLE
Example No. 7-3  Storm Drain along ADOT Boulevard

Problem:

Determine the appropriate pipe sizes and profile for the system. Evaluate the HGL and EGL.

Use concrete pipe with a Manning’s n of 0.013, a minimum pipe diameter of 18-inches.

Given:

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<thead>
<tr>
<th>Structure</th>
<th>Type</th>
<th>D/S Pipe</th>
<th>Pipe Slope (ft/ft)</th>
<th>Length (ft)</th>
<th>D/S Invert (ft)</th>
<th>U/S Invert (ft)</th>
<th>Drainage Area (ac)</th>
<th>C</th>
<th>Tc (min)</th>
</tr>
</thead>
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<tr>
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<td>Tee</td>
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<td>0.005</td>
<td>100</td>
<td>101</td>
<td>101.5</td>
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<td></td>
<td></td>
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<td>0.005</td>
<td>140</td>
<td>101.5</td>
<td>102.2</td>
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<td></td>
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<td>105.43</td>
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</tr>
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<td>5</td>
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<td>5</td>
<td>0.0145</td>
<td>280</td>
<td>105.93</td>
<td>109.99</td>
<td></td>
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<td></td>
</tr>
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<td>6</td>
<td>Manhole</td>
<td>7</td>
<td>0.01</td>
<td>125</td>
<td>106.43</td>
<td>107.68</td>
<td></td>
<td></td>
<td></td>
</tr>
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<td>Manhole</td>
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<td>0.01</td>
<td>250</td>
<td>108.18</td>
<td>111.68</td>
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<td></td>
</tr>
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<td>Inlet Headwall</td>
<td>6</td>
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<td>10</td>
<td>109.99</td>
<td>110.09</td>
<td>13.54</td>
<td>0.59</td>
<td>18</td>
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<td>Drop Inlet</td>
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<td>0.01</td>
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Table 7-2  Rainfall Data

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**Solution:**

The system is first designed from upstream to downstream to establish design discharges and preliminary pipe sizes. The system layout and preliminary pipe data are in Table 7–1, Table 7–2 and Figure 7–7. The Preliminary Storm Drain Computation sheet is used to record the design computations. The computations for each pipe are recorded in the Preliminary Storm Drain Computation sheet (See Table 7–3, Table 7–4, Table 7–5) and described in detail below.
### Table 7–3  Preliminary Storm Drain Computation

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<th>Velocity</th>
<th>Vehicap</th>
<th>Full Design</th>
<th>Sec Time</th>
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<th>&quot;C&quot;</th>
<th>Pipe Dia.</th>
<th>Q</th>
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<th>&quot;Area&quot; X &quot;C&quot;</th>
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<td>(ft)</td>
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<td>(ft)</td>
<td>(ft)</td>
<td>(min)</td>
<td>(min)</td>
<td>(in/hr)</td>
<td>(cfs)</td>
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**Notes:**
- "Str. ID" refers to the_street identification.
- "Runoff Coeff." represents the runoff coefficient.
- "Pipe Dia." denotes the diameter of the pipe used in the design.
- "Time of Concentration" is the time required for water to travel from the point of runoff to the outlet of the system.
- ""Area" X "C"" is the product of the drainage area and the runoff coefficient.

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Structure Losses (ft)
Pipe Losses (ft)
## PRELIMINARY STORM DRAIN DESIGN COMPUTATIONS

### Pipe 8

| Col. 1 | From | 625 |
| Col. 2 | To   | 875 |
| Col. 3 | Run Length | \( L = 875 - 625 = 250 \text{ ft} \) |
| Col. 4 | Inlet Area | \( A_i = 6.2 \text{ ac} \) |
| Col. 5 | Total Area | \( A_t = 6.2 \text{ ac} \) |
| Col. 6 | “C” | \( C = 0.8 \) |
| Col. 7 | Inlet CA | \( CA = 4.96 \text{ ac} \) |
| Col. 8 | Sum CA | \( \sum CA = 4.96 \text{ ac} \) |
| Col. 9 | Inlet Time | \( t_i = 12 + 0.06 = 12.06 \text{ min} \); Travel time in Pipe 9 = 0.06 min |
| Col. 10 | System Time | \( t_c = 12.06 \text{ min} \) |
| Col. 11 | Intensity | \( i = 4.4 \text{ in/hr} \) |
| Col. 12 | Runoff | \( Q = CiA = 4.96(4.4) = 21.82 \text{ cfs} \) |
| Col. 21 | Slope | \( S = 0.014 \) |
| Col. 13 | Pipe Dia. | \[
D = \left( \frac{(Qn)}{(KQ\sqrt{S_o})} \right)^{0.375} = \left[ \frac{(21.82(0.013))}{(0.46\sqrt{0.014})} \right]^{0.375} \\
D = 1.86 \text{ ft} \\
\]
| Col. 14 | Full Cap. | \[
Q_f = \left( \frac{KQ}{n} \right) D^{0.67} \sqrt{S_o} = \left( \frac{0.46}{0.013} \right)^{2.0^{0.67} \sqrt{0.014}} \\
Q_f = 26.65 \text{ cfs} \\
\]
| Col. 15 | Vel. Full | \[
V_f = \left( \frac{KV}{n} \right) D^{0.67} \sqrt{S_o} = \left( \frac{0.59}{0.013} \right)^{2.0^{0.67} \sqrt{0.014}} \\
V_f = 8.54 \text{ ft/s} \\
\]
| Col. 16 | Vel. Design | \[
Q/Q_f = 21.82/26.65 = 0.83 \\
V/V_f = 1.13 \rightarrow V = 1.13(8.54) = 9.64 \text{ ft/s} \\
\]
| Col. 17 | Sect. Time | \( t_s = L/V = 250/9.64/60 = 0.43 \text{ min} \) |
| Col. 20 | Crown Drop | \( H_{ah} = 0 \) |
| Col. 18 | U/S Invert | 111.68 ft |
| Col. 19 | D/S Invert | 108.18 ft |
### Pipe 7

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<tr>
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<td>Col. 4</td>
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</tr>
<tr>
<td>Col. 7</td>
<td>Inlet CA</td>
<td>( CA = 4.30 \text{ ac} )</td>
</tr>
<tr>
<td>Col. 8</td>
<td>Sum CA</td>
<td>( \sum CA = 9.26 \text{ ac} )</td>
</tr>
<tr>
<td>Col. 9</td>
<td>Inlet Time</td>
<td>( t_i = 16.0 + 0.06 = 16.06 \text{ min} )</td>
</tr>
<tr>
<td>Col. 10</td>
<td>System Time</td>
<td>( t_c = 12.06 + 0.43 = 12.49 \text{ min} )</td>
</tr>
<tr>
<td>Col. 11</td>
<td>Intensity</td>
<td>( i = 3.82 \text{ in/hr} )</td>
</tr>
<tr>
<td>Col. 12</td>
<td>Runoff</td>
<td>( Q = CiA = 9.26(3.82) = 35.39 \text{ cfs} )</td>
</tr>
<tr>
<td>Col. 21</td>
<td>Slope</td>
<td>( S = 0.01 )</td>
</tr>
</tbody>
</table>
| Col. 13 | Pipe Dia. | \[
D = \left( \frac{(Q_n)}{(K_q \sqrt{S_o})} \right)^{0.375} = \left( \frac{(35.39(0.013))}{(0.46(0.01))} \right)^{0.375} \\
D = 2.37 \text{ ft} \\
\text{Therefore nominal pipe size} \quad D = 2.5 \text{ ft} 
\]
| Col. 14 | Full Cap. | \[
Q_f = \left( \frac{K_q}{n} \right) D^{2.67} \sqrt{S_o} = \left( \frac{0.46}{0.013} \right) 2.5^{2.67} \sqrt{0.01} \\
Q_f = 40.86 \text{ cfs} 
\]
| Col. 15 | Vel. Full | \[
V_f = (K_V/n)D^{0.67} \sqrt{S_o} = (0.59/0.013)2.5^{0.67} \sqrt{0.01} = 8.39 \text{ ft/s} 
\]
| Col. 16 | Vel. Design | \[
Q/Q_f = 35.39/40.86 = 0.87 \\
V/V_f = 1.14 \rightarrow V = 1.14(8.39) = 9.54 \text{ ft/s} \]
| Col. 17 | Sect. Time | \( t_s = L/V = 125/9.54/60 = 0.22 \text{ min} \) |
| Col. 20 | Crown Drop | \( H_{ah} = 0 \) |
| Col. 18 | U/S Invert | \( = 107.68 \text{ ft} \) |
| Col. 19 | D/S Invert | \( = 106.43 \text{ ft} \) |

### Pipe 5

<table>
<thead>
<tr>
<th>Col. 1</th>
<th>From</th>
<th>500</th>
</tr>
</thead>
<tbody>
<tr>
<td>Col. 2</td>
<td>To</td>
<td>780</td>
</tr>
<tr>
<td>Col. 1</td>
<td>From</td>
<td>370</td>
</tr>
<tr>
<td>Col. 2</td>
<td>To</td>
<td>500</td>
</tr>
<tr>
<td>Col. 3</td>
<td>Run Length</td>
<td>( L = 500 - 370 = 130 ) ft</td>
</tr>
<tr>
<td>Col. 4</td>
<td>Inlet Area</td>
<td>( A_i = 11.03 ) ac</td>
</tr>
</tbody>
</table>

Col. 3 Run Length: \( L = 780 - 500 = 280 \) ft

Col. 4 Inlet Area: \( A_i = 13.54 \) ac

Col. 5 Total Area: \( A_t = 13.54 \) ac

Col. 6 “C”: \( C = 0.59 \)

Col. 7 Inlet CA: \( CA = 7.99 \) ac

Col. 8 Sum CA: \( \sum CA = 7.99 \) ac

Col. 9 Inlet Time: \( t_i = 18.0 + 0.06 \text{ min} = 18.06 \text{ min} \)

Col. 10 System Time: \( t_c = 18.06 \) min

Col. 11 Intensity: \( i = 3.60 \) in/hr

Col. 12 Runoff: \( Q = C_i A = 7.99(3.60) = 28.76 \) cfs

Col. 21 Slope: \( S = 0.0145 \)

Col. 13 Pipe Dia.: \[
D = \left[ \frac{(Qn)}{(KQV_nS_o)} \right]^{0.375} = \left[ \frac{(28.76(0.013))}{(0.46\sqrt{0.0145})} \right]^{0.375} = 2.05 \text{ ft} \]

Based on allowable headwater for pipe culvert in a headwall

Col. 14 Full Cap.: \[
Q_f = \left( \frac{Kq}{n} \right) D^{2.67} \sqrt{S_o} = (\frac{0.46}{0.013})3.0^{2.67} \sqrt{0.0145} = 80.06 \text{ cfs} \]

Col. 15 Vel. Full: \[
V_f = \left( \frac{KV}{n} \right) D^{0.67} \sqrt{S_o} = (\frac{0.59}{0.013})3.0^{0.67} \sqrt{0.0145} = 11.41 \text{ ft/s} \]

Col. 16 Vel. Design: \( Q/Q_f = 28.76/80.06 = 0.36 \)

\( V/V_f = 0.92 \rightarrow V = 0.92(11.41) = 10.50 \text{ ft/s} \)

Col. 17 Sect. Time: \( t_s = L/V = 280/10.50/60 = 0.44 \text{ min} \)

Col. 20 Crown Drop: \( H_{ah} = 0 \)

Col. 18 U/S Invert: \( = 109.99 \) ft

Col. 19 D/S Invert: \( = 105.93 \) ft

**Pipe 4**

Col. 1 From: 370

Col. 2 To: 500

Col. 3 Run Length: \( L = 500 - 370 = 130 \) ft

Col. 4 Inlet Area: \( A_i = 11.03 \) ac
| Col. 5 | Total Area | \( A_t = 13.62 + 13.54 + 11.03 = 38.19 \text{ ac} \) |
| Col. 6 | “C” | \( C = 0.62 \) |
| Col. 7 | Inlet CA | \( CA = 0.62 \times 11.03 = 6.84 \text{ ac} \) |
| Col. 8 | Sum CA | \( \sum CA = 9.26 + 7.99 + 6.84 = 24.09 \text{ ac} \) |
| Col. 9 | Inlet Time | \( t_i = 15.06 \text{ min} \) |
| Col. 10 | System Time | \( t_c = 18.50 \text{ min} \) |
| Col. 11 | Intensity | \( i = 3.50 \text{ in/hr} \) |
| Col. 12 | Runoff | \( Q = CiA = 24.09(3.50) = 84.32 \text{ cfs} \) |
| Col. 21 | Slope | \( S = 0.014 \) |
| Col. 13 | Pipe Dia. | \( D = \left( \frac{Qn}{KQ\sqrt{S_o}} \right)^{0.375} = \left( \frac{(84.32(0.013))}{(0.46\sqrt{0.014})} \right)^{0.375} \) |
| & | \( D = 3.08 \text{ ft} \) |
| & | Therefore nominal pipe size \( D = 3.5 \text{ ft} \) |
| Col. 14 | Full Cap. | \( Q_f = \left( \frac{KQ}{n} \right) D^{2.67}\sqrt{S_o} = \left( \frac{0.46}{0.013} \right) 3.5^{2.67}\sqrt{0.014} \) |
| & | \( Q_f = 118.72 \text{ cfs} \) |
| Col. 15 | Vel. Full | \( V_f = \left( \frac{KV}{n} \right) D^{0.67}\sqrt{S_o} = \left( \frac{0.59}{0.013} \right) 3.5^{0.67}\sqrt{0.014} \) |
| & | \( V_f = 12.43 \text{ ft/s} \) |
| Col. 16 | Vel. Design | \( Q/Q_f = 84.32/118.72 = 0.71 \) |
| & | \( V/V_f = 1.10 \rightarrow V = 1.10(12.43) = 13.67 \text{ ft/s} \) |
| Col. 17 | Sect. Time | \( t_s = L/V = 130/13.67/60 = 0.16 \text{ min} \) |
| Col. 20 | Crown Drop | \( H_{ah} = 0 \) |
| Col. 18 | U/S Invert | \( = 105.43 \text{ ft} \) |
| Col. 19 | D/S Invert | \( = 103.61 \text{ ft} \) |

**Pipe 3**

| Col. 1 | From | 240 |
| Col. 2 | To | 370 |
| Col. 3 | Run Length | \( L = 370 - 240 = 130 \text{ ft} \) |
| Col. 4 | Inlet Area | \( A_i = 3.76 \text{ ac} \) |
| Col. 5 | Total Area | \( A_t = 38.19 + 3.76 = 41.95 \text{ ac} \) |
| Col. 6 | “C” | \( C = 0.53 \) |
| Col. 7 | Inlet CA | CA = 1.99 |
| Col. 8 | Sum CA   | $\Sigma CA = 26.08\text{ ac}$ |
| Col. 9 | Inlet Time | $t_i = 10.0 + 0.06 = 10.06\text{ min}$ |
| Col. 10 | System Time | $t_c = 18.66\text{ min}$ |
| Col. 11 | Intensity | $i = 3.50\text{ in/hr}$ |
| Col. 12 | Runoff | $Q = CiA = 26.08(3.50) = 91.29\text{ cfs}$ |
| Col. 21 | Slope | $S = 0.007$ |
| Col. 13 | Pipe Dia. | $D = \left[ \frac{(Qn)}{(KQ\sqrt{So})} \right]^{0.375} = \left[ \frac{(91.29(0.013))}{(0.46\sqrt{0.007})} \right]^{0.375}$ |
|         |         | $D = 3.62\text{ ft}$ |
|         |         | Therefore nominal pipe size $D = 4.0\text{ ft}$ |
| Col. 14 | Full Cap. | $Q_f = \left( \frac{KQ}{n} \right) D^{2.67}\sqrt{S_o} = \left( \frac{0.46}{0.013} \right) 4.0^{2.67}\sqrt{0.007}$ |
|         |         | $Q_f = 119.91\text{ cfs}$ |
| Col. 15 | Vel. Full | $V_f = \left( \frac{KV}{n} \right) D^{0.67}\sqrt{S_o} = \left( \frac{0.59}{0.013} \right) 4.0^{0.67}\sqrt{0.007}$ |
|         |         | $V_f = 9.61\text{ ft/s}$ |
| Col. 16 | Vel. Design | $Q/Q_f = 91.29/119.91 = 0.76$ |
|         |         | $V/V_f = 1.13 \rightarrow V = 1.13(9.61) = 10.86\text{ ft/s}$ |
| Col. 17 | Sect. Time | $t_s = L/V = 130/10.86/60 = 0.20\text{ min}$ |
| Col. 20 | Crown Drop | $H_{ah} = 0$ |
| Col. 18 | U/S Invert | $= 103.11\text{ ft}$ |
| Col. 19 | D/S Invert | $= 102.20\text{ ft}$ |

**Pipe 2**

| Col. 1 | From | 100 |
| Col. 2 | To | 240 |
| Col. 3 | Run Length | $L = 240 - 100 = 140\text{ ft}$ |
| Col. 4 | Inlet Area | $A_i = 4.95 + 4.39 = 9.34\text{ ac}$ |
| Col. 5 | Total Area | $A_t = 9.34 + 41.95 = 51.29\text{ ac}$ |
| Col. 6 | “C” | $C = (0.53(4.95) + 0.49(4.39)) / (4.95 + 4.39) = 0.51$ |
| Col. 7 | Inlet CA | $CA = 4.77\text{ ac}$ |
| Col. 8 | Sum CA | $\Sigma CA = 30.86\text{ ac}$ |
Col. 9  | Inlet Time  | \( t_i = 12.00 + 0.06 = 12.06 \)  
Col. 10 | System Time | \( t_c = 18.86 \) min  
Col. 11 | Intensity   | \( i = 3.49 \) in/hr  
Col. 12 | Runoff      | \( Q = CiA = 30.86(3.49) = 107.70 \) cfs  
Col. 21 | Slope       | \( S = 0.005 \)  
Col. 13 | Pipe Dia.    | \( D = \left[ \frac{(Q_n)}{(KQ/S_o)} \right]^{0.375} = \left[ \frac{(107.70(0.013))}{(0.46\sqrt{0.005})} \right]^{0.375} \)  
|              |             | \( D = 4.10 \) ft  
|              |             | Therefore nominal pipe size \( D = 4.0 \) ft  
Col. 14 | Full Cap.    | \( Q_f = \left( \frac{KQ_n}{n} \right)D^{2.67}\sqrt{S_o} = \left( \frac{0.46}{0.013} \right)4.0^{2.67}\sqrt{0.005} \)  
|              |             | \( Q_f = 101.34 \) cfs  
Col. 15 | Vel. Full    | \( V_f = Q/A = 107.70/(0.25\pi4^2) = 8.57 \) ft/s  
Col. 16 | Vel. Design  | \( V = V_f = 8.57 \) ft/s  
Col. 17 | Sect. Time   | \( t_s = L/V = 140/8.57/60 = 0.27 \) min  
Col. 20 | Crown Drop   | \( H_{ah} = 0 \)  
Col. 18 | U/S Invert   | \( = 102.20 \) ft  
Col. 19 | D/S Invert   | \( = 101.50 \) ft  

**Pipe 1**

Col. 1 | From | 0  
Col. 2 | To   | 100  
Col. 3 | Run Length | \( L = 100 - 0 = 100 \) ft  
Col. 4 | Inlet Area | \( A_i = 6.48 \) ac  
Col. 5 | Total Area | \( A_t = 57.77 \) ac  
Col. 6 | “C” | \( C = 0.51 \)  
Col. 7 | Inlet CA | \( CA = 3.30 \) ac  
Col. 8 | Sum CA | \( \sum CA = 30.86 + 3.30 = 34.16 \) ac  
Col. 9 | Inlet Time | \( t_i = 10.06 \) min  
Col. 10 | System Time | \( t_c = 19.13 \) min  
Col. 11 | Intensity | \( i = 3.43 \) in/hr  
Col. 12 | Runoff | \( Q = CiA = 34.16(3.43) = 117.18 \) cfs
<table>
<thead>
<tr>
<th>Col. 21</th>
<th>Slope</th>
<th>S = 0.005</th>
</tr>
</thead>
</table>
| Col. 13 | Pipe Dia. | \[ D = \left( \frac{Q_n}{K_Qv_0} \right)^{0.375} = \left( \frac{117.18(0.013)}{0.46\sqrt{0.005}} \right)^{0.375} \]  
\[ D = 4.23 \text{ ft} \]  
Therefore nominal pipe size \( D = 4.0 \text{ ft} \) |
| Col. 14 | Full Cap. | \[ Q_f = \left( \frac{K_q}{n} \right)D^{2.67} \sqrt{S_0} = \left( \frac{0.46}{0.013} \right)4.0^{2.67}\sqrt{0.005} \]  
\[ Q_f = 101.34 \text{ cfs} \] |
| Col. 15 | Vel. Full | \( V_f = Q/A = 117.18/(0.25\pi4^2) = 9.32 \text{ ft/s} \) |
| Col. 16 | Vel. Design | \( V = V_f = 9.32 \text{ ft/s} \) |
| Col. 17 | Sect. Time | \( t_s = L/V = 100/9.32/60 = 0.18 \text{ min} \) |
| Col. 20 | Crown Drop | \( H_{ah} = 0 \) |
| Col. 18 | U/S Invert | = 101.50 ft |
| Col. 19 | D/S Invert | = 101.00 ft |
Evaluation of EGL and HGL

Starting at the most downstream location computations proceed in the upstream direction. The EGL’s Table 7–4 and Table 7–5 are used to record the design computations. The computations for each pipe are described in detail below and recorded in the EGL Table 7–4 and Table 7–5.

Structure 1

Step 3  Col. 1A, 1B Str. ID = 1
  Col. 2A  D = 4.0 ft
  Col. 3A  Q = 117.18 cfs
  Col. 4A  L = 100 ft

Step 4  Col. 5A  V = 9.32 ft/s
  Col. 8A  $V^2/2g = \frac{9.32^2}{2(32.2)} = 1.35$ ft
  Col. 10A  HGL₀ = 105.0 ft
  Col. 9A  EGL₀ = HGL₀ + $V^2/2g = 105 + \frac{9.32^2}{2(32.2)} = 106.35$ ft

Step 11  Col. 11A  $S_f = \left(\frac{Qn}{K_QD^{2.67}}\right)^2 = \left(\frac{(117.18)(0.013)}{\left(\left(0.46(4.0)^{2.67}\right)\right)^2}\right)^2$
  $S_f = 0.00668$ ft/ft

Step 12  Col. 3B  $H_f = S_f L = 0.00668 (100) = 0.67$ ft
  $H_b, H_c, H_e = 0$ ft
  Col. 7B  $Q_i = Q_o - Q_i = 117.18 - 107.18 = 9.48$ cfs
  $V_i = Q/A = 9.48/(0.25\pi(1.5)^2) = 5.36$ ft/s
  $H_j = \left(\left(Q_o V_o - (Q_i V_i) - (Q_i V_i \cos \theta_j)\right)/\left(0.5g (A_o + A_i)\right)\right) + h_i - h_o$
  $H_j = \left(\left(117.18(9.32) - 107.7(8.57) - (9.48(5.36) \cos 90)\right)/\left(0.5(32.2)(12.57 + 12.57)\right)\right)$
  $+ (8.572 - 9.322) / (2)32.2$
  $H_j = 0.21$ ft
  Col. 8B,
  Col. 12A  Total pipe loss = 0.67 + 0.21 = 0.88 ft

Step 13  Col. 13A  EGLᵢ = EGL₀ + Total pipe loss = 106.35 + 0.88 = 107.23 ft
  Col. 14A  HGLᵢ = EGLᵢ + $V^2/2g = 107.23 + \frac{9.32^2}{2(32.2)} = 105.88$ ft

Step 14  Case A?  HGLᵢ ≥ TOCᵢ
105.55 ft ≥ (101.50 + 4.0) ft
105.55 ft ≥ 105.50 ft  ✓ Pressure Flow

Col. 6A "FULL"
Col. 7A "NA"

Step 23 Col. 17A US TOC = 101.50 + 4.0 = 105.50 ft

No access hole, Skip to next pipe.

Structure 2

Step 3 Col. 1A, 1B Str. ID = 2
Col. 2A D = 4.0 ft
Col. 3A Q = 107.70 cfs
Col. 4A L = 140 ft

Step 4 Col. 10A HGL₀ = 105.88 ft
Col. 9A EGL₀ = 107.23 ft

Step 11 Col. 11A \( S_f = \left(\frac{Qn}{K_Q D^{2.67}}\right)^2 = \left(\frac{107.70(0.013)}{(0.46(4.02))^{2.67}}\right)^2 \)
\( S_f = 0.00565 \text{ ft/ft} \)

Step 12 Col. 3B \( H_f = S_f L = 0.00565 \times 140 = 0.79 \text{ ft} \)
\( H_b, H_c, H_e = 0 \)

Col. 8B, Col. 12A Total pipe loss = 0.79 ft

Step 13 Col. 13A EGLᵢ = EGL₀ + Total pipe loss = 107.23 + 0.79 = 108.02 ft
Col. 14A HGLᵢ = EGLᵢ + \( V^2/2g \) = 108.02 + 1.14 = 109.16 ft

Step 14 Case A? HGLᵢ ≥ TOCᵢ
\( 108.02 \text{ ft} ≥ (102.20 + 4.0) \text{ ft} \)
\( 108.02 \text{ ft} ≥ 106.20 \text{ ft}  ✓ Pressure Flow \)

Col. 6A "FULL"
Col. 7A "N/A"

Step 15 Col. 9B \( E_i = EGL_i - BOC_i = 108.02 - 102.20 = 5.82 \text{ ft} \)
Col. 10B \( y + (P/\gamma) = E_i - V^2/2g = 5.82 - 1.14 = 4.68 \text{ ft} \)
Col. 11B \( DI = Q/(A(Dg)^{0.5}) = 107.70/((0.25\pi(4.0)^2)(4(32.2))^{0.5}) \)
\( DI = 0.755 \)
Step 16a  Not supercritical flow
\[ E_{aio} = E_i + 0.2\left(V^2/g\right) = 5.82 + 0.2(1.14) = 6.05 \text{ ft} \]

Step 16b  \[ E_{ais} = (1.0)(DI)^2D = (1.0)(0.755)^24.0 = 2.28 \text{ ft} \]

Step 16c  \[ E_{aiu} = (1.6)(DI)^{0.67}D = (1.6)(0.755)^{0.67}4.0 = 5.30 \text{ ft} \]

Col. 12B  \[ E_i = \max(E_{aio}, E_{ais}, E_{aiu}) = 6.05 \text{ ft} \]

Step 17  Col. 13B  \[ C_B = -0.05 \]

Step 18  Col. 14B  \[ Q_B = CiA = 0.53(4.40)(4.95) = 11.45 \text{ cfs} \]

\[ Q_C = CiA = 0.49(4.40)(4.39) = 9.46 \text{ cfs} \]

\[ \theta_w = \sum(Q_j\theta_j)/\sum Q_j \]

\[ \theta_w = \left(\frac{(11.54(90) + 91.29(180) + 9.46(90))/(11.54 + 91.29) + 9.46}{11.54 + 91.29 + 9.46}\right) \]

\[ \theta_w = 163.3^\circ \]

\[ C_\theta = 4.5\left(\sum Q_j/Q_o\right)(\cos \theta_w/2) \]

\[ C_\theta = 4.5(11.54 + 91.29 + 9.46/107.7)(\cos(163.3/2) \]

\[ C_\theta = 0.68 \]

Step 19  Col. 15B  Inverts of both pipes (B and C) are less than \( E_i \) (6.05 ft) above invert

Col. 15A  \[ C_P = 0 \]

Step 20  Col. 16B  \[ H_a = (E_{ai} - E_i)(C_B + C_\theta + C_P) = (6.05 + 5.82)(-0.05 + 0.68 + 0) \]

\[ H_a = 0.14 \text{ ft} \]

Step 21  Col. 17B,

Col. 15A  \[ E_a = H_a + E_{ai} = 0.14 + 6.05 = 6.19 \text{ ft} \]

Step 22  Col. 16A  \[ EGL_a = E_a + BOC_i = 6.19 + 102.20 = 108.39 \text{ ft} \]

Step 23  Col. 17A  \[ US \text{ TOC} = 102.20 + 4.0 = 106.20 \text{ ft} \]

**Structure 3**

Step 3  Col. 1A, 1B  Str. ID = 3

Col. 2A  \[ D = 4.0 \text{ ft} \]

Col. 3A  \[ Q = 91.29 \text{ cfs} \]

Col. 4A  \[ L = 130 \text{ ft} \]

Step 4  Case B?  \[ EGL_a > TOC_o \]

\[ 108.39 > (102.20 + 4.0) \]
108.39 > 106.20 ft √

Col. 5A  
\[ V = \frac{Q}{A} = \frac{91.29}{(0.25\pi4^2)} = 7.26 \text{ ft/s} \]

Col. 8A  
\[ \frac{V^2}{2g} = \frac{7.26^2}{2(32.2)} = 0.82 \text{ ft} \]

Col. 2B  
\[ H_0 = 0.4(V^2/2g) = 0.4(0.82) = 0.33 \text{ ft} \]

Col. 9A  
\[ \text{EGL}_0 = \text{EGL}_a + H_0 = 108.39 + 0.33 = 108.72 \text{ ft} \]

Col. 10A  
\[ \text{HGL}_0 = \text{EGL}_0 - \frac{V^2}{2g} = 108.72 - 0.82 = 107.90 \text{ ft} \]

Step 11  
Col. 11A  
\[ S_f = \left( \frac{Qn}{(K_QD^{2.67})} \right)^2 = \left( \frac{91.29(0.013)}{(0.46(4.02))^{2.67}} \right)^2 \]
\[ S_f = 0.00406 \text{ ft/ft} \]

Step 12  
Col. 3B  
\[ H_f = S_fL = 0.00406(130) = 0.53 \text{ ft} \]

Col. 6B  
\[ H_e = K_e(V_f^2/2g) \]
\[ D_2/D_1 = 4/3.5 = 1.14, \ V_i = 8.76 \text{ ft/s}, K_e = 0.1 \]
\[ H_e = 0.1(8.762/64.4) = 0.12 \text{ ft} \]

Col. 7B  
\[ Q_1 = C_iA = 0.53(4.8)(3.76) = 9.57 \text{ cfs} \]
\[ V_i = \frac{Q}{A} = \frac{9.57}{(0.25\pi1.5^2)} = 5.42 \text{ ft/s} \]
\[ H_j = \left( \left( Q_oV_o - (Q_iV_i) - (Q_iV_i \cos \theta_j) \right)/(0.5g (A_o + A_i)) \right) + h_i - h_o \]
\[ H_j = \left( (91.29(7.26) - 82.32(8.76) - (9.57(5.42) \cos 45))/(0.5(32.2)(12.57 + 9.62)) \right) \]
\[ + (8.762 - 7.262) / (2)32.2 \]
\[ H_j = 0.06 \text{ ft} \]

Col. 8B,

Col. 12A  
Total pipe loss = 0.53 + 0.12 + 0.06 = 0.71 ft

Step 13  
Col. 13A  
\[ \text{EGL}_i = \text{EGL}_0 + \text{Total pipe loss} = 108.72 + 0.71 = 109.42 \text{ ft} \]

Col. 14A  
\[ \text{HGL}_i = \text{EGL}_i + \frac{V^2}{2g} = 108.60 \text{ ft} \]

Step 14  
Case A?  
\[ \text{HGL}_i \geq \text{TOC}_i \]
108.60 ft ≥ (103.11 + 4.0) ft
108.60 ft ≥ 107.11 ft  √ Pressure Flow

Col. 6A  
"FULL"

Col. 7A  
"NA"

Step 23  
Col. 17A  
US TOC = 103.61 + 3.5 = 107.11 ft
No access hole, Skip to next pipe.

**Structure 4**

<table>
<thead>
<tr>
<th>Step</th>
<th>Column</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>1A, 1B</td>
<td>Str. ID = 4</td>
</tr>
<tr>
<td></td>
<td>2A</td>
<td>$D = 3.5$ ft</td>
</tr>
<tr>
<td></td>
<td>3A</td>
<td>$Q = 84.32$ cfs</td>
</tr>
<tr>
<td></td>
<td>4A</td>
<td>$L = 130$ ft</td>
</tr>
<tr>
<td>4</td>
<td>9A</td>
<td>$\text{EGL}_0 = 109.42$ ft</td>
</tr>
<tr>
<td></td>
<td>10A</td>
<td>$\text{HGL}_0 = 108.60$ ft</td>
</tr>
<tr>
<td>11</td>
<td>11A</td>
<td>$S_f = \left( \frac{Q}{\left(K_Q D^{2.67}\right)} \right)^2 = \left( \frac{84.32}{\left(0.013\right)} / \left(\left((0.46)(3.5)\right)^{2.67}\right) \right)^2$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$S_f = 0.00706$ ft/ft</td>
</tr>
<tr>
<td>12</td>
<td>3B</td>
<td>$H_f = S_fL = 0.00706 \times 130 = 0.92$ ft</td>
</tr>
<tr>
<td></td>
<td>8B, 12A</td>
<td>Total pipe loss $= 0.92$ ft</td>
</tr>
<tr>
<td>13</td>
<td>13A</td>
<td>$\text{EGL}_i = \text{EGL}_0 + \text{Total pipe loss} = 109.42 + 0.92 = 110.34$ ft</td>
</tr>
<tr>
<td></td>
<td>14A</td>
<td>$\text{HGL}_i = \text{EGL}_i + V^2/2g = 110.34 + 1.19 = 109.15$ ft</td>
</tr>
<tr>
<td>14</td>
<td>Case A?</td>
<td>$\text{HGL}_i \geq \text{TOC}_i$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$109.15$ ft $\geq (105.43 + 3.5)$ ft</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$109.15$ ft $\geq 108.93$ ft $\checkmark$ Pressure Flow</td>
</tr>
<tr>
<td>15</td>
<td>9B</td>
<td>$E_i = \text{EGL}_i - \text{BOC}_i = 110.34 - 105.43 = 4.91$ ft</td>
</tr>
<tr>
<td></td>
<td>10B</td>
<td>$y + (P/\gamma) = E_i - V^2/2g = 4.91 - 1.19 = 3.72$ ft</td>
</tr>
<tr>
<td></td>
<td>11B</td>
<td>$\text{DI} = Q/(A(Dg)^{0.5}) = 84.32 / \left(0.25\pi(3.5)^2 \right) (4(32.2))^{0.5}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\text{DI} = 0.83$</td>
</tr>
<tr>
<td>16a</td>
<td></td>
<td>Not supercritical flow</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$E_{ai0} = E_i + 0.2(V_o^2/2g) = 4.91 + 0.2(1.19) = 5.15$ ft</td>
</tr>
<tr>
<td>16b</td>
<td></td>
<td>$E_{ais} = (1.0)(\text{DI})^{2}D = (1.0)(0.826)^23.5 = 2.39$ ft</td>
</tr>
<tr>
<td>16c</td>
<td></td>
<td>$E_{aiu} = (1.6)(\text{DI})^{0.67}D = (1.6)(0.826)^{0.67}3.5 = 4.93$ ft</td>
</tr>
</tbody>
</table>
Col. 12B  \( E_{ai} = \max(E_{aio}, E_{ais}, E_{aiu}) = 5.15 \text{ ft} \)

Step 17  Col. 13B  \( C_B = -0.05 \)

Step 18  Col. 14B  
\[
\theta_w = \sum \left( Q_j \theta_j \right)/\sum Q_j
\]
\[
\theta_w = ((23.94(90) + 35.39(135) + 28.76(180))/(23.94 + 35.39 + 28.76)
\]
\[
\theta_w = 140.8^\circ
\]
\[
C_\theta = 4.5(\sum Q_j/Q_o)(\cos \theta_w/2)
\]
\[
C_\theta = 4.5(23.9 + 35.4 + 28.73/84.32)(\cos (140.8/2)
\]
\[
C_\theta = 1.58
\]

Step 19  Col. 15B  Inverts of both pipes (P7 and E) are less than \( E_{ai} \) (5.15 ft) above invert
\( C_p = 0 \)

Step 20  Col. 16B  
\[
H_a = (E_{ai} - E_i)(C_B + C_\theta + C_p) = (5.15 + 4.91)(-0.05 + 1.58 + 0)
\]
\[
H_a = 0.36 \text{ ft}
\]

Step 21  Col. 17B,

Col. 15A  \( E_a = H_a + E_{ai} = 0.36 + 5.15 = 5.51 \text{ ft} \)

Step 22  Col. 16A  \( EGL_a = E_a + BOC_i = 5.51 + 105.43 = 110.94 \text{ ft} \)

Step 23  Col. 17A  US TOC = 105.93 + 3.0 = 108.93 ft

**Structure 5**

Step 3  Col. 1A, 1B  Str. ID = 5

Col. 2A  \( D = 3.0 \text{ ft} \)

Col. 3A  \( Q = 28.76 \text{ cfs} \)

Col. 4A  \( L = 280 \text{ ft} \)

Step 4  Case B?  \( EGL_a > TOC_o \)

\[ 110.94 > (105.93 + 3.0) \]
\[ 110.94 > 108.93 \text{ ft} \checkmark \]

Col. 5A  \( V = Q/A = 28.76/(0.25\pi 3^2) = 4.07 \text{ ft/s} \)

Col. 8A  \( V^2/2g = \frac{4.07^2}{2(32.2)} = 0.26 \text{ ft} \)

Col. 2B  \( H_0 = 0.4(V^2/2g) = 0.4(0.26) = 0.10 \text{ ft} \)

Col. 9A  \( EGL_0 = EGL_a + H_o = 110.94 + 0.10 = 111.04 \text{ ft} \)
Col. 10A  \( \text{HGL}_0 = \text{EGL}_0 - \frac{V^2}{2g} = 111.04 - 0.26 = 110.78 \text{ ft} \)

Step 11  Col. 11A  \( S_f = \left( \frac{Qn}{(KQD^{2.67})} \right)^2 = \left( \frac{(28.76)(0.013)}{((0.46)(3.0)^{2.67})} \right)^2 \)

\( S_f = 0.00187 \text{ ft/ft} \)

Step 12  Col. 3B  \( H_f = S_fL = 0.00187(280) = 0.52 \text{ ft} \)

\( H_b, H_c, H_e, H_j = 0 \)

Col. 8B, Col. 12A  Total pipe loss = 0.52 ft

Step 13  Col. 13A  \( \text{EGL}_i = \text{EGL}_0 + \text{Total pipe loss} = 111.04 + 0.52 = 111.56 \text{ ft} \)

Col. 14A  \( \text{HGL}_i = \text{EGL}_i + \frac{V^2}{2g} = 111.30 \text{ ft} \)

Step 14  Case A?

\( \text{HGL}_i \geq \text{TOC}_i \)

\( 111.30 \text{ ft} \geq (109.99 + 3.0) \text{ ft} \)

\( 111.30 \text{ ft} \geq 112.99 \text{ ft} \quad \checkmark \)

Case B?

Calculate normal and critical depth

\( \frac{Q}{Q_f} = 3.36 \)

Col. 5A  \( V = 0.89(11.41) = 10.50 \text{ ft/s} \)

Col. 6A  \( d_n = 1.25 \text{ ft} \)

Col. 7A  \( d_c = 1.75 \text{ ft} \)

\( d_n < \text{HGL}_i < \text{TOC}_i \) and \( d_c < \text{HGL}_i \)

\( (109.99 + 1.25) < 111.30 < (109.99 + 3) \) and \( (109.99 + 1.75) < 111.30 \)

\( 111.24 < 111.30 < 112.99 \text{ and } 111.74 < 111.30 \) \( \checkmark \)

Case C?

\( d_c < \text{HGL}_i < \text{TOC}_i \) and \( \text{HGL}_i \leq d_n \)

\( (109.99 + 1.75) < 111.30 < (109.99 + 3) \) and \( 111.30 \leq (109.99 + 1.25) \)

\( 111.74 < 111.30 < 112.99 \) and \( 111.30 \leq 111.24 \) \( \checkmark \)

Case D?

\( \text{HGL}_i < d_c \)

\( 111.30 < (109.99 + 1.75) \)

\( 111.30 < 111.74 \quad \checkmark \) Supercritical flow. Go to Step 9.

Step 9  Col. 8B, Col. 12A  Total pipe loss = 0

Step 10  Col. 14A  \( \text{HGL}_i = d_n - \text{BOC}_i = 1.25 - 109.99 = 111.24 \text{ ft} \)

Col. 13A  \( \text{EGL}_i = \text{HGL}_i + \frac{V^2}{2g} = 111.24 + 1.71 = 112.95 \text{ ft} \)
Step 15  Col. 9B  \( E_i = EGL_i - BOC_i = 112.95 - 109.99 = 2.96 \text{ ft} \)

Col. 10B  \( y + (P/\gamma) = E_i - V^2/2g = 5.96 - 1.71 = 1.25 \text{ ft} \)

Col. 11B  \( DI = Q/(A(Dg)^{0.5}) = 28.76/\left( (0.25\pi(3.0)^2)(3(32.2))^{0.5} \right) \)

\( DI = 0.414 \)

Step 16a  Supercritical flow

\( E_{ai0} = 0 \)

Step 16b  \( E_{ais} = (1.0)(DI)^2D = (1.0)(0.826)^23.0 = 0.514 \text{ ft} \)

Step 16c  \( E_{aiu} = (1.6)(DI)^{0.67}D = (1.6)(0.826)^{0.67}3.0 = 2.66 \text{ ft} \)

Col. 12B  \( E_i = \max(E_{ai0}, E_{ais}, E_{aiu}) = 2.66 \text{ ft} \)

\( 2.66 \text{ ft} < 2.96 \text{ ft} \)

Col. 16B  \( H_a = 0 \)

Col. 15A,

Col. 17B  \( E_a = E_i = 2.96 \text{ ft} \)

Skip to Step 22

Step 22  Col. 16A  \( EGL_a = E_a - BOC_i = 2.96 - 109.99 = 112.95 \text{ ft} \)

Step 23  Col. 17A  \( \text{US TOC} = 109.99 + 3.0 = 112.99 \text{ ft} \)

**Structure 6**

Step 3  Col. 1A, 1B  Str. ID = 6

Col. 2A  \( D = 3.0 \text{ ft} \)

Col. 3A  \( Q = 28.76 \text{ cfs} \)

Col. 4A  \( L = 10 \text{ ft} \)

Step 4  Case B?  \( EGL_a \geq TOC_o \)

\( 112.95 > (109.99 + 3.0) \)

\( 112.95 > 112.99 \times \)

Step 5  Col. 5A  \( Q/Q_f = 0.36 \)

\( V = 0.89(11.41) = 10.50 \text{ ft/s} \)

\( d = 1.25 \text{ ft} \)

Col. 8A  \( V^2/2g = \frac{10.50^2}{2(32.2)} = 1.71 \text{ ft} \)

Step 6  Col. 7A  \( d_c = 1.75 \text{ ft} \)
Step 7 Case F?  
(BOC_o + d) < EGL_a < (BOC_o + 3)  
(109.99 + 1.25) < 112.95 < (109.99 + 3)  
111.24 < 112.95 < 112.99 ✓  

Col. 6A \[ d = EGL_a - BOC_o = 112.95 - 109.99 = 2.96 \text{ ft} \]  
Col. 5A \[ d/d_f = 2.96/3 = 0.99 \]  
\[ A = A_f - 0.25\pi3^2 = 7.07 \text{ ft}^2 \]  
\[ V = Q/A = 28.76/7.07 = 4.07 \text{ ft/s} \]  

Col. 8A \[ V^2/2g = 0.26 \text{ ft} \]  

Col. 2B  
\[ H_o = 0.4(0.26) = 0.10 \text{ ft} \]  

Col. 9A  
\[ EGL_0 = EGL_a + H_o = 112.95 + 0.1 = 113.05 \text{ ft} \]  

Col. 10A  
\[ HGL_o = 113.05 - 0.26 = 112.80 \text{ ft} \]  

Step 8 Case A?  
\[ d > d_{cc} \]  
\[ 2.96 > 1.75 ✓ \text{ Flow is subcritical} \]  

Step 11 Depth of 2.96 feet assumed flowing full. Calculate friction slope  

Col. 11A \[ S_f = \left( \frac{Qn}{(K_QD^{2.67})} \right)^2 = \left( \frac{28.76(0.013)}{((0.46)(3.0))^{2.67}} \right)^2 \]  
\[ S_f = 0.00187 \text{ ft/ft} \]  

Step 12 Col. 3B  
\[ H_f = S_fL = 0.00187(10) = 0.019 \text{ ft} \]  
\[ H_b, H_c, H_e, H_j = 0 \]  

Col. 8B,  

Col. 12A  
Total pipe loss = 0.019 ft  

Step 13 Col 13A  
\[ EGL_i = EGL_0 + \text{Total pipe loss} = 113.05 + 0.02 = 113.07 \text{ ft} \]  

Col. 14A  
\[ HGL_i = EGL_i + V^2/2g = 112.82 \text{ ft} \]  

Step 14 Case B?  
\[ d_n < HGL_i < TOC_i \text{ and } d_c < HGL_i \]  
\[ (110.09 + 1.25) < 112.82 < (110.09 + 3) \text{ and } (109.99 + 1.75) < 112.82 \]  
\[ 111.34 < 112.82 < 113.07 \text{ and } 112.74 < 112.82 ✓ \]  

Culvert inlet upstream  

**Structure 7**  

Step 3  
Col. 1A, 1B Str. ID = 7  
Col. 2A  
\[ D = 2.5 \text{ ft} \]
Col. 3A  \( Q = 35.39 \text{ cfs} \)

Col. 4A  \( L = 125 \text{ ft} \)

Step 4  Case B?

\[ \text{EGL}_a > \text{TOC}_o \]

\[ 110.94 > (106.43 + 2.5) \]
\[ 110.94 > 108.93 \text{ ft} \checkmark \]

Col. 5A  \( V = Q/A = \frac{35.39}{0.25\pi(2.5^2)} = 7.21 \text{ ft/s} \)

Col. 8A  \( V^2/2g = \frac{7.21^2}{2(32.2)} = 0.81 \text{ ft} \)

Col. 2B  \( H_0 = 0.4(V^2/2g) = 0.4(0.81) = 0.323 \text{ ft} \)

Col. 9A  \( \text{EGL}_0 = \text{EGL}_a + H_0 = 110.94 + 0.32 = 111.27 \text{ ft} \)

Col. 10A  \( \text{HGL}_0 = \text{EGL}_0 - V^2/2g = 111.27 - 0.81 = 110.46 \text{ ft} \)

Step 11  Col. 11A  \( S_f = \left( \frac{Qn}{K_QD^{0.67}} \right)^2 = \left( \frac{35.39(0.013)}{((0.46)(2.5))^{0.67}} \right)^2 \)

\[ S_f = 0.00750 \text{ ft/ft} \]

Step 12  Col. 3B  \( h_f = S_fL = 0.00750 (125) = 9.4 \text{ ft} \)

\[ H_b, \ H_c, \ H_e, \ H_j = 0 \]

Col. 8B,

Col. 12A  Total pipe loss = 0.94 ft

Step 13  Col. 13A  \( \text{EGL}_i = \text{EGL}_0 + \text{Total pipe loss} = 111.27 + 0.94 = 112.20 \text{ ft} \)

Col. 14A  \( \text{HGL}_i = \text{EGL}_i + V^2/2g = 111.40 \text{ ft} \)

Step 14  Case A?

\( \text{HGL}_i \geq \text{TOC}_i \)

\[ 111.40 \text{ ft} \geq (107.68 + 2.5) \text{ ft} \]
\[ 111.40 \text{ ft} \geq 110.18 \text{ ft} \checkmark \]

Col. 6A  "FULL"

Col. 7A  "NA"

Step 15  Col. 9B  \( E_i = \text{EGL}_i - \text{BOC}_i = 112.20 - 107.68 = 4.52 \text{ ft} \)

Col. 10B  \( y + (P/\gamma) = E_i - V^2/2g = 4.52 - 0.81 = 3.72 \text{ ft} \)

Col. 11B  \( DI = \frac{Q}{(A(Dg)^{0.5})} = 35.39/\left(0.25\pi(2.5)^2(2.5(32.2))^{0.5}\right) \)

\[ DI = 0.80 \]

Step 16a  Not supercritical flow

\( E_{ailo} = E_i + 0.2(V_o^2/2g) = 4.52 + 0.2(0.81) = 4.69 \text{ ft} \)
Step 16b  \[ E_{ais} = (1.0)(DI)^2D = (1.0)(0.80)^22.5 = 1.61 \text{ ft} \]

Step 16c  \[ E_{aiu} = (1.6)(DI)^{0.67}D = (1.6)(0.80)^{0.67}2.5 = 3.45 \text{ ft} \]

Col. 12B  \[ E_{ai} = \max(E_{aio}, E_{ais}, E_{aiu}) = 4.69 \text{ ft} \]

Step 17  \[ C_B = -0.05 \]

Step 18  \[ Q_G = CiA = 0.58(3.82)(7.42) = 16.53 \text{ cfs} \]

\[ \theta_w = \sum(Q_j\theta_j)/\sum Q_j \]

\[ \theta_w = ((16.53(90) + 21.82(180))/(16.53 + 21.82) \]

\[ \theta_w = 141.2^\circ \]

\[ C_\theta = 4.5(\sum Q_j/Q_o)(\cos \theta_w/2) \]

\[ C_\theta = 4.5(16.53 +21.82/35.39)(\cos (141.2/2) \]

\[ C_\theta = 1.62 \]

Step 19  \[ \text{Inverts of inflow pipe (G) is less than } E_{ai}(4.69 \text{ ft) above invert} \]

\[ C_P = 0 \]

Step 20  \[ H_a = (E_{ai} - E_i)(C_B + C_\theta + C_P) = (4.69 - 4.52)(-0.05 + 1.62 + 0) \]

\[ H_a = 0.25 \text{ ft} \]

Step 21  \[ \text{Col. 17B,} \]

Col. 15A  \[ E_a = H_a + E_{ai} = 0.25 + 4.69 = 4.94 \text{ ft} \]

Step 22  \[ \text{Col. 16A} \]

\[ \text{EGL}_a = E_a + \text{BOC}_i = 4.96 + 105.43 = 112.62 \text{ ft} \]

Step 23  \[ \text{Col. 17A} \]

\[ \text{US TOC} = 108.18 + 2.0 = 110.18 \text{ ft} \]

**Structure 8**

Step 3  \[ \text{Col. 1A, 1B} \]

Str. ID = 8

Col. 2A  \[ D = 2.0 \text{ ft} \]

Col. 3A  \[ Q = 21.82 \text{ cfs} \]

Col. 4A  \[ L = 250 \text{ ft} \]

Step 4  \[ \text{Case B?} \]

\[ \text{EGL}_a > \text{TOC}_o \]

\[ 112.62 > (108.18 + 2.0) \]

\[ 112.62 > 110.18 \text{ ft} \sqrt{\text{Pressure Flow}} \]

Col. 5A  \[ V = Q/A = 21.82/(0.25\pi2.0^2) = 6.95 \text{ ft/s} \]

Col. 8A  \[ V^2/2g = \frac{6.95^2}{2(32.2)} = 0.75 \text{ ft} \]
Col. 2B  \( H_0 = 0.4(V^2/2g) = 0.4(0.75) = 0.30 \text{ ft} \)
Col. 9A  \( \text{EGL}_0 = \text{EGL}_a + H_o = 112.62 + 0.30 = 112.92 \text{ ft} \)
Col. 10A  \( \text{HGL}_0 = \text{EGL}_0 - V^2/2g = 112.92 - 0.75 = 112.17 \text{ ft} \)

Step 11  Col. 11A  \( S_f = \left( \frac{Qn}{(K_QD^{2.67})} \right)^2 = \left( \frac{(21.82)(0.013)}{\left( \left( \frac{(0.46)(2.0)}{2.67} \right) \right)^2} \right)^2 \)
\( S_f = 0.00939 \text{ ft/ft} \)

Step 12  Col. 3B  \( H_f = S_fL = 0.00939 (250) = 2.35 \text{ ft} \)
\( H_b, \ H_c, \ H_e, \ H_j = 0 \)

Col. 8B,
Col. 12A  Total pipe loss = 2.35 ft

Step 13  Col. 13A  \( \text{EGL}_i = \text{EGL}_0 + \text{Total pipe loss} = 112.92 + 2.35 = 115.27 \text{ ft} \)
Col. 14A  \( \text{HGL}_i = \text{EGL}_i + V^2/2g = 114.52 \text{ ft} \)

Step 14  Case A?
\( \text{HGL}_i \geq \text{TOC}_i \)
\( 114.52 \text{ ft} \geq (111.68 + 2.0) \text{ ft} \)
\( 114.52 \text{ ft} \geq 113.68 \text{ ft} \quad \checkmark \) Pressure Flow

Col. 6A  "FULL"
Col. 7A  "NA"

Step 15  Col. 9B  \( E_i = \text{EGL}_i - \text{BOC}_i = 115.27 - 111.68 = 3.59 \text{ ft} \)
Col. 10B  \( y + (P/\gamma) = E_i - V^2/2g = 3.59 - 0.75 = 2.84 \text{ ft} \)
Col. 11B  \( DI = \frac{Q}{(A(Dg)^{0.5})} = 21.82/\left( (0.25\pi(2.0)^2)(2.0(32.2))^{0.5} \right) \)
\( DI = 0.87 \)

Step 16a  Not supercritical flow
\( E_{ai0} = E_i + 0.2(V_o^2/2g) = 3.59 + 0.2(0.75) = 3.74 \text{ ft} \)

Step 16b  \( E_{ais} = (1.0)(DI)^2D = (1.0)(0.87)^22.0 = 1.50 \text{ ft} \)

Step 16c  \( E_{aiu} = (1.6)(DI)^{0.67}D = (1.6)(0.87)^{0.67}2.0 = 2.91 \text{ ft} \)

Col. 12B  \( E_{ai} = \max(E_{ai0}, E_{ais}, E_{aiu}) = 3.74 \text{ ft} \)

Step 17  Col. 13B  \( C_B = -0.05 \)

Step 18  Col. 14B  \( \theta_w = 140.8^\circ \)
\( C_\theta = 4.5(\sum Q_i/Q_o)(\cos \theta_w/2) \)
\( C_\theta = 4.5(21.82/21.82)(\cos (135/2)) \)
\[ C_\theta = 1.72 \]

**Step 19**  
Col. 15B  
Inverts of inflow pipe (P9) less than \( E_{ai} \) (3.86 ft) above invert  
\[ C_p = 0 \]

**Step 20**  
Col. 16B  
\[ H_a = (E_{ai} - E_i)(C_B + C_\theta + C_p) = (3.74 + 3.59)(-0.05 + 1.72 + 0) \]  
\[ H_a = 0.25 \text{ ft} \]

**Step 21**  
Col. 16A  
\[ H_a = (E_{ai} - E_i)(C_B + C_\theta + C_p) = (3.74 + 3.59)(-0.05 + 1.72 + 0) \]

**Structure 9**

**Step 3**  
Col. 1A, 1B  
Str. ID = 9  
Col. 2A  
\( D = 2.0 \text{ ft} \)  
Col. 3A  
\( Q = 21.82 \text{ cfs} \)  
Col. 4A  
\( L = 30 \text{ ft} \)

**Step 4**  
Case B?  
\( \text{EGL}_a > \text{TOC}_0 \)  
\[ 115.67 > (111.68 + 2.0) \]  
\[ 115.67 > 113.68 \text{ ft} \] ✓ Pressure Flow

**Step 5**  
Col. 5A  
\[ V = Q/A = 21.82/(0.25\pi2.0^2) = 6.95 \text{ ft/s} \]

**Step 6**  
Col. 8A  
\[ V^2/2g = \frac{6.95^2}{2(32.2)} = 0.75 \text{ ft} \]

**Step 7**  
Col. 2B  
\[ H_0 = 0.4(V^2/2g) = 0.4(0.75) = 0.30 \text{ ft} \]

**Step 8**  
Col. 9A  
\[ \text{EGL}_0 = \text{EGL}_a + H_0 = 115.67 + 0.30 = 115.97 \text{ ft} \]

**Step 9**  
Col. 10A  
\[ \text{HGL}_0 = \text{EGL}_0 - V^2/2g = 115.97 - 0.75 = 115.22 \text{ ft} \]

**Step 10**  
Col. 11A  
\[ S_f = \left(\frac{Qn}{(K_QD^{2.67})}\right)^2 = \left(\frac{21.82(0.013)}{(0.46(2.0)^{2.67})}\right)^2 \]  
\[ S_f = 0.00939 \text{ ft/ft} \]

**Step 11**  
Col. 12A  
Total pipe loss = 0.28 ft  
\( H_b, H_c, H_e, H_j = 0 \)

**Step 12**  
Col. 13A  
\( \text{EGL}_i = \text{EGL}_0 + \text{Total pipe loss} = 115.97 + 0.28 = 116.25 \text{ ft} \)
Col. 14A \[ \text{HGL}_i = \text{EGL}_i + \frac{V^2}{2g} = 115.50 \text{ ft} \]

There is a drop inlet upstream of this structure.
Chapter 8

STORAGE FACILITIES

This chapter contains details on the following:

- Storage facility types and applications
- Storage facility analysis and design

8.1 INTRODUCTION

This chapter provides guidance for the design of detention/retention storage facilities. Procedures for performing preliminary and final sizing and reservoir routing calculations are presented. Stormwater storage facilities may be designed to; handle drainage local to the highway system, address off-site flows coming to the highway, or be part of a larger regional system. The type of facility and cooperating stakeholders will affect the facility design.

8.1.1 Design Resources

Detention/Retention facility design is described in Chapter 8 of FHWA's Hydraulic Engineering Circular No. 22, Urban Drainage Design Manual (HEC 22, 2009). It is anticipated that designers of detention and retention facilities on ADOT projects will utilize HEC 22 as a primary resource. Therefore, equations, procedures, and figures from HEC 22 will not be included in this Manual.


The US Army Corps of Engineers Hydrologic Modeling System (HEC-HMS, 2013) computer program developed by the Hydrologic Engineering Center can be used to assist with storage facility routing calculations.

Guidance for use of HEC-HMS and procedures for hydrologic analysis of storage facilities is provided in Chapter 6, Storage Routing of DDM Volume 2 - Hydrology.

8.1.2 Storage Facility Use and Location

Stormwater storage facilities are used to reduce the impact of storm runoff on downstream areas. The impact concerns are mainly of quantity and/or quality of runoff. The most common design objective is a lower peak discharge. The peak discharge reduction is achieved by directing runoff into a storage facility and restricting the outflow to a controlled lower rate though a properly sized outlet. The design is performed such that the outflow hydrograph will have a lower peak discharge and a longer duration.
Stormwater storage facilities are typically classified as either detention or retention facilities. Detention facilities are those that are designed to detain runoff for some short period of time during a storm event sufficient to reduce the peak discharge. They are designed using dynamic stage-storage-discharge relationships. Retention facilities are designed to retain all of the runoff until the storm has passed. They are drained by either gravity flow or pumping.

Storage facilities are also classified on the basis of whether they are dry or wet between storm events. A “dry pond” has on outlet positioned at or below the lowest elevation in the facility, such that the stormwater drains completely between storm events. A “wet pond” has its lowest outlet at an elevation above the bottom of the facility. Water remains in the pond between storm events and is emptied only by infiltration and/or evaporation.

Detention facilities are further classified by their approach to peak flow attenuation as in-line or off-line basins. In-line basins route the entire inflow hydrograph through the basin structure. Off-line basins allow low flows to bypass the basin entirely. Flows above a set low flow discharge are diverted into the basin. Off-line basins frequently require less storage volume to achieve the same peak flow reduction as in-line basins, however, the diversion structures add complexity to the design.

The effectiveness of a storage facility depends on the amount of storage provided, its location within the system, and its operational characteristics. In addition to controlling the peak discharge from the outlet works, storage facilities will change the timing of the entire hydrograph. Therefore, it is important to understand the storage facility as a drainage structure that controls runoff from a defined area that also interacts with other drainage structures within the watershed. Multiple storage facilities located in the same watershed will affect the timing of the runoff through the conveyance system which could result in a decrease or increase in flood peaks at different downstream locations. If several storage facilities are located within a particular watershed it is important to determine the effects each facility may have on combined hydrographs in downstream locations. Effective stormwater management must be coordinated on a regional or basin-wide basis.

The evaluation of storage facilities should include comparison of the design discharge at a point or points downstream of the proposed storage facility site with and without the additional storage. This may require channel routing calculations to be extended downstream to a point where the effect of the proposed storage facility hydrograph on the existing downstream hydrograph can be assessed.

### 8.1.3 Water Quality Aspects

The analysis and design of storage facilities presented in this chapter does not directly consider water quality compliance strategies; however, once in place a retention/detention basin will provide significant benefit to water quality of the receiving stream. By containing the flow, pollutants are captured in the basin and prevented from reaching downstream regulated water bodies. Sediment is trapped in the basins and can be removed during maintenance activities, and disposed of appropriately. Attenuation of peak flow results in reduction of velocity and associated erosive forces of the flow. Storage facilities, from this perspective serve as permanent
post construction best management practices. Detailed information and literature about the design and implementation of BMPs on ADOT projects can be found in the ADOT Post Construction Best Management Practices Manual for Water Quality (BMP Manual, 2013). Additional resources are available on the ADOT Environmental Services and Planning website.

8.2 DESIGN CONCEPTS

The basic concept of detention basin design is the reduction of peak flow rates through the application of a hydrograph routing analysis for a given inflow hydrograph, storage volume, and outlet configuration. At any given time interval during the routing process one of the following conditions is occurring:

- If the average inflow rate is larger than the average outflow rate for a given time interval, the volume of water stored increases, the water surface increases, and the average outflow rate for the next time interval increases.
- If the average inflow rate is equal to the average outflow rate for a given time interval, the volume of storage and the water surface stay constant.
- If the average inflow rate is less than the average outflow rate for a given time interval, the volume of water stored decreases, the water surface decreases, and the average outflow rate for the next time interval decreases.

8.2.1 Inflow Hydrograph

The inflow hydrograph should be developed using procedures from DDM Volume 2 - Hydrology. The selection of the most effective type of storage facility for a given location will be influenced by the shape of the inflow hydrograph. The characteristics of the contributing watershed will determine the hydrograph shape. A short, steep watershed with a relatively short distance to its headwaters will generate a “peaky” hydrograph with a short duration and high peak discharge. An in-line detention basin can achieve effective peak flow reduction on this type of contributing watershed due to the small volume required. In contrast, a long, flat watershed with a large contributing area will generate a hydrograph with a relatively long time base and a large runoff volume. An off-line basin may be more effective at peak flow reduction on this type of watershed. The flow-by-discharge can be selected to pass a substantial volume of runoff at low flows, preserving the storage facility volume for shaving off the runoff near the time of peak flow. Effective detention approaches can be identified by reviewing the hydrograph shapes, particularly when tributary hydrographs from two different watersheds are combined. It may be determined that one tributary should be left to flow unimpeded and the other detained in order to create an offset in the time of peak discharge so that the peak discharge resulting from the combined hydrographs is substantially reduced.

The effect of detention on hydrograph shape and discharge is illustrated on Figure 8–1. The pre- and post-development hydrographs are the inflow hydrographs to the facility. The routed post-development outflow is the discharge hydrograph from the facility. The volume under the inflow hydrograph is the total volume of runoff entering the basin. The volume under the outflow
hydrograph is the total volume of runoff leaving the basin. The change in the magnitude and the time of peak flow is accomplished by the flow attenuation through the basin. The volume above the outflow hydrograph and below the inflow hydrograph is the volume of storage required. The maximum volume occurs when the two hydrographs intersect, which is also when the maximum outflow rate occurs.

For most detention basins the outlet is a culvert or weir structure that is uncontrolled. For a basin with an uncontrolled outlet to a free outfall, the peak storage and the peak outflow will occur at the point where the outflow hydrograph intersects the inflow hydrograph.

![Image of hydrograph and stage-storage relationship](image_url)

**Figure 8–1 Detention Routing: Inflow-Outflow Hydrograph**

### 8.2.2 Stage-Storage Relationship

The relationship between available storage volume and water depth (stage) for a particular basin configuration has a direct impact on the storage routing computation and resulting peak discharge. This relationship is defined as the stage-storage curve. The data for this type of curve are usually developed using surface modeling software and tabulating the volume between two surfaces; the first surface being the basin grading plan finished surface and the second surface being the various water surface elevations to be tabulated, typically every foot or fraction of a foot of depth. In this way a table can be developed for the full range of storage depths (stages) showing the total storage volume at each stage. The relationship can also be developed manually using a topographic map and one of the following formulas: the average-end area, frustum of a
pyramid, or prismoidal formulas. Formulas for applying these manual methods are presented in HEC 22. The average-end area formula is usually preferred as the manual method to be used on non-geometric areas. This is usually developed from the stage versus surface area data. The information should extend from the basin invert to the top of the embankment. The precision of the calculation increases as the interval of stage decreases.

8.2.2.1 Retention

A stage-storage relationship is not necessary for a retention basin since a dynamic routing of the inflow hydrograph is not performed. A retention basin is sized for the entire volume of runoff entering the basin for the design storm. The basin is drained slowly after the storm has passed. A particular use for retention basins is for first flush retention of runoff as a water quality measure. For first flush applications the first flush runoff should be captured and stored in the basin while the runoff in excess of the first flush is allowed to continue downstream.

First Flush

First Flush means the minimum level of control at which stormwater pollution prevention practices must be put in place. The “first flush” standard consists of retaining or treating the first 0.5 inches of direct runoff from a storm event. The first flush requirement can be addressed by: (1) retaining the required minimum first flush volume, (2) treating the first flush discharge, or (3) utilizing a combination of both approaches.

- The minimum First Flush volume, \(V_{FF}\), is calculated as \(V_{FF} = \frac{P}{12}A\) where: \(V_{FF}\) = minimum First Flush volume in acre-feet, \(P\) = first 0.5 inches of direct runoff, and \(A\) = area of project site, in acres.

The project site for area calculations should include the area disturbed by construction tributary to the first flush storage facility.

8.2.2.2 In-Line Detention

In developing the design of an in-line detention facility it is often useful to have a preliminary estimate of the storage volume required to achieve a particular peak outflow target discharge. This will guide the design of an initial grading plan and reduce the number of grading plan iterations necessary to complete the design. Methods for developing a preliminary estimate of the storage volume required for peak flow attenuation are presented in HEC 22. The Triangular Hydrograph method replaces the actual inflow and outflow hydrographs with the standard triangular shapes shown in Figure 8–2 below.
Figure 8–2 Triangular Shaped Hydrographs

The required storage volume may be estimated from the area between the outflow hydrograph and the inflow hydrograph.

Once a grading plan is developed from the preliminary volume estimate the stage-storage curve can be developed for use in the hydrograph routing computations. Several iterations and refinements to the grading plan and stage-storage relationship are typically required to meet the design outflow objective.

8.2.2.3 Off-Line Detention

Development of an initial estimate of required storage volume for an off-line detention facility can be as simple as computing the volume of the inflow hydrograph above the target flow-by-discharge. This can be done initially with the triangular hydrograph described above, or it can be estimated by performing a hydrograph diversion in the HEC-HMS hydrology model, if one has been developed for the watershed. Once the initial volume estimate is completed, the basin grading plan is developed similar to a retention basin since the runoff diverted into the off-line basin will be stored and drained slowly after the storm event with no dynamic hydrograph routing procedure required.

Once the final diversion structure design is completed a diversion rating curve will need to be developed for the flow being diverted into the off-line basin. The hydrograph routing will then need to be updated with the diversion rating curve to determine the volume of water diverted.
into the basin and the flow-by peak discharge based on the actual design. Since the initial volume estimate was for a constant discharge, the diversion structure should be designed to minimize the change in stage between the minimum and maximum diverted flows.

![Figure 8–3 Off-Line Basin Hydrographs](image)

**8.2.3 Stage-Discharge Relationship**

A stage-discharge curve defines the relationship between the depth (stage) of water and the discharge or outflow from a storage facility. This curve is also called a *rating curve*. A typical storage facility has two outlets: principal and emergency. The principal outlet can be a culvert, weir, or spillway. The emergency outlet is typically a spillway or a planned low spot in the basin perimeter grading for a planned overflow of the facility. The rating curve should take into account the discharge characteristics of both the principal and emergency outlets. The individual rating curves for the principal and emergency spillways are added together to generate the composite rating curve.

In addition to the operational design flow, the composite rating curve should be used to evaluate flows in excess of the design flow that might be expected to pass through the storage facility (i.e., 100-year flood), especially for the emergency spillway.
Tailwater influences and structure losses must be considered when developing discharge rating curves. If a combination of outlet structures is used, backwater effects of one structure may affect the discharge of the combination of structures. HEC 22 presents methods for calculating the stage-discharge relationship for weirs. Chapter 4 presents methods for calculating the stage-discharge relationship for structures having an HW/D ratio greater than 1.2 or affected by tailwater.

8.2.4 Debris and Access Control

Debris in various forms must be considered in the design of storage facilities. In rural areas sediment and/or vegetative debris may be washed into the basin by the stormwater runoff. In urban areas, landscape material or trash may accumulate. As the velocity drops to near zero with the flow expansion into the basin the debris may settle out of the flow and reduce the storage volume of the basin. Floating debris may make its way to the outlet and reduce the outlet capacity. In areas of known sediment issues, providing a sediment storage volume in addition to water storage volume may be prudent. For floating debris, use of trash racks at culvert inlets should be considered with caution. In most highway applications it is preferable to avoid trash racks and let floating debris pass through the structure. Trash racks may increase the potential for plugging and should only be used when other considerations, such as access control, warrant their use. Access control is often considered at the inlet to a long culvert or storm drain system.

Protective treatment may be required for facilities that present a potential hazard to the public. Safety considerations include limiting the maximum depth and/or including ledges and mild slopes to prevent people from falling in and to facilitate their escape from the basin. Access control by fencing the basin may be required for detention areas where one or more of the following conditions exist:

- Water depths exceed 3-feet for more than 24-hours.
- A low-flow watercourse or ditch passing through the detention area has a depth greater than 0.5 ft or a flow velocity greater than 5 ft/s.
- Side slopes steeper than 2.5:1.

8.3 DESIGN PROCEDURES

The basic concept involved in stormwater detention analysis is a routing procedure using the storage-indication method to transform the inflow hydrograph into the outflow hydrograph. To perform the calculations the following data are required:

- Inflow hydrograph for all selected design storms (Normally derived by use of HEC-HMS).
- Stage-storage curve for the proposed storage facility (see Section 8.2.2).
- Stage-discharge curve for all outlet structures (see Section 8.2.3).

Using these data, a routing procedure is used to route the inflow hydrograph through the storage facility in an iterative process using different basin and outlet geometry until the desired outflow hydrograph is achieved.
8.3.1 Develop Inflow Hydrograph

The hydrograph reaching the basin site will generally be developed using HEC-HMS and procedures from DDM Volume 2 - Hydrology. At the basin site, inlet structures may be designed to modify the hydrograph by creating a flow-by and diverted hydrograph into an off-line basin or the entire hydrograph may be routed directly into an In-line basin. Inlet structures may be necessary where the inflow is being dropped into the basin. Riprap or other energy dissipation measures may be necessary. If by-pass flows are desired for an off-line basin, then a diversion structure is necessary in the upstream channel. Diversion structures are of two types. One type diverts a fixed percentage of the approach flow. This type of diversion is accomplished using a flow splitter. The second type diverts the discharge above a base flow. Flows below the base level are not captured into the detention basin. Diversion of flows above a base level is more effective at reducing the required size of storage.

8.3.2 Establish Target Discharge

In some cases, a target discharge may be based on a specific criteria or regulation that must be met. In other cases there may not be a specific target discharge and the storage facility design will be based on optimizing the basin effectiveness vs. cost for the specific site conditions encountered. This section will describe the target discharge based on the following potential scenarios:

- A specific target discharge exists based on conditions external to the project,
- The design is constrained by the size of the available basin site with no external restriction on target discharge, or,
- No specific constraint on basin size or target discharge is imposed.

8.3.2.1 Specific Target Discharge

A specific target discharge for storage facilities may be based on:

- Historic discharge for specific design conditions (i.e., the post-development peak must equal the pre-development peak for a specified frequency of occurrence),
- Limiting discharge based on the capacity of the downstream drainage system, or
- A specified value set by jurisdictional regulation or agreement (such as determined by joint project agreements for downstream improvements).

When a target discharge is based on one of these constraints, the outlet structure can be sized according to the target discharge and the required storage volume will be determined from the difference between the inflow and outflow hydrograph as described in Section 8.2.

8.3.2.2 Basin Size Constraint

When a specific target discharge is not imposed, the design should optimize the basin’s effectiveness vs. cost. In some cases a storage facility site conforms to an existing available land parcel, a traffic interchange infield area, or some other limiting constraint in available land area.
In this case, a grading plan can be developed to fully utilize the available site. This should take into account needs for setbacks, access, multiple uses, and design constraints such as maximum depth, allowable side slope, grading of the basin floor, and gravity outfall elevation. When a grading plan is developed that maximizes the storage volume subject to these constraints, an outfall can be sized to restrict the inflow hydrograph such that the routed water surface elevation in the basin fully utilizes the available storage. The maximum discharge from the resulting outfall sizing becomes the “target discharge”.

8.3.2.3 Unconstrained Basin Size

When an externally dictated target discharge is not imposed and the available land area for a basin is not restricted, the reduction in peak discharge per acre of land area utilized can be used as a good measure of the basin effectiveness. This measure considers that there is a land cost to be balanced against the reduction in peak discharge. For a given inflow hydrograph there will be a combination of storage volume and outlet structure size that maximizes the peak flow reduction per acre of land used (cfs/acre). To find this combination, alternative grading plans can be developed varying in size from small to large. HEC-HMS can be used to size an outlet to fully utilize the available storage volume as previously described. The resulting peak discharge reduction per acre can be plotted for each basin concept to show the combination of storage volume and peak discharge reduction that maximizes the peak flow reduction per acre of land used. This measure is the Basin Effectiveness Ratio and is defined as follows:

\[
ER = \frac{(Q_i - Q_o)}{A_B}
\]

8.1

where:
- \(ER\) = Basin Effectiveness Ratio (cfs/acre),
- \(Q_i\) = Peak discharge of inflow hydrograph (cfs),
- \(Q_o\) = Peak discharge of basin outflow hydrograph (cfs), and
- \(A_B\) = Land area for basin site (acres).

The following will help to illustrate the concept:

- If a large watershed with a high peak discharge and runoff volume is routed through a relatively small basin area with a small storage volume, the outlet size required to avoid overtopping will be large and there will be a small peak flow reduction.
- As grading plans for larger storage volumes are developed, smaller outlet sizes can be used to avoid overtopping. As a result, the peak flow reduction will increase and required land area will increase. Initially, the peak flow reduction will increase as compared to basin area resulting in a greater “effectiveness ratio”.
- At some point as the basin volume gets larger and the resulting outlet gets smaller, the land area “cost” to achieve marginal additional reductions in peak discharge will cause the Basin Effectiveness Ratio to begin to decrease meaning that further peak flow reduction is not as effective.
- If similar effectiveness ratios exist for a range of basin sizes, the basin configuration with the largest peak flow reduction that is still considered effective should be selected.
With this approach to basin development, factors that contribute to ineffective storage become apparent. These factors may include steep land slopes that result in “dead” storage at the uphill side of the basin that is required for excavation of the basin but has shallow ponded depths. The effectiveness ratio may decrease as these areas are added to the basin.

8.3.3 Design Development

Given an inflow hydrograph, the design of a detention basin involves iterations and refinement of the size and interaction of the basin outlet and grading plan to meet the design objective. As described previously, the design objective may be to meet a target discharge imposed by regulatory or downstream constraints or it may be to optimize the basin effectiveness based on site constraints and opportunities. The following sections describe the steps in the iterative process.

8.3.3.1 Storage Type Selection

The selection of in-line or off-line detention basin type will dictate the remaining steps in the design. An in-line basin will require a hydrologic routing using the stage-storage-discharge relationships developed for the site. An off-line basin will require a more simplified hydrologic routing to divert the portion of the hydrograph that will be directed into the basin. The flow-by amount will be routed downstream and the volume of flow diverted into the basin will be held until after the storm.

The inlet structure design for an In-line basin may include a spillway if the inlet channel or stream is above the basin bottom elevation or a headwall and energy dissipator if the inflow is from a culvert or storm drain situated near the basin floor. These inlet structures will not impact the inflow hydrograph.

The inlet structure for an off-line basin will likely involve detailed hydraulic design of a splitter structure and a hydrologic splitting of the inflow hydrograph to separate the basin inflow hydrograph from the flow-by hydrograph. The analysis of splitter structures can be complex and their performance in the field is not always reliable due to debris or sediment deposition or other unanticipated field conditions. A culvert or other flow restriction is sometimes used to create a backwater pool paired with a side weir to allow flows above the flow-by target to spill into the basin.

Since an in-line basin does not rely on the performance of a splitter structure, it may be useful to develop an in-line concept first to see if the design objective can be met. If sufficient storage volume cannot be provided for an in-line basin, the off-line concept can then be developed as an alternative.
8.3.3.2 Outlet Size/ Configuration

The outlet structure allows flows to discharge from the storage basin at a controlled rate. Outlet works selected for storage facilities typically include a principal outlet and an emergency overflow spillway. The principal outlet is intended to convey the design storm without requiring flow to enter an emergency spillway.

For in-line basins a composite stage-discharge relationship must be developed for all outlet structures as described in Section 8.2.3.

Outlets can be designed in a wide variety of configurations. Outlet works may be sized to vary the outflow with varying depth. Outlet works can take the form of combinations of drop inlets, pipes, weirs and orifices at various levels. Slotted riser pipes are discouraged because of potential clogging. If the outlet is a pipe through an embankment, then an anti-seepage collar should be provided to minimize piping by water leakage of the soil particles surrounding the pipe. Design relationships, equations, and example applications are presented in HEC 22 for the following outlet controls:

- Orifices
- Sharp Crested Weirs
- Broad Crested Weirs
- V-Notch Weirs
- Proportional Weirs
- Discharge Pipes

8.3.3.3 Basin Configuration

The construction of storage facilities usually requires excavation or placement of earthen embankments to obtain sufficient storage volume. Storage facilities situated above ground may become subject to the Arizona Revised Statutes (A.R.S.) 45-1201 Dams and Reservoirs as administered by the Arizona Department of Water Resources (ADWR). Structures that have embankment heights greater than 6 feet and storage volumes greater than 15 ac-ft fall under the jurisdiction of the ADWR and are subject to stringent design, review, and ongoing maintenance and reporting requirements. See Appendix 8A for ADWR jurisdiction criteria. It is ADOT’s goal not to construct or own any facilities that come under ADWR jurisdiction. ADOT prefers to create storage facilities by excavation below existing ground.

A basin may have multiple levels: one level may be used to hold the smaller storms, and a second level, which is rarely inundated may be used for other purposes and to store the larger storms. Other considerations when setting depths include flood elevation requirements, public safety, land availability, land value, present and future land use, water table fluctuations, soil characteristics, maintenance requirements and required freeboard. Aesthetically pleasing features are also important in urban areas.
Storage facilities shall be designed to address the following maintenance concerns:

- Maintenance of fences and perimeter plantings
- Grass and vegetation maintenance
- Bank deterioration
- Blockage of outlet structures
- Litter accumulation
- Weed growth
- Standing water or soggy surfaces
- Sedimentation control
- Mosquito control

An operation and maintenance road (O&M Road) should be provided at the top of the basin to provide access around the entire perimeter of the basin. Invert access ramps should also be provided at locations near significant inlet or outlet structures. To minimize rill erosion on the basin side slopes, runoff entering the basin should only be allowed to enter the basin at planned locations with erosion protection. O&M roads should be graded with a cross-slope directed away from the top of the basin such that local runoff is captured by a swale adjacent to the O&M road. The swale should drain to planned inlet locations such as a spillway, pipe inlet, or catch basin. Generally, a minimum of 16-feet is preferred between the top edge of the basin and the right-of-way line to allow for the O&M road.

Areas above the normal high water elevations of storage facilities should be sloped to allow drainage to flow in the natural flow path away from the basin or into a swale to a planned inlet location and to prevent standing water. Careful finish grading is required to avoid creation of upland surface depressions that may retain runoff.

Side slopes are limited to those that are compatible with the landscape treatment.

The bottom area of storage facilities should be graded toward the outlet to prevent standing water conditions. A minimum 1% bottom slope is recommended. A low flow or pilot channel constructed across the facility bottom from the inlet to the outlet is recommended to convey low flows, and prevent standing water conditions. If sediment deposition is expected, an initial sedimentation trap basin with access should be considered.

### 8.3.3.4 Hydrograph Routing

The routing calculations are based on level-pool storage routing and are typically performed using HEC-HMS software as described in Chapter 6 of the DDM Volume 2 - Hydrology. The fundamental principle is the conservation of mass, i.e., the change in storage during a computational time interval is equal to the difference between the inflow and outflow volumes. The method described below is a finite difference solution of the conservation of mass relationship, it is commonly known as the storage-indication method but is referred to as level-pool storage routing in HEC-HMS. The calculations are the process of analyzing the difference between the flow entering and the flow leaving the basin for a series of computational time increments. The difference determines the change in volume and water surface elevation.
As stated previously, to perform the calculations one must have the inflow hydrograph, the stage-storage and stage-discharge relationships. The computation begins with inflow. At a given time interval, the inflow is known. Using the storage at the end of the previous time interval, add the inflow volume from the current time interval to get the new Stage, using the new stage get the outflow rate. Use the outflow rate to get the new outflow volume. Use the beginning of period volume plus the inflow volume minus the outflow volume from the current period to determine the end of period volume. The next time interval is then calculated. A detailed description and example showing manual calculations for the generalized routing procedure is contained in HEC 22. An example of HEC-HMS storage routing for a box culvert roadway crossing embankment is contained in DDM Volume 2 - Hydrology. The example shows the development of a composite stage-discharge relationship combining the principal outlet (box culvert) and the emergency spillway (roadway overtopping).

8.3.3.5 Review Results

Once the HEC-HMS model is assembled, the stage-discharge and stage-storage relationships can be easily modified to simulate various alternatives as the design iterations are prepared. With the results from each simulation the results can be reviewed and evaluated to determine which modifications to the design would be most effective in achieving the design objective. Important results for review are the maximum water surface elevation in the basin, the maximum storage volume in the basin, and the peak discharge leaving the basin. With this information and the inflow hydrograph, the Basin Effectiveness Ratio can be computed. If the results are not satisfactory, the process just described should be repeated beginning with review of the storage type that was selected, the outlet size and configuration, the associated storage volume and grading concept.

8.3.4 Design Details

Once an acceptable design is achieved, the concept can move into final design. Typically there will be further refinements during final design and construction plan preparation. It is important that the final design details be incorporated into the hydrology model to ensure that the analysis provided in the drainage report matches the design shown on the plans.
8.4 REFERENCES


APPENDIX 8A

ADWR JURISDICTIONAL DAM CRITERIA
Figure 8–4 ADWR Jurisdictional Dam Criteria
APPENDIX 8B
HEC-HMS HYDROGRAPH FOR RATIONAL BASIN
Example No. 8-1  HEC-HMS Hydrograph for Rational Basins

Problem:
There are times one will need a full 24-hour hydrograph for a basin that is analyzed using the Rational Method. The suggested procedure is described below. The procedure is a two-step process of first matching the volume of runoff and then matching the peak discharge. The procedure is demonstrated using the data from the Youngtown Basin.

Given:
- Area = 0.13 sq. mi. = 83.2 acres
- Length = 4420 feet = 0.84 mi.
- Urban watershed, \( K_b = 0.025 \) with \( \frac{1}{4} \) acre lots
- For 100 year rainfall, \( P_{24} = 3.8 \) in. \( C = 0.73 \)

Solution:
Assume slope = 1% = 52.8 ft/mi
\[
T_c = 11.4 L^{0.5} K_b^{0.52} S^{-0.31} i^{-0.38}
\]
\[
T_c = 11.4 (0.84)^{0.5} (0.025)^{0.52} (52.8)^{-0.31} i^{-0.38}
\]
\[
T_c = 0.488/i^{-0.38}
\]
Solving for \( T_c = 0.216 \) hr = 13 min., \( i = 6.8 \) in/hr.
\[
Q = CI A = (0.73)(6.8)(83.2) = 413 \text{ cfs}
\]
Volume = \( CP_{24} A = (0.73)(3.8/12)(83.2) = 19.2 \) ac-ft

For HEC-HMS with \( T_c = 0.216 \) hr,
\[
R = 0.37T_c^{1.11} L^{0.8} A^{-0.57}
\]
\[
R = 0.37(0.216)^{1.11}(0.84)^{0.8}(0.13)^{-0.57}
\]
\[
R = 0.19
\]
\[
CP_{24} = (0.73)(3.8) = 2.77
\]

HEC-HMS results adjust % impervious to get Runoff ≈ 2.77:
- Try 70% impervious
  - Runoff = 2.73 in, \( Q_p = 280 \) cfs
- Try 75% impervious
  - Runoff = 2.91 in, \( Q_p = 296 \) cfs
- Try 71% impervious
  - Runoff = 2.77 in, \( Q_p = 283 \) cfs
Use 71% impervious and adjust $T_c$ and $R$ for $Q \cong 413$ cfs

Try $T_c = 0.15$ and $R = 0.14$: $Q_p = 326$ cfs

Try $T_c = 0.10$ and $R = 0.09$: $Q_p = 390$ cfs

Try $T_c = 0.07$ and $R = 0.06$: $Q_p = 414$ cfs, Use for hydrograph.
Chapter 9

PUMP STATIONS

This chapter contains details on the following:

- Pump station design criteria and philosophy
- Pump station types and preferences

9.1 INTRODUCTION

Stormwater pump stations are used to remove stormwater from highway sections that cannot be drained by gravity. Because of their high construction and operational cost and the potential cost of malfunctions, their use should be limited to where no other system is feasible. Stormwater pump stations generally have high short-term capacity requirements and infrequent use, making them very costly on a per-use basis. The cost of a gravity system must be compared with the life-cycle costs (construction, maintenance and operation) of the pumping station. A pumping station should only be used when the life-cycle cost of the pump station is demonstratively less than the alternative gravity system. Due to high maintenance requirements the selection of pump station components should be coordinated with District maintenance staff.

9.1.1 Design Resources

The design of pump stations is described in FHWA’s Hydraulic Engineering Circular No. 24, Highway Stormwater Pump Station Design (HEC 24, 2001). HEC 24 includes all the applicable charts, equations, and theory needed for pump station analysis and design. It is anticipated that pump station designers will utilize HEC 24 as a primary resource; therefore, equations, procedures, and figures from HEC 24 are not included in this chapter:


9.1.2 Hydraulic Analysis

The hydraulic analysis of a pump station involves the interrelationship of 3 components:

- The inflow hydrograph,
- The storage capacity of the wet well and the outside storage, and
- The discharge rate of the pumping system.
The inflow hydrograph is determined by the physical factors of the watershed and regional climatological factors. The discharge rate of the pump station is often controlled by outfall capacity. Therefore, the main objective in pump station design is to store enough inflow (volume of water under the inflow hydrograph) to allow station discharge to meet specified limits while not exceeding the freeboard requirements at the inlets. A minimum volume of storage is required to prevent excessive pump cycling. Even where there are no limitations to pump station discharge, a design that balances storage and pumping capacity provides the most economical design since storage permits use of smaller and/or fewer pumps.

**9.2 DESIGN CONSIDERATIONS**

There are many choices to be made regarding pump stations for management of storm water stormwater along highways; some are driven by topography and the interaction of collection and discharge systems, others are related to construction and operating costs. Some of the areas to be considered are shown in Table 7–4 below and discussed further in this chapter.
<table>
<thead>
<tr>
<th>Table 9–1 Pump Station Design Considerations</th>
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<tr>
<td><strong>Operational/Mechanical Design</strong></td>
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<tr>
<td>Station Type:</td>
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<tr>
<td>Wet Pit</td>
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<tr>
<td>Dry Pit</td>
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<tr>
<td>Pumps:</td>
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<tr>
<td>Power &amp; Back-up Systems</td>
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<tr>
<td>Type</td>
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<tr>
<td>Drive</td>
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<td>Operational Size and Volume Requirements</td>
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<td>Side Clearances</td>
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<td>Cycling Plan</td>
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<td>Hours, Number of Starts</td>
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<td>Solids Handling Capacity</td>
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<td>Operation Plan/Storage</td>
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<td>Intake System:</td>
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<td>Trash Racks</td>
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<td>Inlet type</td>
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<td>Discharge:</td>
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<td>Force Main vs. Gravity</td>
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<td>Flap Gates</td>
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<td>Safe Workspace Environment:</td>
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<td>Potable water supply</td>
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<td>Ventilation</td>
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<td>Confined Space Hazard sensors</td>
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Many of the decisions regarding the considerations listed in Table 9–1 are currently based on engineering judgment and experience. To ensure cost-effectiveness, the designer should assess each choice and develop economic comparisons of alternatives on the basis of annual cost. However, some general recommendations can be made which will help minimize the design effort and the cost of these expensive drainage facilities. These are discussed in the following pages of this chapter.

In the development of the Valley Freeway System in the Phoenix area, ADOT has adopted/standardized a design approach applicable to urban freeway pump stations. The station is based on a peak pump capacity that consists of multiples of 50 cfs pumps as needed plus a small 5 or 10 cfs pump to drain the wet well. The elements of this standardized design will be presented as appropriate. Discussions with ADOT maintenance personnel should be conducted early in the design process.

9.2.1 Location

Economic and design considerations dictate that the pump station be located relatively near the low point of the highway. Hopefully a frontage road or overpass is available for easy access to the station. The station and access road should be located on high ground so that access can be obtained if the highway becomes flooded. Soil borings should be made during the selection of the site to determine the allowable bearing capacity of the soil and to identify any potential problems.

Decisions regarding location and building architecture need to be made during the concept phase so station design can proceed in an efficient manner. Following are some items that are to be considered for locating the pump station site:

- Building and site access is safe and monitorable.
- Pump stations shall be situated so that the facility is accessible during storm events up to the 100-year event.
- Ample parking and working areas should be provided adjacent to the station for maintenance and repair vehicles.
- The station facade and grounds are aesthetically compatible with the surrounding area.

9.2.2 Pit Types

Basically, there are two types of stations: wet-pit and dry-pit.

9.2.2.1 Wet-Pit Stations

In the wet-pit station, the pumps are submerged in a wet well or sump with the motors and the controls located overhead. With this design, the stormwater is pumped vertically through a riser pipe. The motor is commonly connected to the pump by a long drive shaft located in the center of the riser pipe. See Figure 9–2 for a typical layout.
9.2.2.2 Dry-Pit Stations

Dry-pit stations consist of two separate elements: the storage box or wet well and the dry well. ADOT does not typically use dry-pits in pump station designs. Stormwater is stored in the wet well which is connected to the dry well by horizontal suction piping. Centrifugal pumps are usually used. Power is provided by either close-coupled motors in the dry well or long drive shafts with the motors located overhead. The main advantage of the dry-pit station for stormwater is the availability of a dry area for personnel to perform routine and emergency pump and pipe maintenance. See Figure 9–3 for a typical layout.

9.2.2.3 Pit Size

In the wet pit, the plan area must accommodate the distance between the pumps, the clearance to the sidewalls, the flow length approaching the pump, and the minimum distance from floor to the underside of the bell. The station depth should be minimum. No more depth than that required for pump submergence and clearance below the inlet invert is necessary, unless foundation conditions dictate otherwise. For the design event, typically the highway operational frequency, the top water surface shall be no higher than that which will result in a hydraulic grade line at the critical inlets with a freeboard of 0.5 foot to the gutter elevation.

9.2.2.4 Method of Construction

The construction method employed has a major impact on the cost to construct a new pump station. The annual operating costs are usually insignificant compared to construction costs. Therefore, the type of construction should be chosen carefully between open-pit construction or braced construction. Soil conditions are the primary factor in selecting the most cost-effective type of construction.

9.2.2.5 Recommendation for Design

Since dry-pit stations are as much as 60% more expensive than wet-pit stations, wet-pit stations are recommended.

9.2.2.6 ADOT Freeway Design Practice

ADOT practice is to use a wet well design. Wet well sizing is based on guidelines from the Hydraulic Institute Standards. The wet-well must be sealed from the engine room and control room. The wet well area will include explosion proof lighting. A gas tight seal shall be provided between the wet well and the engine area. Access to the wet well will be by a separate enclosed stairway that has a push/pull intake and exhaust system.
Figure 9–2 Typical Wet-Pit Station
9.2.3 Design Capacity

9.2.3.1 Frequency

The design capacity of a pump station is determined by its location within the drainage system. It is ADOT practice that pump stations and appurtenant storage system for draining roadway sumps that are considered “depressed” shall accommodate the inflow for a 50-year storm event in a manner that meets the design spread criteria. It is desirable to evaluate the total drainage system for the 100-year storm event to determine the extent of flooding and the associated risk.

9.2.3.2 Contributing Area

Hydrologic design should be based on the ultimate development of the area that must drain to the station. Every consideration should be made to keep the contributing drainage area as small as possible. Water that originates outside of the depressed areas should not be allowed to enter the depressed areas because of the need to pump all of this water. Only flows that cannot be by-
passed or passed through should be collected. The contributing drainage area should be isolated to prevent off-site flows being diverted to the pump station.

9.2.3.3 Storage

Storage, in addition to that which exists in the wet well, should be evaluated at all pump station sites. For most highway pump stations, the high flows of the inflow hydrograph will occur over a relatively short time. Additional storage will greatly reduce the peak pumping rate required. An economic analysis can be used to determine the optimum combination of storage and pumping capacity. Since most highway-related pump stations are associated with a localized low point, it is not reasonable to consider above ground storage. The simplest form of storage for these depressed situations is either the enlargement of the collection system or the construction of an underground storage facility or a combination of the two. These can typically be constructed under the roadway area and will not require additional right-of-way.

When storage is used to reduce peak flow rates, a routing procedure must be used to design the system. To determine the discharge rate, the routing procedure integrates three independent elements: the inflow hydrograph, the stage-storage relationship, and the stage-discharge relationship.

9.2.4 Pump Types

9.2.4.1 Flow Types

The most common types of stormwater pumps are axial flow (propeller), radial flow (impeller) and mixed flow (combination of the previous two). Each type of pump has its particular merits. It is difficult to have a totally objective selection procedure. Cost, reliability, operating and maintenance requirements are all important considerations when making the selection. First costs are usually of more concern than operating costs in stormwater pump stations since the operating periods during the year are relatively short. Ordinarily, first costs are minimized by providing as much storage as possible, with two or three small pumps.

Axial Flow Pumps - Axial flow pumps lift the water up a vertical riser pipe; flow is parallel to the pump axis and drive shaft. They are commonly used for low head, high discharge applications. Axial flow pumps do not handle debris particularly well because the propellers will bend or possibly break if they strike a relatively large, hard object. Also, fibrous material will wrap itself around the propellers.

Radial Flow Pumps - Radial flow pumps utilize centrifugal force to move water up the riser pipe. They will handle any range of head and discharge, but are the best choice for high head applications. Radial flow pumps generally handle debris quite well. A single vane, non-clog impeller handles debris the best because it provides the largest impeller opening. The debris handling capability decreases with an increase in the number of vanes since the size of the openings decrease.

Mixed Flow Pumps - Mixed flow pumps are very similar to axial flow except they create head by a combination of lift and centrifugal action. An obvious physical difference is the presence of the
impeller "bowl" just above the pump inlet. They are used for intermediate head and discharge applications and handle debris slightly better than propellers. These pumps can be driven by motors or engines housed overhead or in a dry well or by submersible motors located in a wet well. Submersible pumps frequently provide special advantages in simplifying the design, construction, maintenance and, therefore, cost of the pumping station. Use of anything other than a constant speed, single stage, single suction pump would be rare. Pumps shall be capable of handling solids 3” or smaller.

9.2.4.2 Submergence

Submergence is the depth of water above the pump inlet necessary to prevent cavitation and vortexing. It varies significantly with pump type, speed, inlet bell diameter and atmospheric pressure. This dimension is provided by the pump manufacturer and is determined by laboratory testing. A very important part of submergence is the required net positive suction head (NPSH) because it governs cavitation. The available NPSH should be calculated and compared to the manufacturer’s requirement. Additional submergence may be required at higher elevations. As a general rule, radial flow pumps require the least submergence while axial flow pumps require the most.

One popular method of reducing the submergence requirement (and therefore the station depth) for axial and mixed flow pumps, when cavitation is not a concern, is to attach a suction umbrella. A suction umbrella is a dish-shaped steel plate attached to the pump inlet that improves the entrance conditions by reducing the intake velocities. Required submergence criteria developed by the Hydraulic Institute is shown in Figure 9–4 as presented in American National Standard for Pump Intake Design (HI 9.8, 2012).
9.2.4.3 Standby/Spare Pumps

Considering the short duration of high inflows, the relative infrequency of the design storm event, the odds of a malfunction and the typical consequences of a malfunction, spare or standby pumps are not warranted in stormwater applications. If the consequences of a malfunction are particularly critical, it is more appropriate to add another main pump and reduce their size accordingly.
9.2.4.4 Sump Pumps

It is ADOT preference for freeway installations to use a station electric sump pump that has approximately 1/5 to 1/10 the capacity of the primary pumps in order to dewater the wet well. The pump shall be capable of handling up to three-inch solids.

9.2.4.5 ADOT Freeway Design Practice

ADOT preference is to have a small submersible sump pump (5-10 cfs) with multiple 50 cfs mixed flow primary pumps. The number of 50 cfs pumps is dependent on the inflow hydrograph.

Use a long shaft vertical turbine. The pump assembly consists of a bowl assembly with impeller, a long shaft, and a pipe column with an attached elbow. The long shaft will require lubrication at each bearing. This is to be provided by an automatic system that provides lubricant at each start.

9.2.5 Power

Several types of power may be available for a pump station. Examples are electric motors, and diesel or natural gas engines. The maintenance engineer should be contacted for input in the selection process. The designer should select the type of power that best meets the needs of the project based on an estimate of future energy considerations and overall station reliability. A comparative cost analysis of alternatives is helpful in making this decision. The need for backup power is dependent upon the consequences of failure. The decision to provide it should be based on economics and safety.

9.2.5.1 Electric Motors - Back-up/Standby Power

For electric motors, two independent electrical feeds from the electric utility with an automatic transfer switch may be the cost-effective choice when backup power is required. A standby generator is generally less cost-effective because of its initial costs. Also, standby generators require considerable maintenance and testing to ensure operation in times of need.

For extensive depressed freeway systems involving a number of electric motor-driven stations, a mobile generator may be the cost-effective choice for backup power. A skid-mounted generator can be stored at any one of the pump stations. If a power outage occurs, maintenance forces can move the generator to the affected station to provide temporary power. If a mobile generator is used as the source of backup power, it may be necessary to add additional storage to compensate for the time lag that results in moving the generator from site to site. This lag can typically be 1.0 to 1.5 hours from the time the maintenance forces are notified.

Consideration should be given to whether the pump station is to have standby power (SBP). If it is preferred that stations have a SBP receptacle, manual transfer switch and a portable engine/generator set, then the practical power limit of the pumps becomes 55 kW (75 HP) since this is the limit of the power generating capabilities of most portable generator units.
9.2.5.2 ADOT Freeway Design Practice

Natural gas or diesel engines driving an electric generator are preferred. The standard engine is non-turbocharged, water-cooled, operating at 1200 rpm. Engine exhaust must be equipped with mufflers and be directed in a way to reduce noise impacts to residences and/or businesses. A minimum clearance of 4-feet around all pumps, engines, and other mechanical equipment shall be provided in the station design.

9.2.6 Monitoring and Control

Pump stations are vulnerable to a wide range of operational problems from malfunction of the equipment to loss of power. Monitoring systems such as on-site warning lights and remote alarms can help minimize such failures and their consequences.

Telemetering is an option that should be considered for monitoring critical pump stations. Operating functions may be telemetered from the station to a central control unit. This allows the central control unit to initiate corrective actions immediately if a malfunction occurs. Such functions as power, pump operations, unauthorized entry, explosive fumes and high water levels can be monitored effectively in this manner. Perhaps the best overall procedure to ensure the proper functioning of a pump station is the implementation of a regular schedule of maintenance conducted by trained, experienced personnel.

Instruments such as hour meters and number-of-starts meters should be used on each pump to help schedule maintenance. Input from maintenance forces should be a continuous process so that each new generation of stations will be an improvement.

9.2.6.1 Water-Level Sensors

The water-level sensors activate the pumps and, therefore, are a vital component of the control system. There are a number of different types of sensors that can be used. Types include the float switch, electronic probes, ultrasonic devices, mercury switch and air pressure switch. The location or setting of these sensors control the starting and stopping of the pump motors. Their function is critical because the pump motors or engines must not start more frequently than an allowable number of times per hour (i.e., the minimum cycle time) to avoid damage. To prolong the life of the motors, sufficient volume must be provided between the pump start and stop elevations to meet the minimum cycle time requirement. The on-off setting for the first pump is particularly important because it defines the most frequently used cycle.

9.2.6.2 ADOT Freeway Design Practice

Control System

A control room separate from the wet well or engine/pump room shall contain the control equipment. The control room is completely isolated from both the wet well and the engine room. It is separately air-conditioned. The room shall be temperature controlled. Should the station
electric power fail, the stand-by generator will be activated, and the generator will continue to drive the control room air-conditioner, as well as the other ventilation systems.

The sump pump will always be the first pump on and the last pump off. The primary pumps will be consecutively lagged as the water level rises in the wet well. The order of lagging shall be by manual selection.

Uninterruptible Power Supply (UPS)

UPS shall be provided to furnish power for controlling and monitoring the pump station. Batteries sized for one-half hour outage at full capacity will supply power to the UPS. A signal from the UPS is provided to start a stand-by engine generator set when there is a power outage. An adjustable time will delay start of the standby generator. The UPS will be equipped with a static transfer switch, a manual maintenance switch, metering, and alarms and status lights.

Emergency/Stand-by Power Supply

A diesel engine-generator set is provided to provide all electrical station power and air-conditioning in the event of commercial power outage. The stand-by generator also is used to recharge the UPS system. The stand-by engine/generator set will start on a signal from the UPS after a set delay. Power is provided to the UPS via an automatic transfer switch. Explosion proof fixtures should be utilized.

Sensors and Monitors

For each engine driven pump and the sump pump a level-sensing probe located in the wet well will provide signals to start and stop. Pilot lights on a control panel will indicate the status of each pump. Any pump running will also actuate external signals to signify the station is operating. Sensors are attached at each of the discharge line flap gates to determine if the pump station is discharging. A sensor shall be provided in the discharge box to measure the head at the weir. The head shall be converted to discharge. Means will be provided to transmit the pump on/off and discharge status to a central location.

Probes will be provided for low water and high water conditions. Either of these conditions will activate local and remote indicators. These two alarms will only be capable of being reset by manual means at the pump station control panel.

Communications Link

A reliable means of communicating the status of the following conditions is to be provided:

- Pump station discharge
- High level in wet well
- Pump failure
- Pump status (on/off)
- Sump pump (overload, on/off)
- Engine failure
9.2.7 Collection System

Storm drains leading to the pumping station are usually designed on a flat grade to minimize depth and cost. A minimum grade that produces a velocity of 3 ft/sec in the pipe while flowing at a depth of one-quarter (1/4) full is suggested to avoid siltation problems in the collection system. Minimum cover or local head requirements will usually govern the depth of the uppermost inlets. Storm drainage systems tributary to the pump station can be quite extensive and costly. Linear or intermediate storage along the storm drain may be used to reduce peak flows and pipe sizes. For some pump stations, the storage available in the collection system may be significant. However, it is often necessary to provide additional storage near the pump station. This may be done by oversizing the collection system or designing an underground vault.

The collector lines should preferably terminate at a forebay or storage box. However, they may discharge directly into the station. Under the latter condition, the capacity of the collectors and the storage within them is critical to providing adequate cycling time for the pumps and must be carefully calculated. The inlet pipe should enter the station perpendicular to the line of pumps. It should be aligned with the centerline of the wet well and should have a straight run of at least 100 feet before entering the station. The inflow should distribute itself equally to all pumps. Baffles may be required to ensure that this is achieved.

Grate inlets and/or slotted drain inlets are recommended for lines that connect to pump stations. They will act as screens to prevent large objects from entering the system and possibly damaging the pumps. This approach has an additional advantage of simplifying debris removal since debris can be more easily removed from the roadway than the wet well.

9.2.7.1 Trash Racks and Grit Chambers

Trash racks should be provided at the entrance to the wet well if large debris is anticipated. For stormwater pumping stations, simple steel bar screens are adequate. Usually, the bar screens are inclined with bar spacing approximately 1.5 inches. Constructing the screens in modules facilitates removal for maintenance. An emergency overflow should be provided to protect against clogging and subsequent surcharging of the collection system.
9.2.8 Discharge System

The discharge piping should be kept as simple as possible. Pumping systems that lift the stormwater vertically and discharge it through individual lines to a gravity storm drain as quickly as possible are preferred. The pump location with respect to the outfall chamber should be set to provide as short a distance as possible. Individual pump discharge lines are the most cost-effective system for short outfall lengths. Individual lines may exit the pumping station either above or below grade. Damaging pump reversal could occur with very long force mains. Check valves should be installed. The effect of stormwater returning to the sump after pumping stops should be considered.

Gate valves should be provided in each pump discharge line to provide for continued operation during periods of repair, etc. A cost analysis should be performed to determine what length and type of discharge piping justifies a manifold. Number of valves required shall be kept to a minimum to reduce cost, maintenance and headloss through the system.

It may be necessary to pump to a higher elevation using long discharge lines. This may dictate that the individual lines be combined into a force main via a manifold. For such cases, check valves must be provided on the individual lines to keep stormwater from running back into the wet well and restarting the pumps or prolonging their operation time. Check valves should preferably be located on horizontal layouts rather than vertical, to prevent sedimentation on the downstream side after the valve closing.

The discharge line is typically either steel or cast iron and must be sized to be at least the size of the pump discharge diameter. The need for a larger pipe required to limit the maximum velocity to 10 ft/sec should be checked by the following equation.

\[ D = 1.12 \left( \frac{Q}{V} \right)^{0.5} \]  

9.2.9 Flap Gates and Valving

9.2.9.1 Flap Gates

The purpose of a flap gate is to restrict water from flowing back into the discharge pipe and to discourage entry into the outfall line. Flap gates are usually not watertight so the elevation of the discharge pipe should be set above the normal water levels in the receiving channel. If flap gates are used, it may not be necessary to provide for check valves.

9.2.9.2 Check Values

Check valves are watertight and are required to prevent backflow on force mains that contain sufficient water to restart the pumps. They also effectively stop backflow from reversing the direction of pump and motor rotation. They must be used on manifolds to prevent return flow from perpetuating pump operation. Check valves should be "non-slam" to prevent water hammer. Types include: swing, ball, dashpot and electric.
9.2.9.3 Gate Valves

A gate valve is used as a shut-off device on force mains to allow for pump or valve removal. These valves should not be used to throttle flow. They should be either totally open or totally closed.

9.2.9.4 Air/Vacuum Valves

Air/Vacuum valves are used to allow air to escape the discharge piping when pumping begins and to prevent vacuum damage to the discharge piping when pumping stops. They are especially important with large diameter pipe. If the pump discharge is open to the atmosphere, an air-vacuum release valve is not necessary. Combination air release valves are used at high points in force mains to evacuate trapped air.

9.2.9.5 ADOT Freeway Design Practice

The pump discharge line is connected to a discharge box. The box drains to the discharge outfall or storm drain. The discharge box is divided by a low wall that performs as a weir. The wall also creates a chamber that provides storage for pump recirculation operation. The chamber for recirculation is sized for testing and exercising one pump/engine set or the sump pump. The wall is beveled to form a sharp crested weir.

Two manually operated sluice gates are provided; one between the recirculation chamber and the outlet chamber and one at the recirculation pipe to prevent backflow during normal operations. A flap gate, usually of cast iron, is provided for each pump discharge line.

9.2.10 Safety

All elements of the pump station should be carefully reviewed for safety of operation and maintenance. Ladders, stairwells and other access points should facilitate use by maintenance personnel. Adequate space should be provided for the operation and maintenance of all equipment. Particular attention should be given to guarding moving components such as drive shafts and providing proper and reliable lighting. It may also be prudent to provide air-testing equipment in the station so maintenance personnel can be assured of clean air before entering.

Pump stations may be classified as a confined space in which case access requirements along with any safety equipment are all defined by code. Pump stations should be designed to be secure from entry by unauthorized personnel and as few windows as possible should be provided.

The designer should be aware of established safety procedures for working in pump stations. The designs shall meet the established safety criteria and enhance the ease of compliance

9.2.10.1 Internal Environment

Ventilation of dry wells is necessary to ensure a safe working environment for maintenance personnel. The ventilation system can be activated by a combined light/ventilation switch at the entrance to the station. Maintenance procedures normally require personnel to wait several
minutes after ventilation has started before entering the well. The testing of the air in the wet well prior to allowing entry may be required. If mechanical ventilation is required to prevent buildup of potentially explosive gasses, the pump motors or any spark producing equipment should be rated explosion proof or the fans run continuously.

Heating and dehumidifying requirements are variable. Their use is primarily dependent upon equipment and station type, environmental conditions and station use.

9.2.10.2 Hazardous Spills

The possibility of hazardous spills is always present under highway conditions. In particular, this has reference to gasoline, and the vulnerability of pump stations and pumping equipment to fire damage. There is a history of such incidents having occurred and also of spills of oils, corrosive chemicals, pesticides and the like having been flushed into stations, with undesirable results. The usual design practice has been to provide a closed conduit system leading directly from the highway to the pump station.

9.2.10.3 ADOT Freeway Design Practice

Combustible Gas Detection

Combustible gas detection systems shall be provided for the wet well and engine room. The systems shall include self-contained gas monitors and remote detectors. The systems shall be powered by the UPS. The systems shall be capable of responding to at least two levels of the presence of combustible gases. The lower level shall send signals to the communication system, actuate local signals, and activate the ventilation system. The high level sensor shall shut-off any engines that are running and activate the foam fire suppression system.

Fire Suppression System

The foam fire suppression system shall be capable of three 10-minute applications. The inflow pipe into the station should have provision to be shut off to prevent flammable material from draining into the station.

Ventilation System

The station exhaust fan ventilation system is composed of three separate areas. One each is provided for the wet well, engine room and stairway. The wet well system has a push/pull intake and exhaust system that shall provide for 12 air changes per hour. As noted above, this system is activated either manually or by gas detectors in the wet well.

The engine room has two exhaust fans that are thermostatically controlled. They are set to turn on when the room temperature exceeds 90 and 95 degrees Fahrenheit and the room temperature is higher than the outside ambient temperature. As noted above, this system is activated either manually or by gas detectors in the wet well or engine room.
The stairway has push/pull intake and exhaust systems that exhaust air from four levels. The stairwell light switch activates the stairwell ventilation system. The stairwell is equipped with fire doors at the top and bottom entrances. Each exhaust duct is equipped with a fire damper that closes automatically when smoke is detected in the wet well or engine room.

9.2.11 Building Architecture

The appearance of the pump station should be compatible with the surrounding area. The site layout should provide a secure, easily monitored structure. Applicable building codes shall be followed. The internal areas of the pump station are divided into the wet well, the engine/pump room, the control room, and an enclosed stairwell that provides access to the wet well. Items of particular concern regarding building architecture are the fire protection, security, and air-conditioning and ventilation systems.

9.2.11.1 Building Access

Separate access to the building will be provided for personnel into the engine/pump room, the control room, and the stairwell. In addition to the personnel access doors, access for service vehicles should be accommodated in the design considering adequate clearance height and turning radius into the engine/pump room. Double doors should be provided into the control room.

It will be necessary to remove motors and pumps from the station for periodic maintenance and repair. Removable roof hatches located over the equipment is a cost-effective way of providing this capability.

9.2.12 Site Design

Site location and layout is important to achieve a station that is secure and compatible with the surrounding area.

9.2.12.1 Access

The site should be accessed from a frontage road or a local street. It should not require access from a limited access highway.

9.2.12.2 Security

The area is to be well lighted. An eight-foot high wall shall surround the station with access only through a 24-foot locked gate. The access gate shall be situated so as to allow maximum visibility of as many accessible entrances as possible. If this is not possible, provide an open-weave fence section in the wall to allow observation of doors not visible from the access gate. All accessible exterior doors shall be equipped with proximity sensors that will activate local and remote alarms.
9.2.12.3 Fuel Storage

Generally the fuel storage tanks are located along the block wall. The location must be accessible for refueling and maintenance, but out of the way for other operations. Diesel fuel storage tanks are to have 24-hour capacity. Storage tanks may be in-ground tanks located in a vault or above ground steel-lined concrete. Vaults are to be provided with a roof deck to block solar radiation and water intrusion. The vault will provide for containment of 110 percent of the volume of the diesel tanks. The vault will not have a drain; any accumulation of liquid must be pumped out with a portable pump. Above ground concrete tanks must be fire rated and ballistic proof.

9.2.12.4 Utilities

Required utilities are electricity and water. Sanitary sewers are not required. The minimum electric service is 480 volt, 3-phase, four wire. The minimum size of water main is 6-inch diameter. A fire hydrant should be provided near the pump station and a minimum 4-inch service connection with backflow preventer should be provided to the pump station building.

9.2.12.5 Drainage

Adequate provisions for site drainage shall be provided. If possible, local drainage can be connected to the discharge box or outlet pipe.

9.3 DESIGN CONCEPT AND CRITERIA

The following discussion is made with the objective of minimizing the construction, operation and maintenance costs of highway stormwater pump stations while remaining consistent with the practical limitations of all aspects.

9.3.1 Discharge Head and System Curve

9.3.1.1 Headloss

Stormwater pumps are extremely sensitive to changes in head requiring the head demand on the pumps be calculated as accurately as possible. All valve and bend losses are to be considered in the computations. In selecting the size of discharge piping, consideration should be given to the manufactured pump outlet size vs. the head loss produced by smaller piping. This approach should identify a reasonable compromise in balancing cost. Once the head losses have been calculated for the range of discharges expected, the system curve (Q vs. TDH) can be plotted. This curve defines the energy required to pump any flow through the discharge system. It is especially critical for the analysis of a discharge system with a force main. When overlaid with pump performance curves (provided by manufacturer), it will yield the pump operating points (See Figure 9–5).
The combination of static head, velocity head and various head losses in the discharge system due to friction is called total dynamic head. These head losses are minimized by the selection of correctly sized discharge lines and other components.

**Losses in pipe system:**

<table>
<thead>
<tr>
<th>Loss Type</th>
<th>Coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elbow, 90 degree</td>
<td>0.20</td>
</tr>
<tr>
<td>Miscellaneous couplings and fittings</td>
<td>0.05</td>
</tr>
<tr>
<td>Flap Gate</td>
<td>0.25</td>
</tr>
<tr>
<td>Outlet</td>
<td>1.00</td>
</tr>
</tbody>
</table>

### 9.3.1.2 Total Dynamic Head

Pumps for a given station are selected to all operate together to deliver the Design $Q$ at a Total Dynamic Head computed to correspond with the design water level. Because pumps must operate over a range of water levels, the quantity delivered will vary significantly between the low level of the range and the high level.

A curve of total dynamic head versus pump capacity is available for each pump from the manufacturer. When running, the pump will respond to the total dynamic head prevailing and the quantity of discharge will be in accordance with the curve. The designer must study the pump performance curves for various pumps in order to develop an understanding of the pumping conditions (head, discharge, efficiency, kilowatt, etc.) throughout the full range of head that the
pump will operate under. The system specified must operate properly under the full range of specified head.

When the pump is raising the water from the lowest level, the static head will be greatest and the discharge will be the least. When operating at the highest level, the static head will be the least and the discharge will be the greatest. Pump capabilities must always be expressed in both quantity of discharge and the total dynamic head at a given level. Typically, these conditions are specified for three points on the performance curve. One point will be near maximum TDH, the next will be the design point, and the third will be at about minimum TDH.

A pump is selected to operate with the best possible efficiency at its Design Point, corresponding to the design water level of the station, and its performance is expressed as the required discharge at the resulting total dynamic head. The efficiency of a stormwater pump at its design point may be 75% or 80% or more, but this will depend on the type of pump. When the static lift is greatest (low water in sump), the energy required (horsepower) may be the greatest even though the quantity of water raised is less. This is because the pump efficiency may also be much less. The pump selection should be made so that maximum efficiency is at the design point.

The total dynamic head (TDH) should be determined for a sufficient number of points to draw the system head curve that will be discussed later in this chapter. The TDH is computed as follows:

\[
TDH = H_s + H_f + H_v + H_l
\]

where:
- \(TDH\) = total dynamic head (ft),
- \(H_s\) = max. static head (at lowest pump-off elevation) (ft),
- \(H_f\) = friction head (ft) (i.e., friction loss),
- \(H_v\) = velocity head (ft) \((V^2/2g)\), and
- \(H_l\) = losses through fittings, valves, etc., (ft).

Adjustments may have to be made to these curves to account for losses within the pumping unit provided by the manufacturer.

### 9.3.2 Pumps

#### 9.3.2.1 Number and Capacity

The number of pumps needed are determined by following a systematic process defined in Section 9.4. However, two to three primary pumps plus one submersible pump have been judged to be the recommended minimum. If the total discharge to be pumped is small and the area draining to the station has little chance of increasing substantially, the use of two pumps in one station is preferred. Consideration may be given to over sizing the pumps to compensate, in part, for a pump failure. A two-pump system could have pumps designed to pump 66% - 100% of the required discharge and the three pump system could be designed so that each pump will pump 50% of the design flow. The possible impacts caused by the loss of one pump can be used as a basis for deciding the size and numbers of the pumps.
Economic limitations on power unit size as well as practical limitations governing operation and maintenance should be used to determine the upper limit of pump size. The minimum number of pumps used may increase due to these limitations.

Equal-size pumps are to be used. Identical size and type enables all pumps to be freely alternated into service. This equalizes wear and reduces needed cycling storage. It also simplifies scheduling maintenance and allows pump parts to be interchangeable. Hour meters and start meters are to be provided, these aid in scheduling needed maintenance.

9.3.2.2 Final Selection

For the typical highway application, any of the three pump types described earlier will usually suffice. However, ADOT typically prefers mixed flow pumps. If not, manufacturers' information will likely dictate the type required. However, knowing the operating RPMs, a computation for specific speed can be made to check the appropriateness of the pump type (see Figure 9–6 to determine the ranges where specific impeller types should be used). Suction specific speed may be defined as that speed in revolutions per minute at which a given impeller would operate if reduced proportionally in size so as to deliver a capacity of 3.3 gpm against TDH of 3-feet. This is an index number descriptive of the suction characteristics of a given pump. Higher numerical values are associated with better NPSH capabilities. This number should be checked for dry pit applications and systems with suction lifts. Once the pump type and capacity have been determined, the final selection of the pump can be made.
9.3.3 Wet Well Design

The primary criteria for sizing the wet well involves the number of pumps and pump bell diameter. This criteria includes floor clearance, minimum distance between an inlet bell and the wall, the minimum clearance between adjacent inlet bells, width of partition wall between pumps, and the submergence required for the pump bell diameter. The specific criteria for both circular and rectangular wet well can be found in American National Standard for Pump Station Intake Design (HI 9.8, 2012) by the Hydraulic Institute as well as from pump manufacturers.

The wet well size and shape are important factors for both their contribution to available storage and for providing room for proper sizing and layout of pumps. However, the final number of pumps is not normally known until the final design phase. Therefore, it is necessary to estimate...
wet well dimensions based on a trial number and size of pumps. It may be necessary to increase dimensions to provide additional storage or to accommodate additional pumps.

9.3.3.1 Cycling Sequence and Volumes

Cycling is the starting and stopping of pumps, the frequency of which must be limited to prevent damage and possible malfunction. Sufficient volume must be provided either in the wet well or outside the wet well for safe cycling. The volume required to satisfy the minimum cycle time is dependent upon the characteristics of the power unit, the number and capacity of pumps, the sequential order in which the pumps operate and whether or not the pumps are alternated during operation. However, to keep sediment in suspension, the wet well volume should not be oversized.

Before discussing pump cycling considerations, operation of a pump station will be described. Initially, the water level in the storage basin will rise at a rate depending on the rate of the inflow and physical geometry of the storage basin. When the water level reaches the stage designated as the first pump start elevation, the pump will be activated and discharge water from storage at its designated pumping rate. For pumps that are engine driven, there will be a delay between the initial start signal and the commencement of pumping. This delay will be on the order of 3 minutes for engine warmup.

The volume of inflow that will occur during this delay must be taken into consideration in the design of the storage volume. If the pumping rate exceeds the rate of inflow, the water level will drop until it reaches the first pump stop elevation. With the pump stopped, the basin begins to refill and the cycle is repeated. This scenario illustrates that the cycling time will be lengthened by increasing the amount of storage between pump on and off elevations. This volume of storage between first pump on and off elevations is termed usable volume. In theory, the minimum cycle time allowable to reduce wear on the pumps will occur when the inflow to the usable storage volume is one-half the pump capacity.

There are two basic cycling sequences. One is referred to as the "common off elevation." In this sequence, the pumps start at successively higher elevations as required; however, they all stop at the same off elevation. This is advantageous when large amounts of sediment are anticipated. The other sequence (ADOT Preference) uses a "successive start/stop" arrangement in which the start elevation for one pump is also the stop elevation for the subsequent pump, i.e., the start elevation for pump 1 is the stop elevation for pump 2, the start elevation for pump 2 is the stop elevation for pump 3, etc. (see Figure 9–7). There are countless variations between these two sequences.

There are also different pump start alternation techniques that reduce the cycling volume requirement and equalize wear on the pumps. They range from simply alternating the first pump to start, to continuously alternating all pumps during operation, a technique referred to as "cyclical running alternation". Using this technique, each pump is stopped in the same order in which it starts, i.e., the first pump to start will be the first pump to stop, etc. (see Figure 9–8).

Alternating the first pump to start is sufficient for stormwater pump stations where more than one pump on will be rare and of short duration. This alternation technique coupled with the
successive start/stop cycling sequence requires the smallest total cycling volume possible (see Figure 9–9). This total volume is computed as follows:

\[ V_t = \frac{Q_p t_c}{4N} \]  \hspace{1cm} 9.3

where:
- \( V_t \) = total cycling storage volume (ft³),
- \( Q_p \) = total capacity of all pumps (ft³/sec),
- \( t_c \) = minimum allowable cycle time (sec) (= 3600/max. starts per hour), and
- \( N \) = total number of equal-size pumps.

The proof is as follows:

\[ t = \text{Time between starts}, \]
\[ t = \text{Time to Empty + Time to Fill usable storage volume } V_t, \]
\[ t = \frac{V_t}{Q_p} + \frac{V_t}{Q_i} \quad \text{When } Q_i = \frac{Q_p}{2}, \text{ (ft³/sec)} \]

\[ t = 4\frac{V_t}{Q_p}, \text{ s} \]  \hspace{1cm} 9.4

Or, for \( t \) in minutes \( t = 4V_t/60Q_p = V_t/15Q_p \)

**Pump Alternation Sequence Effect**

Let us take an example when four pumps are installed in the same station with pumps starting in sequence and stopping in reverse order.

- By designing the control system for pump alternation, sump volumes will be reduced as well as distribute the pump operating time more evenly between the four pumps.
- If the inflow is less than the capacity of one pump, pump number one would, without alternation, do all the work.
- With alternation pump number one starts and draws down. Next start would call pump number two, etc.
- This means that with four pumps of the same size and operating in an alternating sequence each pump is called on to pump down sump volume \( V_1 \) every fourth time. The cycle time of each pump will be four times longer than the cycle time of filling and emptying of \( V_1 \).
- The volume required for each pump will vary, depending upon the characteristics of the discharge system. It should be noted that with these volumes, the minimum allowable cycle time will only be experienced when the proportionate inflow to each pump is exactly one-half the capacity of that pump. All other inflows will produce a cycle time longer than the minimum.
- This system works for any number of pumps in a station.
If \( Q_{in} \) is less than the half the capacity of one pump, the pumps will operate \( 1/8 \) of their cycle time.

If \( Q_{in} \) is greater than the capacity of two pumps but less than the capacity of three pumps, the pumps will operate \( 5/8 \) of their cycle time.

If \( Q_{in} \) is greater than the capacity of three pumps but less than the capacity of four pumps, the pumps will operate \( 7/8 \) of their cycle time.

Figure 9–7 Pump Sequences

Figure 9–8 Schematic of Pump Sequences at Different Inflow Rates
Pumps With Cyclical Running Alternation (A Variant of Operation Sequence I in Figure 9–7)
100% corresponds to the volume derived from the formula $V_t = (Q_p T_{\text{min}})/4N$

**Figure 9–9 Comparison of The Pump Volume with/without Cyclical Running Alternation**

**Lowest Pump "Off" Elevation**

It is recommended that the lowest pump "off" elevation be located at or within 1-foot below the inlet invert elevation unless plan dimension constraints dictate that the station floor be lowered to obtain the necessary cycling volume. This recommendation is based on the fact that it is usually less expensive to expand a station’s plan dimensions than to increase its depth. This elevation represents the static pumping head to be used for pumping selection.

**Pump "On and Off" Elevations**

These should be set at the elevations, which satisfy the individual pump cycling volumes ($V_x$). Starting the pumps as soon as possible by incrementing these volumes successively above the lowest pump off elevation will maximize what storage is available within the wet well and the collection system. The depth required for each volume is computed as follows:
\[ H_x = \frac{V_x}{\text{plan area}} \]

**Pumping Range**

It is recommended that the minimum distance between the “on” and “off” elevation of an individual pump be 6-inches. If it is less than 6-inches, then reduce the plan area. Though rare, this could require a reduction in the number of pumps, an increase in station depth, or both.

**Allowable High Water Elevation**

The allowable high water (AHW) elevation in the station should be set such that the water surface elevation at the lowest inlet in the collection system provides 0.5 feet of freeboard below the roadway grate.

**Clearances**

Pump to pump, pump to back wall, and pump to sidewall clearances should be the minimum possible to minimize the potential for sedimentation problems. Consult manufacturer literature or a dimensioning guide. The pump inlet to floor clearance plus the pump submergence requirement constitutes the distance from the lowest pump "off" elevation to the wet well floor. The final elevation may have to be adjusted as the type of pump to be installed is finalized.

**Intake System Design**

The primary function of the intake structure is to supply an even distribution of flow to the pumps. An uneven distribution may cause strong local currents resulting in reduced pump efficiency and undesirable operational characteristics. The ideal approach is a straight channel coming directly into the pump or suction pipe. Turns and obstructions are detrimental, since they may cause eddy currents and tend to initiate deep-cored vortices. The inflow should be perpendicular to a line of pumps and water should not flow past one pump to get to another. Unusual circumstances will require a unique design of the intake structure to provide proper flow to the pumps.

**ADOT Freeway Design Practice**

Pump on/off elevations for the primary pumps are determined during final design of the station considering the pump characteristic minimum cycle time, and total storage available (both underground plus wet well). The recommended minimum pump cycle time is 20-minutes.

The sump pump is set to turn on two feet below the level of the intake pipe invert and turn off at the wet well floor elevation. The first main pump should turn on at a slightly higher level than the sump pump and both pumps will be on for a portion of the total range. When the wet well level backs into the intake/storage pipes, the volume of storage in the sloping pipe is calculated as described in HEC 24 (HEC 24, 2001) for an “ungula”.
### 9.3.4 Inflow-Outflow Routing

#### 9.3.4.1 Storage

The development of the wet-well design as discussed in Section 9.3.3 has general application when it is anticipated that most of the peak flow will be pumped. In that case, pump run time and cycling sequences are of great importance. In the case of many of the highway storm drain situations, it has been the practice to store substantial parts of the flow in order to minimize pumping requirements as well as outflow piping. The demand on the pumping system is different and thus additional considerations need to be made.

The total storage capacity that can or should be provided is an important initial consideration in pump station design. The designer should recognize that a balance should be reached between pump rate and storage volume. This will require a trial and error procedure used in conjunction with an economic analysis. Using the hydrograph and pump-system curves, various levels of pump capacity can be tried and the corresponding required total storage can be determined. The basic principle is that the volume of water as represented by the shaded area of the hydrograph in Figure 9–10 is beyond the capacity of the pumps and must be stored. If a larger part or most of the design storm is allowed to collect in a storage facility, a much smaller pump station can be utilized, with anticipated cost benefits. The principles discussed for minimum run time, pump cycling, etc. in the design of wet wells should also be considered in the case of larger storage volume development. However, it will be noted that differences exist as the volume of storage becomes larger. Typically, the concern for meeting minimum run times and cycling time will be reduced because the volume of storage is sufficient to prevent these conditions from controlling the pump operation. The start and stop elevations will be of different magnitudes because of the volume represented by each increment of storage depth.

![Figure 9–10](image.png)
9.3.4.2 Mass Curve Diagram

The approach used for the design of the pump station is that associated with the development of an inflow mass curve, Figure 9–11. The mass inflow curve procedure is commonly used when significant storage is provided outside of the wet well. The plotting of the performance curve on the mass inflow diagram gives the designer a good graphical tool for determining storage requirements. The procedure also makes it easy to visualize pump start/stop and run times. In the event that a pump failure should occur, the designer can also evaluate the storage requirement and thus the flooding or inundation that could occur. The graphical procedure described below is approximate and labor intensive. Computer solutions of the method should be used for the analysis of pumping systems.

In this process, the designer uses an inflow hydrograph and a developed stage-storage relationship. Trial pumping systems will be applied to the inflow mass curve to develop a mass curve routing diagram. The inflow hydrograph is a fixed design component while the storage and pumping discharge rates are variable. The designer will select a design pumping discharge rate, this may be based on downstream capacity considerations. With the inflow mass curve and an assigned pumping rate, the required storage volume can be determined by various trials of the routing procedure.

As the stormwater flows into the storage basin, it will accumulate until the first pump start elevation is reached. The first pump is activated and if the inflow rate is greater than the pump rate, the stormwater will continue to accumulate until the second pump start elevation is reached. As the inflow rate decreases, the pumps will shut off at their respective pump stop elevations. These conditions are modeled in the mass curve diagram by establishing the point at which the cumulative flow curve has reached the storage volume associated with the first pump-start elevation. This storage volume is represented by the vertical distance between the cumulative flow curve and the base line. A vertical storage line is drawn at this point since it establishes the time at which the pump first starts. The pump discharge line is drawn from the intersection of the vertical storage line and the base line upwards toward the right; the slope of this line is equal to the discharge rate of the pump. The pump discharge curve represents the cumulative discharge from the storage basin, while the vertical distance between the inflow mass curve and the pump discharge curve represents the amount of stormwater stored in the basin.

If the rate of inflow is greater than the pump capacity, the inflow mass curve and the pump discharge curve will continue to diverge until the volume of water in storage is equal to the storage associated with the second pump-start elevation. At this point the second pump starts, and the slope of the pump discharge line is increased to equal the combined pumping rates. The procedure continues until peak storage conditions are reached. At some point on the inflow mass curve, the inflow rate will decrease, and the slope of the inflow mass curve will flatten. To determine the maximum amount of storage required, a line is drawn parallel to the pump discharge curve and tangent to the inflow mass curve. The vertical distance between the lines represents the maximum amount of storage required.
The routing procedure continues until the pump discharge curve intersects the inflow mass curve. At this point the storage basin has been completely emptied, and a pumping cycle has been completed. As the storm recedes, the pumps will cycle to discharge the remaining runoff.

![Mass Curve Diagram](image)

**Figure 9–11 Mass Curve Diagram**

In developing the pump discharge curve, the designer should remember that the pump's performance curve is quite sensitive to changes in head and that the static head will fluctuate as the water level in the storage basin fluctuates. The designer should also recognize that the pump discharge rate represents an average pumping rate.

### 9.4 DESIGN PROCEDURES

The following is a systematic procedure that integrates the hydraulic design variables involved in sump design. It incorporates the above recommended design criteria and yields the required number and capacity of pumps as well as the wet well and storage dimensions. The final dimensions can be adjusted as required to accommodate non-hydraulic considerations such as maintenance.

Theoretically an infinite number of designs are possible for a given site. Therefore, to initiate design, constraints must be evaluated and a trial design formulated to meet these constraints. Then by routing the inflow hydrograph through the trial pump station its adequacy can be evaluated.

**Design Procedure:**

1. Develop Inflow Hydrograph
2. Estimate Pumping Rate, Volume of Storage, and Number of Pumps
3. Determine High Water Level
4. Determine Pump Pit dimensions,
5. Pump Cycling and Usable Storage
6. Estimate Volume of Storage
7. Stage-Storage Relationship
8. Determine Total Dynamic Head, Net Positive Head, and Head Capacity Curves
9. Pump Design Point
10. Power Requirements
11. Mass Curve Routing
12. Documentation

Design Checklist

Initial Data

- Contributing Drainage Area
- Locating of Outfall
- Capacity of Outfall
- Hydrology
- Environmental Considerations

Possible Components

Building Architectural

- Location
- Capacity
- Storage
- Pit Type
- Equipment Access
- Hoisting Equipment
- Safety
- Ventilation Equipment
- Potable Water Supply
Hazardous Material Containment

Possible Components

Mechanical System
  Pump Type
  Power Source
  Monitor & Control Systems
  Sediment Handling

Collection System
  Inlets
  Trash Rack
  Grit Chamber

Discharge System
  Piping
  Valving

Hydraulic Analysis
  Pump Characteristics
  Pipe Losses
  Miscellaneous Losses
  Mass Curve Routing

9.4.2 Pump Station Design

The procedure for pump station design is illustrated in the following 11 steps.

Step 1  Inflow to Pump Station

  Develop inflow hydrograph to the pump station using the procedures presented in DDM Volume 2 - Hydrology. For highway pump stations where the inflow is of short duration and high intensity, a hydrograph that correctly depicts the time/area/discharge relationship should be used, even for small drainage areas. For small basins, HEC-HMS can be used to develop the hydrograph in a two stage process. See Appendix 8B.

Step 2  Estimate Maximum Discharge, Pumping Rate and Number of Pumps
Because of the complex relationship between the variables of pumping rates, storage and pump on-off settings, a trial and error approach is usually necessary for estimating the pumping rates and storage required for a balanced design. A wide range of combinations will produce an adequate design. The goal is to develop an economic balance between volume and pumping capacity.

Determine the required type, size, and capacity of pumps and power unit characteristics. Usually the minimum number is 3, however this may be 2 for small pump stations or more than 3 if each pump is limited by power unit size. Develop the system curve for the proposed discharge system and superimpose it on the pump performance curves.

Determine the maximum number of starts/hour (minimum cycle time) for the type of power unit proposed.

Step 3  Allowable High Water Level

The highest permissible water level must be set as 0.5 to 2-feet below the finished pavement surface at the lowest pavement inlet.

At the design inflow, some head loss will occur through the pipes and appurtenances leading to the pump station. Therefore a hydraulic gradient will be established and the maximum permissible water elevation at the station will be the elevation of the hydraulic gradient. This gradient will be very flat for most wet well designs with exterior storage because of the unrestricted flow into the wet well. Determine the inflow invert elevation

Step 4  Determine Pump Pit Dimensions

Determine the minimum required plan dimensions (LxW) for the pump station from manufacturer's literature or from dimensioning guides such as those provided by the Hydraulic Institute, see Figure 9–12 and Figure 9–13. The dimensions are usually determined by locating the selected number of pumps on a floor plan keeping in mind the guidance given in Section 9.3.3 for clearances and intake system design. Keep in mind the need for clearances around electrical panels and other associated equipment that will be housed in the pump station building.
J = Minimum distance from trash rack to backwall (length of Pump Pit).

VSF= Maximum Velocity of stream flow. (0.5 ft./sec. recommended)

B = Maximum distance from pump centerline to backwall.

C = Average distance from underside of bell to bottom of pit

H = Minimum distance from minimum water level to bottom of pit.

M = Minimum center-to-center of pumps.

Figure 9–12  Recommended Rectangular Pump Pit Dimensions
NOTE: 10° or less preferred with 1 ft/sec velocity max. at screen location shown. 15° max with velocity reduced to 0.5 ft/sec.

Figure 9–13  Sump Dimension, Wet Pit Type Pumps
Step 5  Pump Cycling and Usable Storage

One of the basic parameters addressed initially was that the proper number of pumps must be selected to deliver the design Q. Also, the correct elevations must be chosen to turn each pump on and off. Otherwise, rapid cycling (frequent starting and stopping of pumps) may occur causing undue wear and possible damage to the pumps. The volume of storage between first pump on and off elevations is termed usable volume. For a given pump with a capacity \( Q_p \), cycling will be a maximum (least time between starts) when the inflow \( Q \) to the usable storage is one-half the pump capacity. Assuming this condition, usable volume can be related to cycling time.

Generally, the minimum allowable cycling time, \( t \), for electrically driven pumps is designated by the pump manufacturer based on electric motor size. In general, the larger the motor, the larger is the starting current required, the larger the damaging heating effect and the greater the cycling time required. The pump manufacturer should always be consulted for allowable cycling time during the final design phase of project development.

However, the following limits may be used for estimating allowable cycle time during preliminary design:

<table>
<thead>
<tr>
<th>Motor kW</th>
<th>Cycling Time (t), min</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 11</td>
<td>5.0</td>
</tr>
<tr>
<td>15 - 22</td>
<td>6.5</td>
</tr>
<tr>
<td>26 - 45</td>
<td>8.0</td>
</tr>
<tr>
<td>48 - 75</td>
<td>10.0</td>
</tr>
<tr>
<td>112 - 149</td>
<td>13.0</td>
</tr>
</tbody>
</table>

Knowing the pumping rate and minimum cycling time, the minimum necessary allowable storage, \( V \), to achieve this time can be calculated by:

\[
V = 15 Q_p t
\]

where: \( Q_p = \) pump capacity (cfs), and \( t = \) time between starts (min).

When larger volumes of storage are available, the initial pump start elevations can be selected from the stage-storage curve. Since the first pump turned on should typically have the ability to empty the storage facility, its turn off elevation would be the bottom of the storage basin. The elevation associated with the minimum allowable storage volume in the stage-storage curve is the lowest turn-on elevation that should be allowed for the starting point of the first pump. The minimum allowable storage is calculated by the equation \( V = 15 Q_p t \).
Having selected the trial wet-pit dimensions, the pumping range, $\Delta h$, can then be calculated. The pumping range represents the vertical height between pump start and pump stop elevations. Usually, the first pump stop elevation is controlled by the minimum recommended bell submergence criteria specified by the pump manufacturer or the minimum water level, $H$, specified in the design. The first pump start elevation will be a distance, $\Delta h$, above $H$. When the only storage provided is in the wet pit, the pumping range can be calculated by dividing the required storage volume by the wet pit area.

$$\Delta h = \frac{V}{\text{wet pit area}}$$

The minimum plan area of the wet pit based on the pump layout is checked by comparison of the required pumping range with the available pumping range. Determine the design high water elevations (DHW) by adding the storage depth, $\Delta h$, to inlet invert elevation. $\text{DHW} = \text{INV} + \Delta h$. If $\text{DHW}$ is less than or equal to the allowable high water elevation (AHW), the dimensions are satisfactory. If the $\text{DHW}$ is greater than the AHW, compute

$$X = \frac{V_t}{(L(\text{AHW} - \text{INV}))} - W$$

where:
- $L =$ length of wet well, dimension perpendicular to the line of pumps
- $W =$ width of wet well

If $X$ is less than the width required between pumps compute the new wet well length as

$$L = \frac{V_t}{(W(\text{AHW} - \text{DNV}))}$$

The wet well width should not be increased.

If $X$ is greater than the width required between pumps, add one pump and return to step 2 (i.e., determine the new pump/motor characteristics, wet well dimensions, and cycle time storage volumes).

The second and subsequent pump start elevations will be determined by plotting the pump performance on the mass inflow curve. This distance between pump starts may be in the range of 1 to 3 feet for stations with a small amount of storage and 0.25 to 0.5 feet for larger storage situations.

**Step 6** Estimate Volume of Storage

Some approximation of storage in addition to the pumping rate is necessary to produce the first trial design. Using the approach presented in Section 9.3.4.1 the peak pumping rate is assigned and a horizontal line representing the peak rate is drawn across the top of the hydrograph. The shaded area above the peak pumping rate represents an estimated volume of storage required above the last pump turn.
on point. This area is measured to give an estimated starting size for the storage facility. Once an estimated storage volume is determined, a storage facility can be estimated. The shape, size, depth, etc., can be established to match the site and a stage-storage relationship can be developed.

**Step 7  Stage-Storage Relationship**

Routing procedures require that a stage-storage relationship be developed. This is accomplished by calculating the available volume of water for storage at uniform vertical intervals.

Having roughly estimated the volume of storage required and trial pumping rate by the approximate methods described in the preceding sections, the configuration and elevations of the storage chamber can be initially set. Knowing this geometry, the volume of water stored can be calculated for its respective depth. In addition to the wet-pit, storage will also be provided by the inflow pipes and exterior storage if the elevation of water in the wet-pit is above the inflow invert. If the storage pipe is circular, the volume can be calculated using the ungula of a cone formula as discussed in Chapter 6 of HEC 24 (HEC 24, 2001). A similar procedure would be followed for other storage configurations. Volume in a storage chamber can be calculated below various elevations by formulas depending on the shape of the chamber. A storage vs. elevation curve can then be plotted and storage below any elevation can readily be obtained.
Step 8  Total Dynamic Head

Total Dynamic Head is the sum of the static head, velocity head and various head losses in the pump discharge system due to friction. Knowing the range of water levels in the storage pit and having a trial pump pit design with discharge pipe lengths and diameters and appurtenances such as elbows and valves designated, total dynamic head for the discharge system can be calculated.

The designer must select a specific pump in order to establish the size of the discharge piping that will be needed. This is done by using information either previously developed or established. Though the designer will not typically specify the manufacturer or a specific pump, study of various manufacturers’ literature will assist in establishing reasonable relationships between total dynamic head, discharge, efficiency and energy requirements. This study will also give the designer a good indication of discharge piping needed since pumps that produce the desired results will have a specific discharge pipe size.
To summarize the Total Dynamic Head (TDH) is equal to:

$$TDH = H_s + H_f + H_v + H_p$$

where:
- $H_s$ = static head or height through which the water must be raised (ft),
- $H_f$ = loss due to friction in the pipe (ft),
- $H_v$ = velocity head (ft), and
- $H_p$ = loss due to friction in water passing through the pump valves, fittings and other items (ft).

The Manning’s formula expressed as follows is generally used for discharge lines.

$$H_f = LS_f = L[(Q n)/(1.486 A R^{2/3})]^2$$

where:
- $Q$ = discharge (ft$^3$/sec),
- $L$ = length of pipe (ft),
- $n$ = Manning’s roughness value,
- $A$ = cross-sectional area of discharge pipe (ft$^2$), and
- $R$ = hydraulic radius of discharge pipe, (ft) (for pipe running full, $R = \text{diameter}/4$).

Friction losses can also be computed by the Darcy Formula. This requires computation of the relative roughness of the pipe, the Reynold’s number and the friction factor. The Hydraulic Institute and others have produced line loss tables and charts that make determination of losses quite easy and accurate. The tables and charts have been developed for a variety of pipe materials and are recommended for use in determining line and fitting losses for the discharge side of the pumping system.

Head losses for other components should be evaluated using the information in Chapter 7 Storm Drainage Systems. Standard textbooks and manufacturers’ catalogs may also be consulted.

**Step 9 Pump Design Point**

Using methods described in the previous step, the Total Dynamic Head of the outlet system is calculated for a specific static head and various discharges. These TDHs are then plotted vs. discharge. This plot is called a system head curve. A system head curve is a graphical representation of total dynamic head plotted against discharge $Q$ for the entire pumping and discharge system. The required design point of a pump can be established after the pump curve is superimposed to give a visual representation of both system and pump. As usually drawn, the system head curve starts from a low point on the $Y$-ordinate representing the static head at zero discharge. It then rises to the right as the discharge and the friction losses increase. A design point can be selected on the system head curve and a pump can be selected.
to match that point. The usual pump curve is the reverse of the system head curve so the point of intersection is clearly identifiable. System head curves are often drawn for several different static heads, representing low, design and maximum water levels in the sump.

One, two or more pump curves can be plotted over the system head curves and conditions examined. If a change of discharge line size is contemplated, a new system head curve for the changed size (and changed head loss) is easily constructed. Figure 9–15 shows the difference in the pump operating range when two pumps are connected to a common discharge line versus separate discharge lines. With a common discharge line the design point will move from A, for the first pump operating alone, to B, with both pumps operating at the minimum static head. Whereas, the same two pumps with separate discharge lines operate over a more limited range since the additional loss from a shared pipe is not experienced. In highway design, it is common practice to provide individual discharge lines for each pump. Therefore, the additional loss from a shared pipe is not experienced. It should be noted that the pump will always operate at the intersection of the system curve and the pump curve.

Each pump considered will have a unique performance curve that has been developed by the manufacturer. More precisely, a family of curves is shown for each pump, because any pump can be fitted with various size impellers. These performance curves are the basis for the pump curve plotted in the system head curves discussed above. The designer must study pump performance curves. The designer must have specific information on the pumps available in order to be able to specify pumps needed for the pump station.

Any point on an individual performance curve identifies the performance of a pump for a specific Total Dynamic Head (TDH) that exists in the system. It also identifies the power required and the efficiency of operation of the pump. It can be seen that for either an increase or decrease in TDH, the efficiency is reduced as the performance moves away from the mid-point of the performance curve. It should also be noted that as the TDH increases, the power requirement also increases. The designer must make certain that the motor specified is adequate over the full range of TDHs that will exist. It is desirable that the design point be as close to the mid-point as possible, or else to the left of the mid-point rather than to the right of or above it. The range of the pump performance should not extend into the areas where substantially reduced efficiencies exist.

It is necessary that the designer correlate the design point discussed above with an elevation at about the mid-point of the pumping range. By doing this, the pump will work both above and below the TDH for the design point and will thus operate in the best efficiency range.
Step 10  Power Requirements

To select the proper size of pump motor, compute the energy required to raise the water from its lowest level in the pump pit to its point of discharge. This is best described by analyzing pump efficiency. Pump efficiency is defined as the ratio of pump energy output to the energy input applied to the pump. The energy input to the pump is the same as the driver's output and is called brake kilowatts.

\[
e = \frac{Q \gamma H}{737 \text{ (brake kW)}}
\]

where:
- \(e\) = efficiency = pump output/brake kilowatts,
- \(Q\) = pump capacity (ft\(^3\)/sec),
- \(\gamma\) = specific weight of liquid (62.4 lbs/ft\(^3\) for cold water), and
- \(H\) = head (ft).

Efficiency can be broken down into partial efficiencies — hydraulic, mechanical, etc. The efficiency as described above, however, is a gross efficiency used for the comparison of centrifugal pumps. The designer should study pump performance curves from several manufacturers to determine appropriate efficiency ranges. A
minimum acceptable efficiency should be specified by the designer for each performance point specified.

To compute the energy required to drive a pump, assume that the pump will operate at 80% efficiency. The above equation can then be solved for brake kilowatts.

**Step 11 Mass Curve Routing**

The procedures described thus far will provide all the necessary dimensions, cycle times, appurtenances, etc. to design the pump station. A flood event can be simulated by routing the design inflow hydrograph through the pump station by methods described in Section 9.3.4.2. In this way, the performance of the pump station can be observed at each hydrograph time increment and pump station design evaluated. Then, if necessary, the design can be "fine-tuned."

### 9.5 REFERENCES


APPENDIX 9A
PUMP STATION EXAMPLE
Example No. 9-1  Pump Station Storage Requirements

Problem:
Determine the required storage to reduce the peak flow of 22 cfs to 14 cfs.

Given:
Hydrograph reduction as shown in Figure 9–16. Using the assumed storage pipe shown in Figure 9–17, the stage-storage curve in Figure 9–18, the stage discharge curve in Figure 9–19 and the inflow hydrograph in Figure 9–16, the storage can be determined.

The inflow mass curve is developed in Figure 9–20. Since 14 cfs was to be pumped, it was assumed that two 7 cfs pumps would be used. The pumping conditions are as follows:

<table>
<thead>
<tr>
<th>Pump No. 1 (7 cfs-3500 gpm)</th>
<th>Pump No. 2 (7 cfs-3500 gpm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elev.</td>
<td>Volume</td>
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<tr>
<td>2.0</td>
<td>2000</td>
</tr>
<tr>
<td>3.0</td>
<td>4000</td>
</tr>
</tbody>
</table>

The storage volumes (ft³) are associated with the respective elevations.

Figure 9–16  Example Estimated Required Storage
520’ - 48” dia. PIPE @ 0.004 ‘/’
16’X 22’ Wet Well

HIGH WATER ALARM
6.56 Ft.

Figure 9–17 Storage Pipe Sketch

Figure 9–18 Stage-Storage Curve
Table 9-2 Rainfall Data

<table>
<thead>
<tr>
<th>Elevation (Ft)</th>
<th>Pipe (Ft³)</th>
<th>Wet Well (Ft³)</th>
<th>Total (Ft³)</th>
</tr>
</thead>
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<tr>
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<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>0.5</td>
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</tr>
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<td>2464</td>
<td>8998</td>
</tr>
</tbody>
</table>

Figure 9–19 Pump Stage-Discharge Curve
Solution:

To visualize what is happening during the design period, the pump discharge is superimposed on the inflow mass curve as shown in Figure 9–21. Note that the first pump is turned on at about hour 11.4 when a storage volume of 2000 ft³ has accumulated. At about hour 11.5 pump number one has emptied the storage basin and the pump turns off. At about hour 11.7 the storage volume has again reached 2000 ft³ and a pump is turned on. If an alternating start plan had been developed, this would be the second pump that would turn on at this point. If an alternating start plan had not been designed the first pump would again be started. At about hour 11.8 the volume in storage has increased to 4220 ft³, which is associated with a turn on elevation of 3.0 feet. Both pumps operate until about hour 12.4 when the volume in the storage basin has been essentially pumped out. The pumps will continue to start and stop until the hydrograph has receded and the inflow stops.
The shaded area between the curves (See Figure 9–22) represents stormwater going into storage. Pump cycling at the end of the storm has been omitted in order to simplify the illustration. When the stored volume remaining is equal to the volume (600 ft$^3$) associated with the Pump No. 2 stop elevation (1.0-foot), pump number 2 shuts off. Pump No. 1 shuts off when the storage pipe is emptied at Pump No. 1 stop elevation (0.0).

The trial design is now complete. The peak inflow rate of 21 cfs has been reduced to a peak outflow rate of 14 cfs through use of a maximum storage of 8400 ft$^3$. A reduction of 33 % has been achieved.
It should be noted that the number of starts per hour can be determined by looking at the plots on the inflow mass curve. Once the mass inflow curve has been developed, it is a relatively easy process to try different pumping rates and different starting elevations until a satisfactory design is developed. This is only one possible design option. Other pumping rates are plotted on the inflow mass curve to determine their performance. Other combinations can be considered. The pumping rate can be reduced by providing more storage. The storage may be reduced by reducing starting elevations.

Figure 9–22 Pump Operation Hydrographs