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Flood Management Design Manual

Sonoma County Water Agency
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Sonoma County Water Agency

Flood Management Design Manual

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Acronyms and Abbreviations

A

ac	acre
ARC	Antecedent Runoff Condition
ARF	Areal Reduction Factor

B

Bay Area	San Francisco Bay Area

C

CDFW	California Department of Fish and Wildlife
CEQA	California Environmental Quality Act
CESA	California Endangered Species Act
cfs	cubic feet per second
CMP	corrugated metal pipe
cms	cubic meters per second
CN value	Curve Number
County	Sonoma County
CWA	federal Clean Water Act of 1972

E

EGL	energy grade line
ESA	federal Endangered Species Act

F

FCDC Manual	Flood Control Design Criteria Manual
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
FIRM	Flood Insurance Rate Map
FMDM	Flood Management Design Manual
ft	feet
ft^2	square feet
ft/sec	feet per second

G

GIS	Geographic Information System
GUI	graphical user interface

H

HARN	High Accuracy Reference Network
HEC	U.S. Army Corps of Engineers, Hydrologic Engineering Center
HGL	hydraulic grade line
HMS	Hydrologic Modeling System
hr	hour
HY-8	FHWA hydraulic analysis software package

I

IRM	Incremental Rational Method

L

lb	pounds
LID	low impact development
LiDAR	light detection and ranging (remote sensing technology)
LOMR	Letter of Map Revision

M

MAP	mean annual precipitation
min	minute
MSL	mean sea level

N

NAD 27	North American Datum of 1927
NAD 83	North American Datum of 1983
NAVD 88	North American Vertical Datum of 1988
NEPA	National Environmental Policy Act
NFIP	National Flood Insurance Program
NGVD 29	National Geodetic Vertical Datum of 1929
NMFS	National Marine Fisheries Service
NOAA	National Oceanic and Atmospheric Administration
NPDES	National Pollutant Discharge Elimination System
NRCS	Natural Resource Conservation Service

P

PFDS	Precipitation Frequency Data Server
Porter-Cologne Act	Porter-Cologne Water Quality Control Act
PVC	polyvinyl chloride

R

RAS	River System Analysis
RECP	rolled erosion control product
RWQCB	Regional Water Quality Control Board

S

sec	second
Sonoma Water	Sonoma County Water Agency
sq mi	square miles
SSURGO	Soil Survey Geographic Database
State Water Board	State Water Resources Control Board
SUHM	Synthetic Unit Hydrograph Method

T

TP-40	*Technical Paper No. 40 – Rainfall Frequency Atlas of the United States for Durations from 30 Minutes to 24 Hours and Return Periods from 1 to 100 Years*

U

USACE	U.S. Army Corps of Engineers
USDA	U.S. Department of Agriculture
USGS	U.S. Geological Survey
UTM	Universal Transverse Mercator

°F	degrees Fahrenheit

Chapter 1

Introduction

1.1 Purpose of the Flood Management Design Manual

The purpose of this manual is to guide public agencies and private entities in Sonoma County that are planning, designing, constructing, or maintaining projects that affect hydrologic processes, including infiltration, runoff, and storage. The Sonoma County Water Agency (Sonoma Water) uses its *Flood Management Design Manual* (FMDM) to provide criteria for the following hydrologic and flood management topics:

- The approach and methods to conduct hydrologic analysis to support the flood management design process;
- The approach and procedures to design flow conveyance systems;
- The analysis and design procedures for projects that will be maintained by Sonoma Water; and
- The project design review process, whereby Sonoma Water will review project designs under referral and agreement with other agencies.

These topics are further described in the chapters of this manual, with supporting materials provided in the appendices. The remaining chapters of this manual are as follows:

Chapter 2: Flood Management Design Review Process

Chapter 3: Hydrology

Chapter 4: Flow Conveyance

Chapter 5: References

> **Sonoma Water's Mission:**
> The mission of Sonoma Water is to effectively manage the water resources in our care for the benefit of people and the environment through resource and environmental stewardship, technical innovation, and responsible fiscal management.

Appendix A, *Glossary*, defines and describes technical terms used elsewhere in this FMDM. Terms cited in the glossary are indicated at their first reference in **bold type**. Appendices B-D provide detailed technical information for use by the Project Applicant. Appendix E provides three example problems that demonstrate analysis methods presented in Chapters 3 and 4. Appendix E also demonstrates how to comply with submittal requirements of this manual.

This manual was developed to be consistent with Sonoma Water's mission, which calls for the effective management of resources for the benefit of people and the environment in the Sonoma Water service area.

Prior to this version of the manual, revised in 2019, Sonoma Water's previous versions of this manual, then called the *Flood Control Design Criteria Manual* (FCDC Manual), were developed or updated in 1966, 1973, 1983, and 1999. The previous editions of the FCDC Manual guided drainage analysis and provided design criteria for the development of waterways, channels, and closed conduits. The general approach of the previous manual editions was to provide design guidance for the conveyance of large-magnitude design storms and flood events. The overall purpose of the FCDC Manual was to provide flood management guidance such that "*existing and projected building sites will be free from flood hazard in all flood events up to and including a flood of magnitude which is equaled or exceeded once in one hundred years*" (Sonoma Water 1983).

This revised FMDM (2019) is consistent with past versions of the manual in providing design criteria for hydrologic analysis and facilities to avert a flood hazard. The overall purpose of this manual has not changed.

Flood hazard is defined as the potential for loss of life or property through exposure to a large runoff, discharge, or flow event. Erosion, sediment, and/or debris carried by a large flow event are also considered part of the flood hazard.

1.2 Applicability of the FMDM

The criteria and analysis guidelines provided in this manual have multiple applications, including being used for:

- Sonoma Water–led projects;
- Land use planning jurisdictions whereby Sonoma Water provides drainage review and analysis for projects and plans; and
- Projects for which a Sonoma Water–revocable license, easement, or consent agreement is required.

Sonoma Water's Vision Statement:
Sonoma Water is a regional leader in water resources management. Sonoma Water strives to look forward, beyond today's issues, to anticipate ways to advance its mission. Additionally, Sonoma Water continues to adapt its mission in response to changing opportunities, keeping Sonoma Water at the forefront of developments in the water industry.

The criteria in this manual are applied to projects in unincorporated areas of Sonoma County that are under review by the County and Sonoma Water. The criteria in this manual are also applied when Sonoma Water reviews drainage designs, plans, or improvements for projects in the cities of Santa Rosa, Rohnert Park, Cotati, Sonoma, Petaluma, and Cloverdale and the Town of Windsor.

For the unincorporated areas of Sonoma County and for several of the County's municipalities, the goals, criteria, and procedures specified in this manual will guide the flood management design review process and govern the permitting of individual project proposals or land entitlement efforts with respect to flood management. In some cases, additional performance standards or more conservative interpretations of the criteria presented in this manual may be required by the partner agency or local jurisdiction. Chapter 2, *Flood Management Design*

Review Process, describes Sonoma Water's flood management design review process more completely.

Some local jurisdictions may use this manual to guide hydrologic analysis and flood control design but will not have Sonoma Water review individual project designs for compliance with the manual. In such cases, the local jurisdiction will provide its own review and project approval process. In that situation, standards and terms required for project maintenance will also be under the control of that local public agency, utilizing that agency's own structural and material standards (and not those of Sonoma Water).

A revocable license is necessary for any work within any easements or property Sonoma Water manages or owns. An easement or a consent agreement is required for any permanent proposed structure or feature installed within or on property Sonoma Water manages or owns. This includes any creek outfalls.

In other locations within the County for which the guidance provided in this manual may not be a requirement, this manual can be used as a reference guidebook to benefit resource management or provide consistency with the flood management approach used throughout the remainder of the County.

1.3 Flood Management Goals

The flood management goals presented in Table 1-1, "FMDM Goals," represent core values of Sonoma Water. Sonoma Water seeks to guide projects toward these goals and will use them in applying discretion when reviewing projects. In this way, goals are desirable endpoints that projects should achieve if reasonably possible. While goals represent desired general outcomes, the criteria that are presented in this manual identify more specific requirements for project approvals.

Table 1-1. FMDM Goals

Subject	Sonoma Water Goals
Flood Risk	Projects and activities should minimize the risk of flooding to human beings and property, to the extent practicable.
Public Safety	Projects and activities should protect public safety through their designs and management actions.
Water Quality	Projects and activities should enhance or protect the quality of stormwater runoff, streamflow, and groundwater.
System Dynamics	Project designs and decision making should recognize and accommodate that streams and hydrologic systems are highly dynamic environments.
Integrated Water Management	Projects and activities should consider how hydrologic and flood management conditions can be integrated with other water resources, including managing stormwater, other surface water flows, groundwater, and water supply.
Water Supply	Projects and activities should, where suitable, capture, store, infiltrate, and reuse stormwater and support groundwater recharge as a means of protecting and enhancing water supply.
Multiple Benefits	Projects and activities should seek multiple beneficial outcomes through their design and implementation.
Ecologic Systems	Projects, activities, and management decisions should protect and, where feasible, enhance ecologic systems.
Aesthetic Value	Projects and activities should protect and, where feasible, enhance the aesthetic value of project sites.
Public Use	Projects, activities, and management decisions should incorporate public use of, and access to, stream corridors and public lands where such uses can reasonably be made compatible with surrounding land uses.

1.4 Limitations

The paragraphs below describe limitations that manual users should consider. These limitations relate to the intended use of the FMDM for engineering design; ongoing updates to the manual; and issues related to stormwater regulations, the **National Flood Insurance Program** (NFIP), and other environmental compliance requirements.

1.4.1 Engineering Design

The hydrology and flood management design criteria and guidance contained in this manual are not intended to address all details of engineering design or to be a substitute for sound engineering judgment. Individual project reviews to be conducted by Sonoma Water will evaluate projects for their conformance with the design criteria and qualitative guidance contained in this manual. In most cases, conformance with these criteria will be the basis for Sonoma Water approval. In some cases, however, the substitution of criteria may be approved at the discretion of the Sonoma Water General Manager, or designee, when such a waiver would be in the best interests of the public, based on sound engineering judgment, and consistent with Sonoma Water principles and policies, including those enumerated in this document.

1.4.2 Climate Change

Uncertainties regarding climate change are raising several critical questions and creating challenges for flood management design. Changes in the level of storm intensity and associated runoff may be expected under climate change, though the degree and direction of that change remains uncertain in Sonoma County. Forecasts of sea level rise are more readily available, though considerable uncertainty remains. To address potential sea level rise increases, use of the most current guidance from the State of California[1] and the best available information is recommended.

At this time, Sonoma Water has not established design criteria or specific methodologies to address climate change with respect to either runoff or sea level increases. Nonetheless, climate change should be considered as part of the project design approach, such that the design is as robust as is warranted by the associated flood risk under the anticipated potential range of hydrologic conditions. Such an approach can be expected to increase the efficiency of environmental and regulatory review.

1.4.3 Using the Current Version of the FMDM

It is anticipated that this manual will be revised from time to time as procedural, regulatory, and/or technical changes require updates. Sonoma Water will provide the most current version of this manual at its website (www.sonomawater.org/flood-protection). It is the responsibility of users to ensure that they are using the most current version of the manual.

1.4.4 Stormwater Regulations

Nonpoint-source runoff, including runoff from impervious surfaces, is generally referred to as stormwater runoff. Potential pollutants carried in stormwater runoff are regulated as a water quality concern. Stormwater runoff may also cause channel erosion due to increased peak flow rates or volumes from urbanized areas. The federal Clean Water Act of 1972 (CWA), Section 402

[1] As of this time, the latest (2018) guidance from the State of California is available at http://www.opc.ca.gov/webmaster/ftp/pdf/agenda_items/20180314/Item3_Exhibit-A_OPC_SLR_Guidance-rd3.pdf.

established the National Pollutant Discharge Elimination System (NPDES) permit program to regulate discharges of pollutants to surface waters. Under the NPDES, public agencies (such as cities, counties, and other agencies) are required to maintain compliance with the conditions of NPDES permits for their stormwater discharges. The municipalities, in turn, require that individual projects within their jurisdictions comply with the requirements of these permits.

The NPDES program is administered at the state level by the California State Water Resources Control Board (State Water Board) and nine Regional Water Quality Control Boards (RWQCBs). In Sonoma County, projects located within the Petaluma River and Sonoma Creek watersheds are regulated by the San Francisco Bay RWQCB (Region 2) and the remainder of Sonoma County is regulated by the North Coast RWQCB (Region 1).

The focus of this FMDM is to provide hydrologic and hydraulic analysis methods and criteria for designing facilities to accommodate flood conditions. The focus of this manual is not to address the more frequent and lower magnitude stormwater flows that are typically the focus of NPDES requirements. The user of this manual, or project applicant, should consult with the appropriate RWQCB office and/or appropriate municipality (or the County) regarding potential NPDES regulatory requirements that may affect a specific project.

1.4.5 National Flood Insurance Program

The **Federal Emergency Management Agency** (FEMA) administers the NFIP to provide subsidized flood insurance to communities that comply with FEMA regulations limiting development in floodplains. FEMA issues **Flood Insurance Rate Maps** (FIRMs) for communities participating in the NFIP. These maps delineate flood hazard zones in the community. Flood insurance is commonly required for homes located in high-risk areas (those areas that have a 1% or greater chance of flooding in any given year). Homes not located in high-risk areas are typically not required to have flood insurance. FIRMs can be reviewed online at the FEMA Map Service Center website (www.msc.fema.gov).

The NFIP is primarily an insurance program for individuals and their properties. Sonoma Water does not participate in the NFIP and is not involved with flood insurance, flood mapping, or updates to FIRMs. Nonetheless, analysis conducted in accordance with this manual may be suitable for use in complying with FEMA requirements or developing material for FEMA submittals, such as an application for a Letter of Map Revision (LOMR). If such a dual purpose is sought for hydrologic or hydraulic analysis or design, the applicant is advised to consider FEMA requirements before selecting an approach that is also consistent with Sonoma Water guidance and requirements as described in this FMDM.

It is also noted that several communities within Sonoma County have prepared stormwater or storm drain master plans. These other local planning efforts may include detailed hydrologic modeling results. Other local planning documents should be used solely for the intended use of the agency they were developed for and not used as a substitute for FEMA maps and Flood Insurance Reports.

1.4.6 Other Environmental Compliance Requirements

Public discretionary activities, as well as private projects that involve potentially significant environmental consequences, may require compliance with federal and state environmental regulations. Project applicants that are using the FMDM to guide and evaluate project hydrologic and hydraulic outcomes and design options may also be required to undertake federal and state environmental compliance and permitting to advance their projects. The primary federal and state regulations governing protection of environmental resources include:

- National Environmental Protection Act (NEPA)
- California Environmental Quality Act (CEQA)
- Federal Clean Water Act (CWA)
- California Porter-Cologne Water Quality Control Act (Porter-Cologne Act)
- Federal Endangered Species Act (ESA)
- California Endangered Species Act (CESA)

The user of this manual is encouraged to identify and seek-out project solutions that not only provide adequate flood protection, but also, to the extent possible, provide ecologic benefits, or at least avoid and minimize potential impacts to habitats and other beneficial uses to the greatest extent possible. This approach is reflected in the Multiple Benefits and Ecologic System goals in Table 1-1 above that summarizes FMDM goals. The regulatory approval process will typically require the project sponsor to demonstrate that potential impacts have been avoided and minimized to the greatest extent practically feasible. While this manual does not provide specific direction or guidance on how to avoid or reduce potential impacts, as each project situation is different, Sonoma Water promotes that users of this manual consider these issues during the engineering design process. Furthermore, the project sponsor who does consider such issues during the engineering design process, and can avoid or minimize potential effects, will likely benefit from some streamlining during the regulatory approval process.

In addition to these state and federal authorizations, the manual user must also check with local jurisdictions regarding ordinances and regulations that may affect their project. For example, Section 4.2.1.2 of Chapter 4 in this manual describes local Sonoma County stream setback ordinances that must be followed for projects adjacent to creeks and waterways.

It is the responsibility of the user of this FMDM to determine the project's status in relation to these and any other applicable environmental acts and regulations. Depending on the project, the requirements of the statutes identified above may not be comprehensive. For example, other biological and cultural resources protection regulations may apply to certain projects.

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Chapter 2

Flood Management Design Review Process

2.1 Introduction

This chapter describes the Sonoma County Water Agency's (Sonoma Water's) flood management design review process. This is the process by which Sonoma Water reviews the designs for flood management systems that are submitted by project applicants. The purpose of this chapter is to provide project applicants with an overview of the administrative sequence of the flood management design review process. The flood management design review process consists of seven iterative steps:

Step 1: Application submittal and project initiation

Step 2: Completeness review of initial Applicant submittal

Step 3: Hydrologic and hydraulic parameter screening

Step 4: Detailed flood management design review

Step 5: Submittal revision

Step 6: Flood management design review compliance

Step 7: Internal Sonoma Water project filing

These steps are described further in Section 2.2 below. Forms discussed in this chapter are provided in Appendix B, *Flood Management Design Review Process Forms and Materials.*

2.2 Flood Management Design Review Process

Step 1: Application Submittal and Project Initiation

The design review process is initiated when the Applicant submits a complete project application, which consists of the following:

Development Submittal Information Form (Appendix B.1): This form provides Sonoma Water with basic information regarding the project, including project name, location, size, and contact information for the owner/developer and drainage engineer.

Flood Management Design Review Plan Submittal Checklist (Appendix B.2): This form identifies the required materials that constitute a complete project application. To initiate the design review process, all of the items on the Plan Submittal Checklist must be submitted to Sonoma Water. This includes the development plans and all analyses and calculations made to design the drainage system. See Appendix E, *Example Problems*, for examples of some of the required information to be submitted. If an item in the Plan Submittal Checklist is considered unnecessary by the Applicant's design engineer, then the Applicant must explain why that information is not needed.

Applicant Submittal Log (Appendix B.3): The Applicant Submittal Log is to be used by the Applicant to log and track each formal submittal to Sonoma Water. The Submittal Log will include tracking of newly submitted materials for Sonoma Water review; as well as tracking of revised versions of submittals that are provided to Sonoma Water for additional review or updates.

Fee Schedule for Design Review: The initial application fee must be paid to initiate the design review process. The application fee is based on the project type and scale as identified in Appendix B.1. Current fee information is available online at https://www.sonomawater.org/flood-protection . If the level of effort exceeds the cost of the initial fee during the review, an invoice will be sent to the Applicant to recover all the costs necessary to perform the review.

Once a project is submitted, there will be no refunds for incomplete project reviews or terminated projects.

If Sonoma Water deems the application complete, it will assign the project an internal project name and a project tracking number according to the project location. Sonoma Water will enter the project into a database to track and log the key steps of the design review process.

Step 2: Completeness Review of Initial Applicant Submittal

Sonoma Water will review the Applicant's project materials submitted in Step 1 for completeness. Sonoma Water will review project details according to the Flood Management Design Review Plan Submittal Checklist (Appendix B.2). If the submittal is deemed complete, Sonoma Water will notify the Applicant (via email or telephone call) that the submittal is considered complete, and the review will proceed to Step 3.

If the submittal is deemed incomplete, Sonoma Water will notify the Applicant and may provide specific guidance as to the deficiencies of the submittal. Sonoma Water may communicate with the Applicant informally through email or by telephone; or may provide formal notification via hard-copy letter. If the Applicant's submitted materials are unclear or confusing, or otherwise do not comply with the submittal instructions and Plan Submittal Checklist, Sonoma Water is unlikely to provide detailed guidance to the Applicant until the basic submittal requirements are met.

Step 3: Hydrologic and Hydraulic Parameter Screening

Once the application has been deemed complete, Sonoma Water will conduct a preliminary screening of the basic hydrologic and hydraulic parameters used in the design calculations. This may include an initial review of the design storm, selected precipitation values, rainfall intensity, and runoff coefficients (**C factors**) or Curve Numbers (*CN* values). This initial review will also verify that the Applicant has used appropriate existing hydraulic grade lines as baselines and/or boundary conditions. Chapter 3, *Hydrology*, and Chapter 4, *Flow Conveyance*, of this *Flood Management Design Manual* (FMDM) identify specific hydrologic and hydraulic parameters that are required for the Applicant's analysis and design. The submitted information will be cross-referenced with the requirements identified in Chapters 3 and 4 during the preliminary hydrologic and hydraulic parameter screening. If any errors or omissions in basic hydrologic and hydraulic parameters or assumptions are identified during the screening process in Step 3, the Applicant will be asked to revise and resubmit the application.

Step 4: Detailed Flood Management Design Review

Following the parameter screening in Step 3, and after confirming that the Applicant has used appropriate assumptions for the design storm and hydrologic and hydraulic parameters, hydrologic analysis method selection, and hydraulic boundary conditions, Sonoma Water will perform a more detailed review of the project's hydrologic and hydraulic evaluation and design review in Step 4.

This step of the review will closely follow the Flood Management Design Review Plan Submittal Checklist (Appendix B.2). If, following Sonoma Water's detailed design review, the application is considered complete and found in conformance with the criteria (without any outstanding issues), then Sonoma Water will proceed to Step 6 and provide the Applicant with a Compliance Letter. However, if the detailed design review results in questions regarding the project, hydrologic analysis, hydraulic analysis, or flood management design, then Sonoma Water will provide the Applicant with comments and/or questions and request a revised submittal as described in Step 5 below.

Step 5: Submittal Revision

If the project submittal requires revision (based on the review conducted in Step 4), Sonoma Water will issue a hard-copy **Revision Letter** to the Applicant. The Revision Letter will identify comments, questions, or other requests to revise or clarify the submittal. The Revision Letter will enumerate each specific comment or question that the Applicant is required to address.

When submitting a revised application, the Applicant will reference the specific comment numbers and issues provided in the Revision Letter. The Applicant is required to address the comments in the Revision Letter. The Applicant must respond to and address the entire list of comments or questions. If necessary, the Applicant can schedule a phone call or meeting with Sonoma Water to request clarification.

For large projects with complex drainage systems (e.g., those with multiple storm drain outlets or subwatersheds), the entire flood management plan and application package need not be resubmitted. Rather, for these situations, Sonoma Water will accept a resubmittal of those portions of the drainage system that require amendments or revisions. In most cases, a large or complex project can be subdivided spatially based on storm drain outlets, channel confluences, or watershed/subwatershed boundaries. In such a situation, the information for the subwatershed in question can be resubmitted, without resubmitting the entire flood management plan. Revised submittals shall clearly indicate the project component or module that has been revised for replacement in the overall submittal application and include an updated Applicant Submittal Log.

Following the submittal of a revised application or component of the application, Sonoma Water will issue a letter to the Applicant that identifies any further revisions or comments on the submitted information. The standard review process assumes up to three rounds of review by Sonoma Water staff but may require additional review in some circumstances. The three rounds of review include the initial screening and design review of Steps 3 and 4 and up to two additional rounds of review in Step 5. The initial fee was established assuming typical costs for this level of review. Sonoma Water tracks the cost to review each project application. If the initial fee is exceeded, Sonoma Water will invoice the Applicant the balance due in Step 6.

Once the Applicant has adequately addressed all of Sonoma Water’s comments, the review process will proceed to Step 6.

Step 6: Flood Management Design Review Compliance

Following Sonoma Water review of the flood management submittal (Step 4) and any required subsequent revisions or amendments (Step 5), Sonoma Water will determine if the project submittal is compliant with the FMDM. As previously described, Sonoma Water will also review its costs, and if the cost covered by the initial fee is exceeded, Sonoma Water will invoice the Applicant for the remaining charges. As appropriate, Sonoma Water will notify the Applicant of the finding and request that the complete project submittal package be submitted in electronic form in one or more specified file formats. The package must also include a digital file with a numbered list (consistent with an up-to-date final Applicant Submittal Log) of file names. On receipt of a complete package and any additional fee (if necessary), Sonoma Water will issue a hard-copy **Compliance Letter** to the Applicant that verifies the project is compliant with Sonoma Water’s FMDM.

Step 7: Internal Sonoma Water Project Filing

Following issuance of the Flood Management Compliance Letter to the Applicant, Sonoma Water will collect, organize, and retain relevant project files, including project communications, the Applicant’s final electronic submittal package, reports, maps, and other supporting documents considered necessary to retain in the project file. Sonoma Water will archive the project file for future reference as necessary.

Chapter 3
Hydrology

3.1 Introduction

This Chapter describes the Sonoma County Water Agency's (Sonoma Water's) hydrologic analysis methods for use in project evaluation and hydraulic facility design. In no instance is the direction provided in this *Flood Management Design Manual* (FMDM) intended to substitute for professional engineering judgment. Use of approaches and parameters differing from those presented in this FMDM will require Sonoma Water approval. For complex or unusual projects, it is the responsibility of the user to contact Sonoma Water regarding the application of these methods. Sonoma Water may impose additional or different requirements in selected cases. This Chapter also provides an introduction to the hydrologic setting in Sonoma County.

The hydrologic analysis methods are illustrated in **Appendix E**. Appendix E.1, Example Problem 1 presents an example of the Incremental Rational Method (IRM) analysis and Appendix E.2, Example Problem 2 presents an example of the Synthetic Unit Hydrograph Method (SUHM) analysis.

3.2 General Considerations: Hydrologic Setting

This section briefly introduces the hydrologic setting of Sonoma County to provide context for the rainfall and **runoff** topics discussed in the remainder of Chapter 3. This section includes discussion of the following topics:

- Topography and landforms
- Sediment and soils
- Climate and precipitation
- Runoff and streamflow
- Water quality
- Major watersheds
- Tidally-influenced systems

3.2.1 Topography and Landforms

The topography of Sonoma County reflects its dynamic history of tectonism, compression, and mountain building along the San Andreas Fault Zone and Coast Ranges (Nilsen 1987). The northwest-southeast alignment of the San Andreas Fault Zone with its characteristic right-lateral strike-slip tensional movement is reflected in the alignment and orientation of the region's northwest-to-southeast–aligned ridgelines and valleys. This alignment has hydrologic implications as it influences where streams are located and the direction in which they flow. Characteristic mountain ridge, alluvial fan, alluvial plain, and alluvial lowland features associated with this geologic setting are seen throughout Sonoma County, as described in the following paragraphs.

The Mayacamas Mountains separate Sonoma and Napa Counties and provide the **headwater** source areas for runoff and sediment to many local streams, including Laguna de Santa Rosa and Sonoma Creek. Steep canyons and mountain streams carry flows and sediment from these source areas to the plains below. Historically, alluvial fans and plains functioned as depositional areas where sediments were stored in the topographic transition between the higher and steeper mountains to the east and the lower and more gently sloping plains to the west. Streams migrated across these alluvial surfaces, distributing sediment eroded from the mountains. Alluvial sediments are also stored in relict terraces where streams previously flowed. Much of the Santa Rosa Plain west of U.S. Highway 101, moving toward the Laguna de Santa Rosa, Mark West Creek, and Santa Rosa Creek systems, has sediment stored in historic terrace deposits.

The county's most significant lowlands include the Laguna de Santa Rosa and the lower **floodplains** and marshes of the Petaluma River and Sonoma Creek watersheds. These lowlands are important regional ecosystems that include many sensitive habitats. The Laguna de Santa Rosa is a unique hydrologic feature in Sonoma County, supporting a mosaic of channels, oxbows, ponds, and wetlands. In the winter, a natural basin at the confluence of the Laguna and Mark West Creek receives backwatered flood flows from the Russian River. This basin effect detains and captures other Laguna watershed flows, providing flood storage and peak attenuation benefits for the lower Russian River valley. This natural flood storage is advantageous for managing flood waters in the lower Russian River watershed downstream. However, elevated water levels in the Laguna can also cause backwatering in the tributaries upstream of the Laguna, which increases the flood risk in upstream tributaries.

3.2.2 Sediment and Soils

Sediment deposition patterns in the county are closely tied to the topographic features described above. For example, in the Rohnert Park area, alluvial sediments transition from coarse-grained to medium-grained to fine-grained, moving west from the Sonoma Mountains down into the alluvial plain, with a corresponding decline in **infiltration** rates. These soil and landform patterns have implications for hydrologic planning. For example, the Rohnert Park urban area is notable for its clay soils and poor infiltration capacity. The Santa Rosa area shows generally better infiltration conditions where coarse-grained soils are found along the pathways of the current channels and the historic creeks and terrace deposits. As one moves away from the active channel areas on the wider Santa Rosa plain, the soils consist of older, finer-grained alluvium, with some reductions in infiltration capacity. In the Laguna de Santa Rosa, the texture of sediments also becomes finer moving north from the Sebastopol area, where stream cobbles and coarse sediments transition to medium-textured materials. Finer alluvium is found farther north in the more depositional basin of the Laguna de Santa Rosa, south of the Mark West Creek confluence.

There are several known and mapped large landslides in the hills of Sonoma County. These landslides are significant sediment sources for the streams and channels downstream. Landslides are prominent in the headwaters to Copeland, Crane, Five, and Hinebaugh Creeks in the Rohnert Park area. Additional landslides and gullies in the headwaters of Cook and Coleman Creeks have exacerbated downstream sedimentation. In the Petaluma River watershed, a large landslide at the common headwater areas to Lichau, Willow Brook, Lynch, Washington, and Adobe Creeks also supplies the creeks with eroded materials. Any project applicant working in a hillslope environment in Sonoma County should evaluate the historic and potential landslide and erosion conditions at the project site (although that evaluation is outside the purview of this Manual or Sonoma Water's hydrologic review).

The *Soil Survey of Sonoma County, California* (U.S. Department of Agriculture, Soil Conservation Service [SCS] 1972) includes 15 soil associations. At the association level, soils are generally distinguished according to their geomorphic and topographic setting: whether they are located in basins, tidal flats, floodplains, terraces, alluvial fans, high terraces, foothills, uplands, or mountains. In general, the soils in the lowland basins, floodplains, and alluvial plains range from gravelly sandy loams to clays, most often composed of clays and clay loams that formed in alluvium from sedimentary and volcanic material. These soils vary in drainage capacity from poor to excessive, with the more clay-textured soils draining more poorly. The soils on the high terraces, foothills, uplands, and mountains consist of gravelly to stony sandy loams to clay loams and range in drainage capacity from moderate to excessive, with the coarser-textured soils draining better. The *Soil Survey of Sonoma County, California* (SCS 1972) and the Soil Survey Geographic Database (SSURGO) can be used to identify more site-specific or parcel-specific soil series, types, and their hydrologic properties. The U.S. Department of Agriculture, Natural Resources Conservation Service (NRCS) provides online soil survey information at: websoilsurvey.sc.egov.usda.gov/App/HomePage.htm.

3.2.3 Climate and Precipitation

The climate of Sonoma County is characterized as two-season Mediterranean with cool, wet winters and warm, dry summers. Annual, seasonal, and geographic variability in temperatures and rainfall are high. For the period 1931–2006, average maximum temperatures for the area ranged from the low 80s (all temperatures in degrees Fahrenheit [^{o}F]) in summer to high 50s in winter, while average minimum temperatures ranged from low 50s in summer to upper 30s in winter (Western Regional Climate Center 2007).

Precipitation falls primarily between November and March and varies across the county, from about 25 inches annually at the margin of San Pablo Bay in the south to 80 inches on the western slope of Cobb Mountain (4,706 feet [ft] above mean sea level [MSL]). The Santa Rosa area in central Sonoma County has an average annual rainfall of about 30 inches. Winter storm fronts typically arrive from the west, southwest, or northwest, and orographic lifting of air masses above mountain ranges causes increased precipitation across the Mayacamas and Sonoma Mountains. The eastern sides of these ranges typically receive less precipitation due to a rain-shadow effect. Some cyclonic storms approach from a more direct southerly direction, beginning at San Pablo Bay and moving northward up the Petaluma and Sonoma Valleys and Santa Rosa Plain.

3.2.4 Runoff and Streamflow

Runoff includes a range of hydrologic flow processes, from surface sheetwash (or **overland flow**) to the collection of surface flows into small rills and land creases. **Streamflow** describes the concentration of runoff into natural creeks or engineered channels. The amount and timing of runoff and streamflow over a given period (e.g., a single storm event, a season, or a year) reflect the regional climate and the watershed's topographic, geologic, and soil conditions. Steeper surfaces generate runoff more quickly than flatter surfaces. Soils and ground covers with higher infiltration capacity (**porosity** and **permeability**) generate less runoff as more precipitation can infiltrate into the ground.

Surface water that does not infiltrate into the soil or groundwater, evaporate into the atmosphere, or become transpired by plants may be available as runoff to streams. In Sonoma County, streams may be ephemeral (conveying flows only immediately after a storm event), intermittent (conveying flows seasonally and supported by shallow groundwater), or perennial (flowing year-round and supported through deeper groundwater sources or human sources such as reservoirs, release of imported flows, urban runoff, or irrigation).

In Sonoma County, surface water hydrology and runoff in streams are a function of watershed size, underlying geology, recent precipitation conditions, and land use. Flow characteristics for creeks, particularly the seasonal duration of intermittent flow, vary according to climatic conditions and how wet or dry the current and past few winter seasons have been. While some channels may not flow perennially, they may sustain cold-water pools throughout the year (particularly where substrate, shading, and groundwater conditions are favorable) that can provide important habitat for many species.

Land development, including urban, suburban, agricultural, and commercial development of the land surface, directly affects the hydrologic cycle, most notably by changing infiltration and runoff conditions. Development typically increases the area of impervious surfaces, with the result of decreased infiltration and increased runoff (stormwater). Urbanized areas typically have engineered storm drain systems to convey the increased runoff to local streams. Urbanized areas with summer irrigation also typically show an increase in summer flows. Creeks that were previously ephemeral or intermittent (seasonal) may become perennial as a result of urban irrigation and dry season runoff.

Several areas of Sonoma County now have low impact development (LID) requirements that seek to minimize the hydrologic effects of urbanization and increased runoff. Section 1.4, “Limitations,” in Chapter 1 describes reference materials that the Applicant should review to determine whether a project site is subject to stormwater permit conditions and LID treatment requirements.

3.2.5 Water Quality

This section presents an overview of water quality conditions related to erosion and sediment, temperature, nutrients, and pathogens and urban contaminants in Sonoma County.

3.2.5.1 Erosion and Sediment

Watersheds are integrated physical systems that move water and eroded sediment from higher to lower locations. Sediments are eroded, deposited, or stored in place within the stream network. Several physical and biological conditions influence erosion and sediment processes in a watershed, including geology, topography and slope, climate and precipitation, soils and vegetation, and the hydrologic conditions of infiltration, runoff, and streamflow. Beyond these physical influences, land use practices, history, and structures further influence erosion and sediment processes. The intensification of land uses through agriculture, grazing, fire management, mining, recreation, and residential and commercial development may also result in increased erosion. Many downstream locations of creek or flood control channels in Sonoma County receive abundant sediment from upstream sources.

3.2.5.2 Temperature

Parameters that influence stream temperature include ambient air temperature, humidity, riparian vegetation, topography, surrounding land use, and flow conditions. Coldwater seeps and groundwater inputs may contribute to moderating and lowering stream water temperatures. Seasonally reduced flows in summer, together with greater solar radiation, result in higher water temperatures. While shading creeks helps to decrease water temperatures, runoff received from urbanized areas may have relatively high water temperatures compared to those from non-urbanized areas. Summer air temperatures in Sonoma County often exceed 90°F. Streams flowing across the Santa Rosa Plain may have historically experienced relatively warm water temperatures prior to development. These conditions are not necessarily indicative of poor water quality and can provide important habitat opportunities for native warm water fish assemblages.

3.2.5.3 Nutrients

Nutrients, specifically nitrogen and phosphorus, are essential for life and play a primary role in ecosystem functions. Nitrogen and phosphorus are naturally occurring inorganic ions present in the atmosphere and in fixed forms within organic matter, such as plants and soils. In addition to naturally present concentrations, nutrients are introduced to water bodies through human or animal waste disposal and agricultural application of fertilizers. Nutrients are commonly the limiting factor for growth in aquatic systems. In freshwater streams of the San Francisco Bay Area (Bay Area), nitrogen is the limiting nutrient (Krottje and Whyte 2003).

Many types of activities, such as agriculture, land development, and channelization of urban runoff, can result in excessive loading of nutrients to water bodies. Excessive nutrient loading in streams can produce toxic or eutrophic conditions, both of which impair aquatic life. Eutrophication can lead to increased algal growth and reduced oxygen levels in the water, thus also reducing aesthetic quality and habitat value. Sediments contain nutrients that stimulate the growth of invasive aquatic weeds. Nutrients can leach from sediments and cause eutrophication in water. Sediments can also deplete oxygen levels from water. Ammonia is a plant growth nutrient that is also toxic to aquatic life under elevated pH conditions associated with eutrophication.

3.2.5.4 Pathogens and Urban Contaminants

Pathogens are microorganisms that cause diseases in other organisms. Bacteria are the primary indicator organisms of pathogens, particularly for the detection of waterborne diseases. Waterborne diseases threaten the health of wildlife and recreational users of waters. Pathogenic bacteria contained within fecal waste are the most common source of waterborne diseases. Fecal contamination can be detected by bacterial indicators, such as total coliforms, fecal coliforms, *Escherichia coli* (*E. coli*), and fecal enterococci. High concentrations of these indicator bacteria – resulting from poor waste management and disposal systems, and sometimes from homeless encampments along the creek banks – can degrade water quality for human consumption, recreation, and wildlife use.

Many stream channels in Sonoma County support native wildlife and plant species. Urban contaminants, especially those that accumulate in the stream sediment, can be harmful to people and wildlife in the community. In urbanized settings, water quality can be adversely affected by petroleum products such as gasoline and diesel, household hazardous wastes such as chlorine-based cleansers and insect sprays, detergent byproducts, and other manufactured chemicals. It is common for these contaminants to be deposited onto the street or flushed directly into storm drains. Most storm drains throughout Sonoma County discharge directly to stream channels, not to treatment facilities. Therefore, during rain events in which runoff occurs, water quality contaminants are conveyed to stream channels.

3.2.6 Major Watersheds

The boundaries of the primary watersheds of Sonoma County are shown in Figure 3-1, "Primary Watersheds of Sonoma County."

3.2.6.1 Russian River Watershed

Approximately 650 square miles (sq mi) of the 1,500-sq-mi Russian River watershed lie in Sonoma County, with the remainder in Mendocino County (Figure 3-2, "Russian River Watershed Map"). In northern and central Sonoma County, the Russian River flows south through a series of river valleys that includes the communities of Cloverdale, Geyserville, and Healdsburg. Farther downstream, the Russian River curves westward through the Coast Ranges and flows to the Pacific Ocean near the town of Jenner. Primary tributaries to the Russian River in the northern and central portions of the county include Austin Creek, Dry Creek, Big Sulphur Creek, and Mayacama Creek. Aside from U.S. Highway 101, relatively little urban development has occurred in the northern portion of the watershed within the county, where the predominant land uses are rural residential, grazing, vineyards, and timber production. The Lake Sonoma reservoir is located on Dry Creek, southwest of Cloverdale.

3.2.6.2 Laguna de Santa Rosa Watershed

The Laguna de Santa Rosa watershed (253 sq mi) is the largest southern tributary to the Russian River (Figure 3-3, "Laguna de Santa Rosa Watershed Map," provided at the end of this chapter). The highest elevation in the watershed, at 2,463 ft above MSL, occurs along the eastern mountains. Elevation at the confluence with Mark West Creek is 50 ft above MSL. Several steep, high-gradient creeks originate in the mountains of the upper Laguna de Santa Rosa watershed. These smaller tributaries combine and collect into primary subbasin creeks that then flow west across the Santa Rosa Plain. These creeks eventually meet the Laguna main channel that flows north and joins the Russian River.

Flood elevations in inland watersheds are typically controlled by streamflow from upstream and channel conditions. Under certain conditions, however, flood elevations can be driven by high downstream water levels or even upstream flow. In the Laguna de Santa Rosa, water levels in the Laguna de Santa Rosa may be controlled by the influx of flood water from the Russian River, downstream. High water levels on the Laguna de Santa Rosa can result in ponding in the lower reaches of its tributaries, such as lower Santa Rosa Creek, where backwater conditions may extend well upstream of Willowside Road.

Land uses in the Laguna de Santa Rosa watershed are varied and include high-density urban development, rural residential uses, public recreational areas, a range of agricultural uses, and some rangeland. Higher elevation areas on the east side of the watershed include public lands, rural residential, agriculture (vineyards), and open ranchlands and woodlands. In the central watershed west of the mountain headwaters, residential and commercial land uses intensify, particularly in the urban areas of Santa Rosa and Rohnert Park. Land uses shift to rural residential and more intensive agriculture moving farther west across the Santa Rosa Plain.

3.2.6.3 Petaluma River Watershed

The Petaluma River watershed (146 sq mi) includes areas in both Sonoma County (112 sq mi of the watershed) and Marin County (34 sq mi of the watershed) (Figure 3-4, "Petaluma River Watershed Map," provided at the end of this chapter). The highest elevation in the watershed is Sonoma Mountain (2,295 ft above MSL). In the northeastern Petaluma River watershed, tributaries generally flow southwest out of the Sonoma Mountains toward the Petaluma River, which then flows southeast to San Pablo Bay. The Petaluma Valley in the central watershed forms a wide basin, with characteristic rolling hills and grasslands, that stretches from Cotati southeast to San Pablo Bay. The Petaluma River experiences tidal influence from San Pablo Bay as far as 14 miles upstream, upstream of downtown Petaluma (City of Petaluma 2008). The lower Petaluma River has been modified to enable access for commercial traffic and is dredged to maintain navigability.

Flooding along the Petaluma River and its lower tributaries is exacerbated during high tide events when elevated river levels restrict discharges from low-lying storm drains and channels, causing upstream backwatering. Additionally, significant inflows from tributaries, such as Willow Brook Creek, can create local backwater zones in the Petaluma River during flood events.

The most common land uses in the Petaluma River watershed include agriculture, rural residential, and urban areas in the City of Petaluma. The urban area of Petaluma is located in the central watershed, surrounded by agricultural lands to the east and a buffer of rural residential areas to the west abutting more agricultural lands.

3.2.6.4 Sonoma Creek Watershed

The Sonoma Creek watershed (170 sq mi) is located east of the Petaluma River watershed (Figure 3-5, "Sonoma Creek Watershed Map," provided at the end of this chapter). The watershed is generally symmetrical along the Sonoma Valley, with small tributaries descending from the Mayacamas Mountains to the east and the Sonoma Mountains to the west. Elevations in the watershed range from about 2,500 ft above MSL at Bald Mountain to sea level. Tidal influence extends upstream from San Pablo Bay north to State Highway 121. This tidal influence affects flooding conditions in the lower watershed, but does not reach as far upstream into the watershed as the Petaluma River. The most problematic areas for flooding in the Sonoma Creek watershed are agricultural and rural residential areas near Schellville and south of State Highway 121, within the marshy margin of San Pablo Bay. Land uses in the Sonoma Creek watershed are mixed but include a high percentage of both agriculture and ranchland/woodland land uses. The City of Sonoma is the main urban area within the watershed.

Elevation (feet)

Below Sea Level	250 - 500
0 - 25	500 - 1,000
25 - 50	1,000 - 1,500
50 - 75	1,500 - 2,000
75 - 100	2,000 - 2,500
100 - 250	Above 2,500

Watershed Boundary
SCWA Flood Control Zone Boundary
Water Body
Highway
Major Drainage

Figure 3-1
Primary Watersheds
of
Sonoma County

Horizon

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Sonoma County Water Agency

Flood Management Design Manual (FMDM)

Russian River Watershed

SCWA Flood Control Zone Boundary

Water Body

Major Tributaries

Russian River

Laguna de Santa Rosa Watershed - See Figure 3-3

Figure 3-2

Russian River Watershed Map

0 1 2 4 6 8

Miles

Horizon

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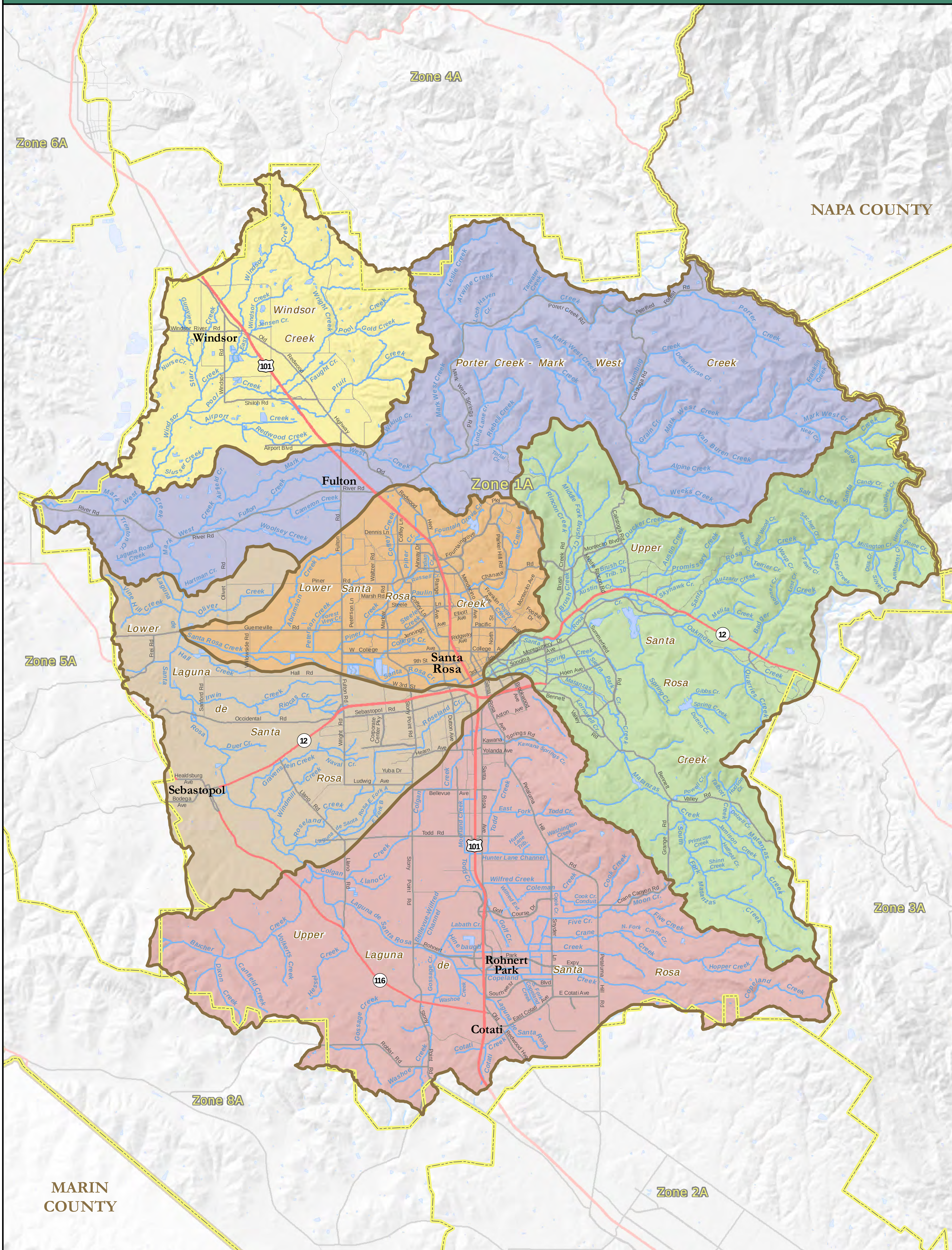
Sonoma County Water Agency

Flood Management Design Manual (FMDM)



Major Sub-basins

- Lower Laguna De Santa Rosa
- Lower Santa Rosa Creek
- Porter Creek-Mark West Creek
- Upper Laguna De Santa Rosa
- Upper Santa Rosa Creek
- Windsor Creek

- SCWA Flood Control Zone Boundary
- Water Body
- Streams and Channels
- Highways
- Roads and Major Streets

Figure 3-3
Laguna de Santa Rosa Watershed Map

0 0.5 1 2 3 4
Miles

Horizon WATER and ENVIRONMENT

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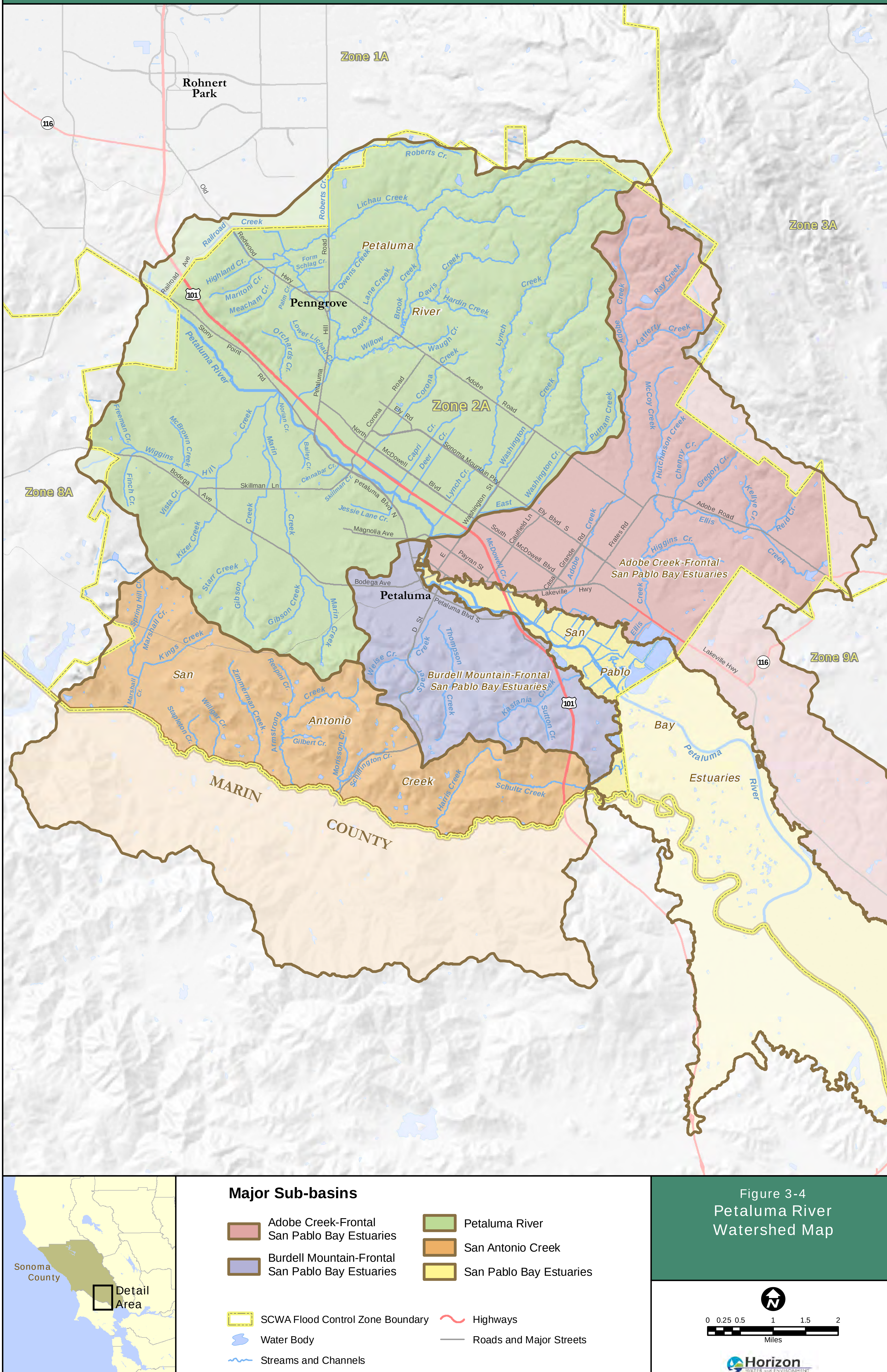

Figure 3-4
Petaluma River Watershed Map

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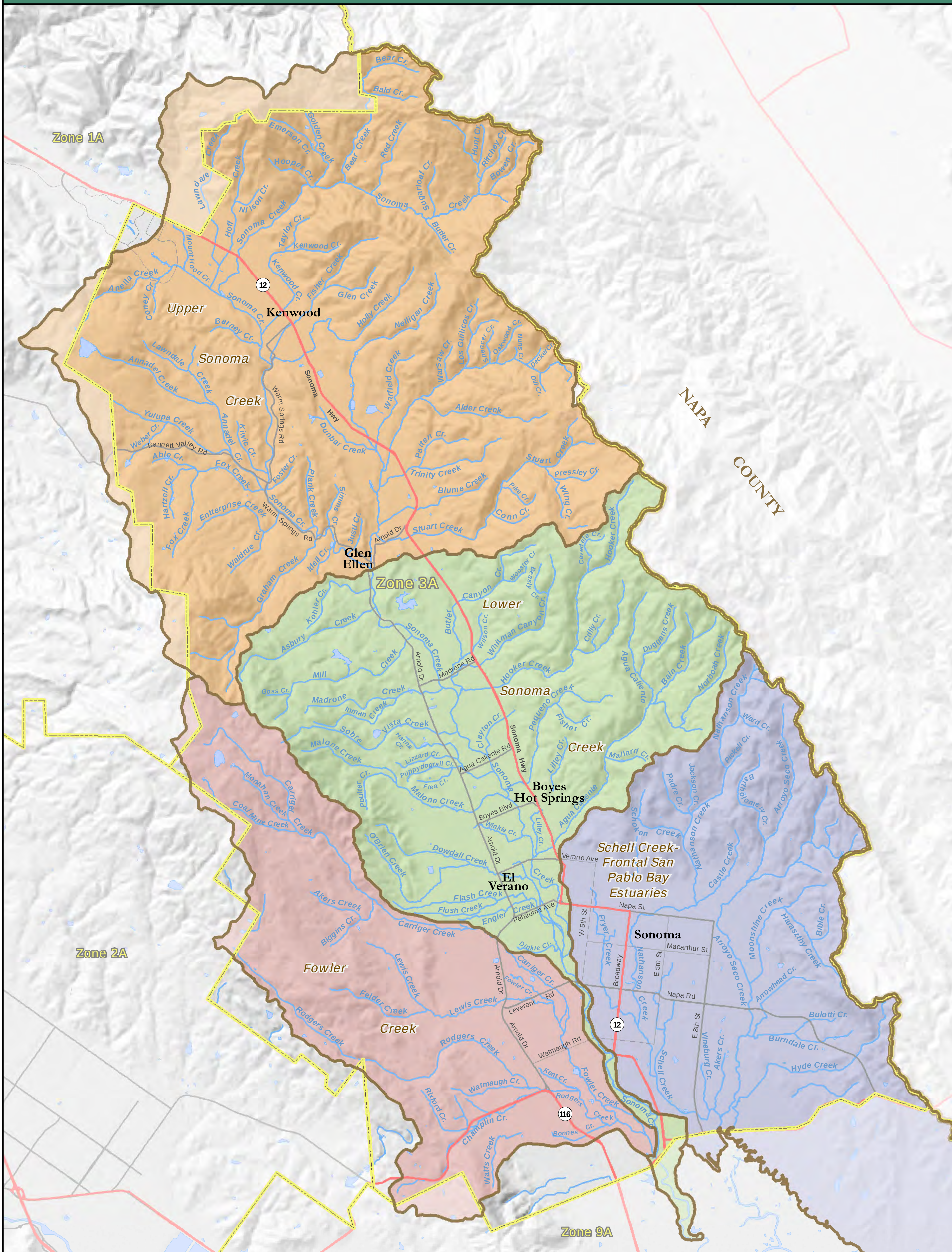
Sonoma County Water Agency

Flood Management Design Manual (FMDM)



Major Sub-basins

- Fowler Creek
- Lower Sonoma Creek
- Schell Creek-Frontal San Pablo Bay Estuaries
- Upper Sonoma Creek

- SCWA Flood Control Zone Boundary
- Water Body
- Streams and Channels
- Highways
- Roads and Major Streets

Figure 3-5
Sonoma Creek Watershed Map

0 0.25 0.5 1 1.5 2
Miles

Horizon

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3.2.7 Tidally Influenced Zones

The southern and western boundaries of Sonoma County drain to San Pablo Bay and the Pacific Ocean, respectively. Waterways draining to these water bodies are subject to tidal influence for some distance upstream, and that distance varies with both streamflow and tide levels. The design of any project being considered within a zone of tidal influence (e.g., along the lower Petaluma River, lower Sonoma Creek, lower Russian River, as well as other smaller drainages) should account for tidal dynamics. Twice-daily peaks and lows drive water levels from downstream, and those tidal flows and the sediment they carry also play a major role in determining the geometry of the channel within the tidally-influenced zone. Local data collection and analysis may be necessary to fully identify and account for the tidal influence at a site scale.

An additional consideration in and near the tidally-influenced zone is the potential for sea level rise as a result of climate change. As described in Section 1.4, Sonoma Water does not currently have design criteria or prescribe a particular methodology for incorporating consideration of sea level rise in project designs, but recommends use of current State design guidance and the best available information to address this consideration.

3.3 Design Criteria

3.3.1 Required Analysis Methods

Sonoma Water requires one of two hydrologic analysis methods for typical projects and facilities, depending on the size of the project/watershed area and the complexity of the situation:

- The Incremental Rational Method (IRM) – for projects less than 200 acres (ac) with no detention[2], or
- The Synthetic Unit Hydrograph Method (SUHM) – for all other projects.

[2] For the purpose of determining the analysis method, “detention” means the capture and temporary storage of significant stormwater volumes, well in excess of the volume capture typical to an LID treatment for localized stormwater (i.e., greater than the volume generated by 1.0 inch of rain over 24 hours).

Both methods estimate runoff based on the rainfall associated with a design event duration and frequency. The design rainfall event frequency is selected based on the project design requirements. Both methods, and the criteria for determining which method should be used, are described in detail in Section 3.4, “Hydrologic Analysis Methods.”

In a very limited number of cases, it may be possible to use stream gauge data to develop or calibrate design flow and volume estimates. Because gauge records of sufficient length are available in only a few locations, this Manual does not address this topic in detail. If gauge data are available and provide an appropriate means to support a project analysis, the Applicant should discuss the potential use of measured data directly with Sonoma Water staff.

3.3.2 Assumptions for Hydrologic Analysis

Sonoma Water requires use of ultimate buildout condition assumptions for all hydrologic calculations.

The following assumptions must be applied to currently undeveloped areas unless a publicly proposed development, precise zoning, or the *County of Sonoma General Plan 2020* (County of Sonoma 2008, amended 2016) indicates a different land use:

- **Gently to moderately sloped land** (up to 15% slope) that is undeveloped at the time of design shall be assumed to be fully developed as single-family and two-family residential zones (lots under ¼ ac in size);
- **Moderately to considerably sloped land** (general average slope of 15–20%) shall be assumed to be fully developed into residential subdivisions with lot size of ¼–½ ac; and
- **Areas of steep terrain** (general average slope greater than 20%) shall be assumed to be fully developed to an intensity of use compatible with the nature of the terrain, such as residential development in lots greater than ½ ac.

Public parks, public golf courses, and other publicly owned open space areas may be considered as vegetated to the extent that they are actually vegetated, unless publicly proposed plans show that the body having jurisdiction intends to alter the existing use of the area to make the surface less pervious.

For most submittals, Sonoma Water will require only the preparation of post-project condition analyses. However, in selected cases, Sonoma Water will require pre-project condition analyses as well and reserves the right to do so. In such a case, buildout assumptions will be specified.

Given that most LID practices are designed to address smaller, high-frequency storms rather than large, infrequent flood events, modification of hydrologic input parameters (e.g., runoff coefficients or infiltration rates as represented by **Curve Numbers**) to reflect LID implementation are not part of this Manual. That said, minimization of impervious area is an accepted LID strategy. Allowance is made in the methods prescribed in this Manual to calculate both runoff coefficients and Curve Numbers to reflect actual impervious area, rather than relying on standard percentages.

3.3.3 Hydrologic Analysis Inputs

Specific sources and calculation methods are required to identify and select input parameters and data for hydrologic analysis of **design storm** events. These equations and sources are specified in Section 3.4, “Hydrologic Analysis Methods.” Primary sources are also summarized below in Table 3-1, “Summary of Hydrologic Input Parameters and Data Sources.”

Table 3-1. Summary of Hydrologic Input Parameters and Data Sources

Input	Source	FMDM Section
Precipitation data	*Atlas 14: Precipitation-Frequency Atlas of the United States* (National Oceanic and Atmospheric Administration [NOAA] 2014)	Section 3.4.1.3, "IRM Rainfall Intensity" Section 3.4.2.2, "SUHM Design Rainfall Event"
Times of concentration, t_c, for overland flow	FMDM **Table 3-3** (or calculate)	Section 3.4.1.3, "IRM Rainfall Intensity"
Overland flow n_o (roughness)	FMDM **Table 3-4**	Section 3.4.1.3, "IRM Rainfall Intensity"
Channel *n* (roughness)	**FMDM Appendix D**, Table D.2-1	Section 3.4.1.3, "IRM Rainfall Intensity"
Pipe *n* (roughness)	FMDM Appendix D, Table D.2-4	Section 3.4.1.3, "IRM Rainfall Intensity"
Runoff coefficients, *C* (IRM)	**FMDM Appendix C**, Table C-1	Section 3.4.1.1, "IRM Runoff Coefficient (C)"
Storm duration	IRM: Based on travel time SUHM: Specified at 24 hours (hr)	Section 3.4.1.2, "Design Storm" Section 3.4.2.2, "SUHM Design Rainfall Event"
Curve numbers, *CN* (SUHM)	FMDM Appendix C, Tables C-2 through C-5	Section 3.4.2.3, "SUHM Watershed Losses – Curve Number (Infiltration)"
Peaking coefficient	Specified at 0.75	Section 3.4.2.4, "SUHM Transform Method"
Lag time (SUHM)	Equation 3.13	Section 3.4.2.4, "SUHM Transform Method"

Note: Definitions of these terms are provided in the FMDM sections where they are discussed, as well as the glossary in ***Appendix A****.*

Precipitation Data: NOAA Atlas 14

Sonoma Water has elected to rely on the NOAA *Atlas 14* (NOAA 2014) precipitation data set for a number of reasons. The NOAA *Atlas 14* data set:

- *uses extensive local gauge data;*
- *accounts for orographic effects;*
- *is peer reviewed;*
- *will be updated periodically by the federal government; and*
- *has user-friendly web support tools.*

3.3.4 Design Flows

As shown in Table 3-2, "Minimum Design Flows" below, the minimum size of a design flow for a project varies with watershed size.

Table 3-2. Minimum Design Flows

Waterway Type	Watershed Area	Design Flow
Minor waterway	1 sq mi or less	10-year peak flow
Secondary waterway	Between 1 and 4 sq mi	25-year peak flow
Major waterway	4 sq mi or more	100-year peak flow

In addition to the minimum design flow, every project submittal must also include analysis of a 100-year design flow for purposes of ensuring minimum risk from a large event (see Chapter 4, *Flow Conveyance*).

3.4 Hydrologic Analysis Methods

Project applicants may use this Manual as a reference guidebook for evaluating potential hydrologic effects of their projects. As indicated in Section 3.3.1 above, this section describes two Sonoma Water–approved methods to evaluate post-project runoff conditions for typical projects and facilities:

- The Incremental Rational Method (IRM) – for projects less than 200 ac with no detention[3], and
- The Synthetic Unit Hydrograph Method (SUHM) – for all other projects.

Both methods estimate runoff from the precipitation depth of a design rainfall event (duration and frequency). The IRM may be used to estimate peak flow for watersheds up to 200 ac in size where storage effects may be neglected. The IRM is not appropriate to analyze watersheds with existing or proposed **retention** or **detention facilities**. For larger, more complex watersheds and those with detention facilities, Sonoma Water requires use of the SUHM and recommends use of the public domain software package **HEC-HMS**. The decision tree shown in Figure 3-6, "Decision Tree for Selection of Hydrologic Method," can be used to determine whether the IRM or SUHM should be applied.

[3] For the purpose of determining the analysis method, "detention" means the capture and temporary storage of significant stormwater volumes, well in excess of the volume capture typical to an LID treatment for localized stormwater (i.e., greater than the volume generated by 1.0 inch of rain over 24 hours).

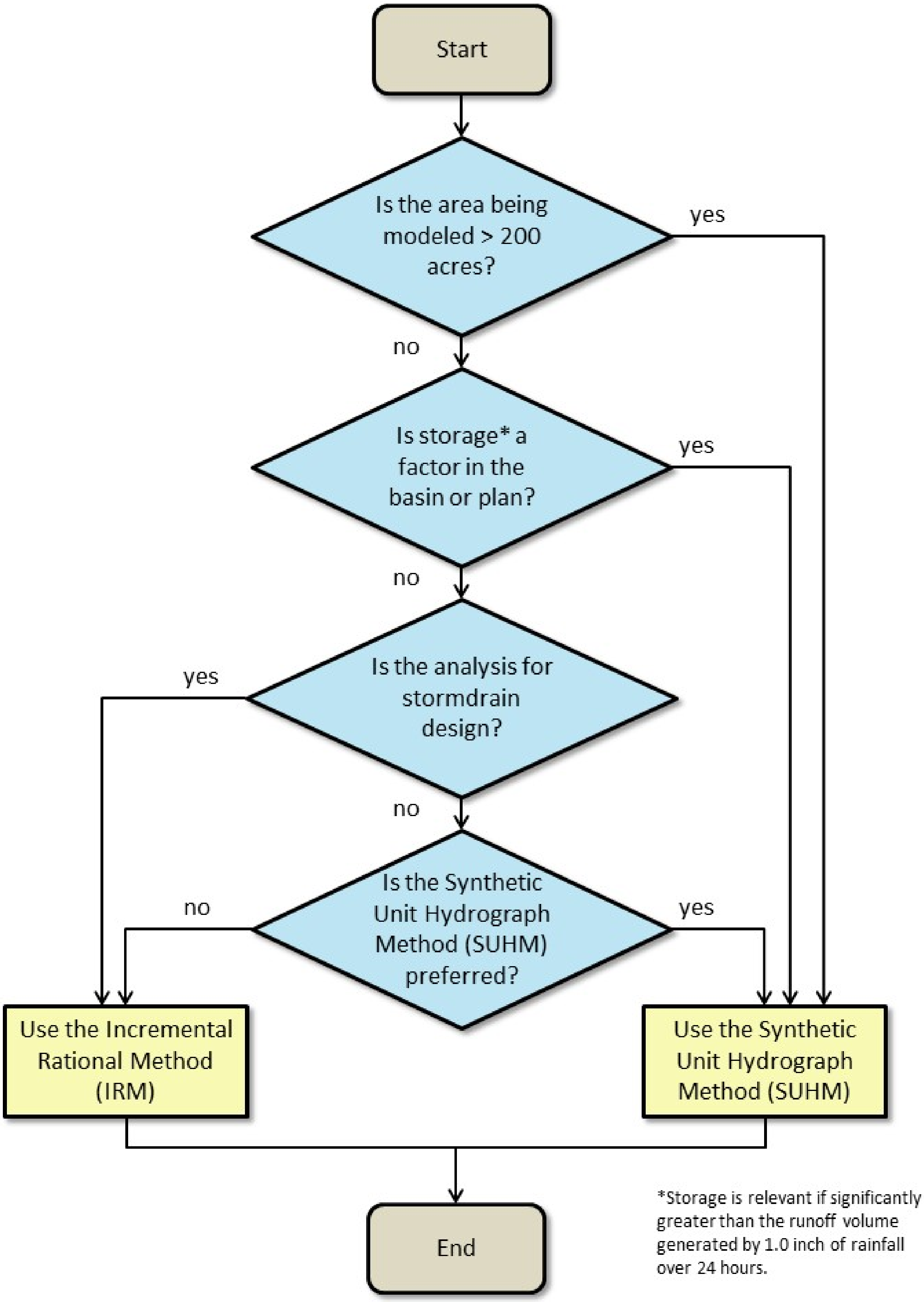

Figure 3-6. Decision Tree for Selection of Hydrologic Method

3.4.1 Incremental Rational Method

The rational formula (*Equation 3.1*) uses drainage area, a land cover runoff coefficient, and rainfall intensity to estimate the peak flow (Q_p) resulting from a design rainfall event. In use since the 19th century, the **Rational Method** is the simple application of the rational formula to characterize watershed runoff. The Incremental Rational Method[4] applies the rational formula incrementally to subwatersheds within the watershed of interest to provide a more detailed estimate of the cumulative peak flow based on the characteristics of each subwatershed. This section briefly describes each component of the rational formula and its application as the IRM.[5] Example Problem 1 in Appendix E.1 provides a sample application of the IRM.

The rational formula estimates the peak watershed runoff rate (Q_p) for a design rainfall event duration and frequency as a proportion of the rainfall intensity (I) corresponding to the same design event. Because the Rational Method uses a single coefficient (C) to estimate losses in each subbasin and assumes a uniform rainfall rate (I), Sonoma Water limits its use to watersheds of less than 200 ac. Each parameter used in the rational formula is discussed in detail following the description of the IRM.

Sonoma Water uses the rational formula as shown in *Equation 3.1*. Some forms of the rational formula include a conversion factor (k[6]) to convert acre-inches per hour to cubic feet per second (cfs); however, since the value of this conversion factor is approximately equal to one, it is often dropped from the equation, as it is in *Equation 3.1* below.

[4] Because the term "Modified Rational Method" is used by engineers in reference to many different alternative Rational Method approaches, Sonoma Water uses the term "Incremental Rational Method" to describe Sonoma Water's preferred method.

[5] The IRM analysis procedure described in this Manual does not differ significantly from the procedure described in the prior Flood Control Design Criteria Manual (FCDC Manual) (1983 edition).

[6] This "*k*" factor is entirely different from the "*k*" used in the prior FCDC Manual that was a coefficient to adjust a base 30 in of rainfall to the appropriate mean annual precipitation value for the location of interest.

$$Q_p = C * I * A \qquad (Equation\ 3.1)$$

Where,

Q_p is the peak flow rate for the design rainfall event (cfs);
C is the runoff coefficient (dimensionless);
I is the rainfall intensity (inches/hr) for the design rainfall event; and
A is the watershed or subwatershed area (ac).

For small, homogeneous watershed areas, the rational formula can be applied directly. Larger, heterogeneous watersheds should be subdivided into relatively homogeneous subwatersheds that are then evaluated using the IRM. When comparing hydrologic change among scenarios (e.g., comparing pre-project and post-project conditions), subwatershed boundaries should be consistent among scenarios, to the extent possible.

To apply the IRM, the rational formula is applied to each subwatershed to calculate a cumulative watershed Q_p. An overview of the nine steps of the IRM is provided in the call-out box below. In this description, letters (A, B, C, etc.) are used to designate subwatersheds and numbers (1, 2, 3, etc.) are used to designate computation points, as indicated in Figure 3-7, "Schematic for Application of the Incremental Rational Method." Example Problem 1 in Appendix E.1 provides a detailed example of the analysis process.

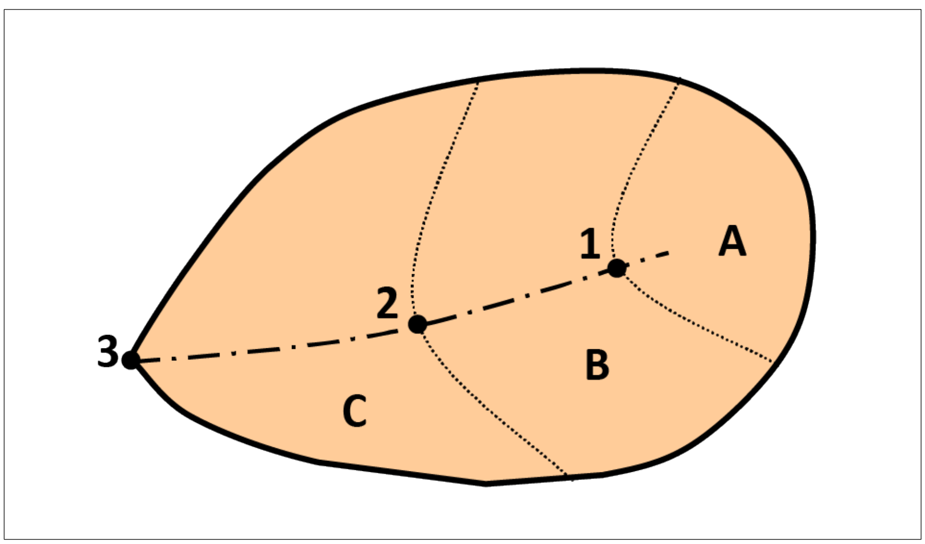

Figure 3-7. Schematic for Application of the Incremental Rational Method

This cumulative computation can be applied sequentially to any number of subwatersheds. Where tributaries enter, the tributary watershed should be added as an additional subwatershed. The travel time from the farthest point in the basin or subbasin through the drainage network to the outlet is termed the **time of concentration**, t_c (see Section 3.4.1.4).

At the confluence point, two times of concentration (t_c) will be calculated – the cumulative t_c for the main watershed and the flow travel time for the tributary subwatershed. The time of concentration that results in a larger Q_p in subsequent steps is used to carry the analysis forward.

Incremental Rational Method

The steps required to implement the Rational Method in the form of the Incremental Rational Method used by Sonoma Water are briefly presented below. In this description, letters (*A, B, C*, etc.) are used to designate subbasins and numbers (*1, 2, 3…n*) are used to designate computation points, as indicated in Figure 3-2. A detailed example of the analysis process is provided in Appendix E.1, Example Problem 1.

Step 1. Delineate the drainage area of interest.

Step 2. Delineate the flow path(s) from the uppermost point in the drainage area to the downstream-most computation point (n).

Step 3. Subdivide the drainage area into subbasins with relatively homogeneous slopes and anticipated flow conveyance types. Measure the subbasin areas (AA, AB, AC) and flow lengths.

Step 4. Identify the time of concentration or tc for the most upstream contributing area, usually an overland flow source. Sonoma Water specifies fixed values for the initial time of concentration (identified in Section 3.4.1.4, “Time of Concentration,” on page 3-32). This establishes the duration (tA) used to select the applicable rainfall intensity, IA, for this most upstream computation point.

Step 5. Select an appropriate C value for area A (CA) and calculate the peak flow for computation point 1 (Q1) using Manning’s Equation (Q1 = CA*IA*AA)

Step 6. Use an appropriate reference (e.g., equation, nomograph, or table) to estimate travel time through subarea B (tB) using Q1, the assumed conveyance mode, and the flow length. Then add tA + tB to estimate the time of concentration for the next computation point (tA + tB = t2). Use t2 to identify the rainfall intensity for computation point 2 (I2).

Step 7. Select an appropriate C value for area B (CB) and estimate the cumulative (area-weighted) C*A value for computation point 2: C2*A2 = (CA*AA) + (CB*AB).

Step 8. Calculate the peak flow for computation point 2 (Q2) using Eqn. 3.1 (Q2 = C2*I2*A2)

Step 9. Repeat steps 6, 7, and 8 until the final computation point (Qn) is reached. Where tributaries enter, they are added as contributing areas and whichever time of concentration controls (creates a larger *Q*) in subsequent steps is used to carry the analysis forward.

3.4.1.1 IRM Runoff Coefficient (C)

The function of the runoff coefficient (*C*) in the rational formula is to adjust the rainfall intensity (*I*) to reflect only excess rainfall that contributes to runoff. *C* therefore represents the proportion of rainfall that is not retained in the watershed due to infiltration and other mechanisms. This proportion is a function of watershed characteristics that include land cover, soil infiltration characteristics, and ground slope. *C* values used by Sonoma Water in applying the IRM are included in Appendix C, Table C-1. Guidance is provided below for determining the appropriate soil group and slope to use when selecting *C* values from Table C-1.

For watershed areas with multiple land uses and soil types, or when applying the IRM to multiple subwatersheds, a composite C value must be calculated using *Equation 3.2* below. Example 3-1 demonstrates how to calculate a composite runoff coefficient.

$$C_w = \frac{\sum_{j=1}^{n} C_j A_j}{\sum_{j=1}^{n} A_j} \qquad \text{(Equation 3.2)}$$

Where,

C_w is the weighted runoff coefficient;
C_j is the runoff coefficient for area j;
A_j is the acreage of area j; and
n is the number of areas with different land use or soil types.

The method described above may also be employed if the Applicant wishes to calculate a C value for a given area instead of relying on the values provided in Appendix C, Table C-1. The C value for impervious area should be assumed to be 0.90 for all slopes and soil types, while the C value for the pervious area may be taken from the “Ag and open space” entry for the appropriate slope and soil type as shown in Appendix C, Table C-1.

Example 3-1. Calculating a Composite Runoff Coefficient

For a 10-year event in a watershed of 10 ac that is entirely located on *C* soils, on a 4% slope, that is 50% pasture, 25% streets and homes, and 25% forest, the composite watershed *C* value (C_w) would be calculated as follows:

Subwatershed Area 1 (Pasture):

$A_1 = 5\ ac$

$C_1 = 0.38$

Subwatershed Area 2 (Streets and homes, medium density):

$A_2 = 2.5\ ac$

$C_2 = 0.74$

Subwatershed Area 3 (Forest):

$A_3 = 2.5\ ac$

$C_3 = 0.38$

Composite Watershed *C* Value:

$$C_w = (A_1*C_1 + A_2*C_2 + A_3*C_3) / (A_1 + A_2 + A_3)$$
$$C_w = (5\ ac * 0.38 + 2.5\ ac * 0.74 + 2.5\ ac * 0.38) / (5\ ac + 2.5\ ac + 2.5\ ac)$$
$$C_w = (1.9\ ac + 1.85\ ac + 0.95\ ac) / 10\ ac$$
$$C_w = 4.7\ ac / 10\ ac$$
$$C_w = 0.47$$

Land Use

Runoff coefficients in Appendix C, Table C-1 are listed by land use category. For watersheds with more than one land use type, the procedure described in Example 3-1 above should be used to develop a composite value for *C.*

Soil Groups

The NRCS has classified soils into four hydrologic soil groups (A, B, C, and D) based on minimum infiltration rate (NRCS, 1986; NRCS 2009). These hydrologic soil groups are described below:

Group A soils have low runoff potential and high infiltration rates even when thoroughly wetted. Group A soils typically have less than 10 percent clay and more than 90 percent sand or gravel and have gravel or sand textures. They consist chiefly of deep sands and gravels that are well drained to excessively drained and have a high rate of water transmission (greater than 0.30 inch/hr).

Group B soils have moderate infiltration rates when thoroughly wetted and consist chiefly of soils that are moderately deep to deep and are moderately well drained to well drained.

Group B soils typically have 10–20 percent clay and 50–90 percent sand and have loamy sand or sandy loam textures. These soils have a moderate rate of water transmission (0.15–0.30 inch/hr).

Group C soils have low infiltration rates when thoroughly wetted and consist chiefly of soils having a layer that impedes downward movement of water and soils of moderately fine to fine textures. Group C soils typically have 20–40 percent clay and less than 50 percent sand and have loam, silt loam, sandy clay loam, clay loam, and silty clay loam textures. These soils have a slow rate of water transmission (0.05–0.15 inch/hr).

Group D soils have high runoff potential. They have very low infiltration rates when thoroughly wetted and consist chiefly of clay soils with high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious material. Group D soils typically have more than 40 percent clay and less than 50 percent sand and have clayey textures. These soils have a very slow rate of water transmission (0–0.05 inch/hr).

Figure 3-8, "Hydrologic Soil Groups," shows hydrologic soil groups for Sonoma County as mapped by NRCS. This map may be used to identify the appropriate hydrologic soil group(s) for Sonoma County projects. For watersheds with more than one soil type, use the procedure described in Example 3-1 to develop a composite value for *C.* Soils data are also available online at: websoilsurvey.sc.egov.usda.gov/App/HomePage.htm.

Runoff coefficients in Appendix C, Table C-1 are categorized by the following slope categories: 0-2%, 2-6%, 6%-12% and greater than 12%. The average watershed (or subwatershed) slope should be used to select the appropriate slope category.

3.4.1.2 Design Storm

The recurrence interval of design storm events is assumed to be equivalent to the recurrence interval of design flow events, such that a 10-year design storm is assumed to produce a 10-year peak flow. Two events will be modeled for most projects: the minimum design flow as specified in Table 3-2, "Minimum Design Flows," and the 100-year flow. The duration of the design rainfall event is selected based on the travel time from the farthest point in the basin or subbasin through the drainage network to the outlet and is termed the time of concentration, t_c.

3.4.1.3 IRM Rainfall Intensity

Rainfall intensity (*I*) is a function of the design event duration and frequency, with short, low-frequency events producing the highest rainfall intensities. When selecting *I*, the design event frequency is determined by the project design requirements. The design event duration is equal to the watershed or subwatershed time of concentration (t_c). Setting the event duration equal to t_c results in the maximum peak flow Q_p, since a longer duration would have a lower rainfall intensity and a shorter duration would not produce runoff from the entire watershed.

NOAA has developed tables of rainfall intensity by event frequency and duration, based on statistical analysis of rainfall gauge data as described in NOAA's *Atlas 14 (NOAA 2014)*. Sonoma Water requires *Atlas 14* as the source for rainfall intensities used in the rational formula.

Atlas 14 precipitation data for California are available from the NOAA Precipitation Frequency Data Server (PFDS) at hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=ca. To retrieve rainfall intensity data, select: **Data type** = *Precipitation intensity*; **Units** = *English*; **Time series type** = *Partial duration* in the Data Description section. In the Select Location section, either manually enter the watershed or subwatershed latitude and longitude, or carefully select (double-click) the location on the map provided. Once the location is selected, a table will appear at the bottom of the page showing precipitation intensity by event duration and frequency. To develop values for intermediate time durations, the tabular values may be interpolated using a power curve (equation of the form $y = ax^b$) or other suitable interpolation method.

3.4.1.4 Time of Concentration

The time of concentration (t_c) is defined as the time required for runoff to travel from the most distant point in the watershed to the computation point where the flow is being estimated. It represents the time required for all areas within the watershed to contribute to runoff at the watershed outlet. Travel time is estimated based on three potential types of flow:

2. Overland flow (generally limited to 300 ft or less before tending to concentrate in a flow path),
3. Shallow concentrated flow (e.g., **gutter** or **swale**), and
4. Concentrated flow (e.g., pipe and/or channel).

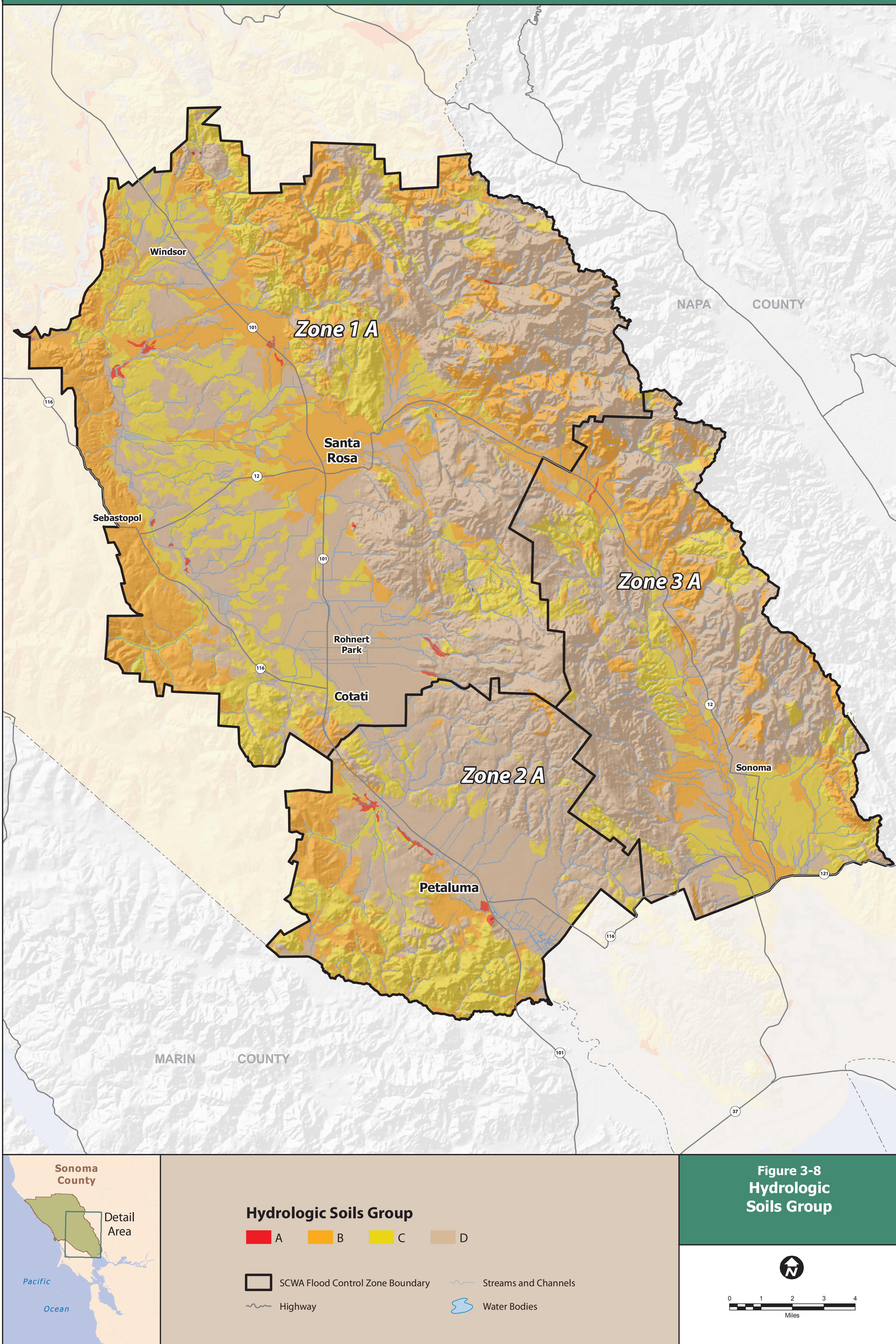

Figure 3-8
Hydrologic Soils Group

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To determine the total travel time through a watershed or subwatershed, the travel time of each type of flow is calculated based on an understanding of the primary flow path. The time of concentration is the sum of the overland flow travel time, the shallow concentrated flow travel time, and the flow travel time in a pipe and/or channel (*Equation 3.3*). These components are discussed below.

$$t_c = t_o + t_s + t_p \qquad \textit{(Equation 3.3)}$$

Where,

t_c is the watershed time of concentration (minutes [min]);
t_o is the overland flow travel time (min);
t_s is the shallow concentrated flow travel time (min); and
t_p is the pipe and/or **channel flow travel time** (min).

Overland Flow Travel Time

Time of concentration for overland flow can be calculated using a simplified method or a more detailed method based on the Kinematic Wave equation. The simple method can be applied for overland flow areas of up to 2 ac or flow paths up to 300 ft in length. Both methods are discussed below.

IRM Overland flow: simple method

Sonoma Water provides approximate overland flow travel times for watershed areas up to 2 ac or flow paths up to 300 ft in length, based on land use. The values in Table 3-3, "Estimated Overland Flow Travel Time," can be used to approximate t_c or t_o for small areas, and are typically applied to the farthest upstream subwatershed in an IRM analysis.

Table 3-3. Estimated Overland Flow Travel Time

Description	Time of concentration for overland flow	Maximum size
Commercial	0.117 hr (7 min)	2.0 ac
Lot sizes < ½ ac	0.167 hr (10 min)	2.0 ac
Lot sizes ≥ ½ ac	0.250 hr (15 min)	2.0 ac

IRM Overland flow: detailed method

Overland flow travel time can be calculated using an approximation of the Kinematic Wave equation, as shown in *Equation 3.4*. This option requires iteration to produce an estimate of t_o. An initial guess for travel time is used to select a value of I for use in the equation. If, after solving the equation, the resulting t_o differs from the assumption used to select I by more than 1 min, the evaluation should be repeated using the calculated t_o select a new value for *I*. The process is repeated until the calculated t_o is within 1 min of the travel time used to select *I*.

$$t_o = \frac{0.94 * L_o^{0.6} * n_o^{0.6}}{(C * I)^{0.4} * S_o^{0.3}}$$ (*Equation 3.4*)

Where,

- t_o is the overland flow travel time (min);
- L_o is the overland flow length (ft);
- n_o is the roughness for overland flow surface (dimensionless) (see **Table 3-4**, "Values of Roughness ($no_)$ for Overland Flow Calculation");
- S_o is the slope of overland flow (ft/ft);
- C is the runoff coefficient, ratio of runoff rate to rainfall intensity (inches/inch); and
- I is the rainfall intensity (inches/hr).

Table 3-4. Values of Roughness (n_o) for Overland Flow Calculation

Surface Description	Overland Flow Roughness (n)
Smooth surfaces (Concrete, asphalt, gravel, or bare soil)	0.014
Fallow (No residue)	0.05
Cultivated soils (Residue cover < 20%)	0.06
Cultivated soils (Residue cover ≥ 20%)	0.17
Grass (Light turf)	0.25
Grass (Dense turf)	0.35
Woods (Light underbrush)	0.40
Woods (Dense underbrush)	0.80

Source: Adapted from SCS 1986 and Los Angeles County Department of Public Works 2006

Shallow Concentrated Flow Travel Time

To estimate shallow concentrated flow travel time, use Equation 3.5 and Equation 3.6 that are provided in Table 3-5, "Velocity Estimates for Shallow Concentrated Flow."

Table 3-5. Velocity Estimates for Shallow Concentrated Flow

Surface	*Equation*[1]	
Paved	$V = 20.32 * S^{0.5}$	(*Equation 3.5*)
Unpaved	$V = 16.13 * S^{0.5}$	(*Equation 3.6*)

1 Where V is the velocity (feet per second [ft/sec]) and S is the slope (ft/ft).

Source: Based on SCS 1986.

Then use *Equation 3.7* to estimate the travel time by dividing the length of shallow concentrated flow by flow velocity to estimate travel time.

$$t_s = \frac{L_s}{V * 60} \qquad \textit{(Equation 3.7)}$$

Where,

t_s is the shallow concentrated flow travel time (min);
L_s is the length of shallow concentrated flow (ft); and
V is the velocity (ft/sec).

Pipe and/or Channel Flow Travel Time

To estimate flow travel time in a pipe and/or channel, use **Manning's equation** to first estimate flow velocity.

Manning's equation is an empirical equation relating open channel flow velocity to flow area and channel slope. Manning's equation is shown in *Equation 3.8*:

$$V = \left(\frac{1.49}{n}\right) * R_h^{\frac{2}{3}} * \sqrt{S} \qquad (\textit{Equation 3.8})$$

Where,

V is the average horizontal velocity in the **cross section** (ft/sec);
n is the Manning's roughness coefficient or "Manning's n" (dimensionless);
S is the friction slope (ft/ft); and
R_h is the hydraulic radius (ft).

Manning's n values for pipe materials are provided in Appendix D.2, Table D.2-4. Values for open channels are provided in Appendix D.2, Table D.2-1. See Section 4.2.4.1, "Manning's Equation," for a more complete description of Manning's equation and its application.

The **hydraulic radius** (R_h) is defined as the ratio of flow area to the **wetted perimeter**, or total length of the cross-section surface in contact with water. R_h can be calculated from *Equation 3.9*:

$$R_h = \frac{A}{P_w} \qquad (\textit{Equation 3.9})$$

Where,

A is the cross-sectional area (square feet [ft^2]); and
P_w is the wetted perimeter (ft).

To calculate the travel time, use *Equation 3.10* and divide the length of pipe and/or channel flow by the flow velocity (calculated in *Equation 3.8* using Manning's equation). Repeat this analysis for each segment of pipe and/or channel with different hydraulic properties.

$$t_p = \frac{L_p}{V * 60} \qquad (\textit{Equation 3.10})$$

Where,

t_p is the pipe and/or channel flow travel time (min);
L_p is the length of pipe (ft); and
V is the velocity (ft/sec).

3.4.2 Synthetic Unit Hydrograph Method

For drainage areas greater than 200 ac, or if the drainage system includes detention storage[7] or a reservoir, Sonoma Water requires use of the SUHM to estimate flow peaks and **hydrographs**. The SUHM transforms a design rainfall event into an estimated time-varying flow rate (hydrograph). Unlike the IRM approach, which only provides peak discharge rates, the SUHM approach calculates a design runoff hydrograph for the duration of the design event. Key

[7] For the purpose of determining the analysis method, "detention" means the capture and temporary storage of significant stormwater volumes, well in excess of the volume capture typical to an LID treatment for localized stormwater (i.e., greater than the volume generated by 1.0 inch of rain over 24 hours).

parameters, including flow peak and runoff volume, can be obtained from the analysis. A detailed example of the analysis process is provided in Example Problem 2 in Appendix E.2.

The basic steps in simulating a design hydrograph using the SUHM, regardless of software tools used, are as follows:

1. Delineate the watershed and any subwatersheds needed for the analysis.
2. Identify the design rain event and develop a rainfall **hyetograph** (bar graph of rainfall distribution).
3. Estimate watershed losses.
4. Use hydrologic software to estimate rainfall excess by subtracting watershed losses from the design hyetograph and then transform rainfall excess into direct runoff, in the form of a hydrograph, using the SUHM.
5. If needed, route the direct runoff hydrograph through channels, pipes, or reservoirs.

The HEC-HMS hydrograph simulation model, developed and maintained by the U.S. Army Corps of Engineers, Hydrologic Engineering Center (HEC), should be used as the default program to conduct this analysis, though other synthetic unit **hydrograph methods** may be used (e.g., TR-55), adhering to the detailed guidance provided for HEC-HMS below to the extent possible. Most of the steps listed above are automated in HEC-HMS. Detailed procedures for using the program are readily available from the HEC website: www.hec.usace.mil. The following subsections describe general steps to conduct the SUHM analysis using HEC-HMS. Additionally, the call-out box below highlights topics involved with HEC-HMS analysis, with cross-references to relevant sections of this Manual.

Steps in developing a SUHM model with HEC-HMS, using the methods described in greater detail below, include the following:

1. Create a new project.
2. In the Basin Model, define the physical characteristics of the watershed by creating a basin model that includes each of the subbasins, routing reaches, computation points, storage areas, etc. and descriptions of their physical (e.g., subbasin area, connection to other watershed elements, etc.) and hydrologic (e.g., **lag time**, peaking coefficient, curve number, **baseflow** assumptions, etc.) characteristics.
3. In the Meteorologic Model, select a method for calculating precipitation and enter the required **depth-duration-frequency** data for a storm of appropriate duration and frequency.
4. Define the Control Specifications, which indicate the start and end dates and times, as well as the time interval for calculations.

5. Run the simulation.
6. Perform model **calibration** and **validation** to the extent possible, and iterate to achieve suitable results.

The subsections below describe the development and identification of key input parameters for the HEC-HMS model. For a demonstrated application of HEC-HMS, please see Example Problem 2 in Appendix E.2.

General Parameters Guide for HEC-HMS

The methods recommended by Sonoma Water for hydrologic analysis using HEC-HMS are outlined below. For more detailed instructions, please refer to the listed section numbers of this Manual and the Hydrologic Modeling System HEC-HMS User's Manual (HEC 2013).

Basin Model Methods:

- *Subbasin*
  - Loss Rate: Curve number (see Section 3.4.2.3)
  - Transform: Snyder's unit hydrograph (see Section 3.4.2.4)
  - Base flow: User may select preferred base flow method
- *Reach*
  - Routing: Muskingum-Cunge (see Section 3.4.2.5)
  - Gain/Loss: User default should be None; if field data or calibration results suggest the need for inclusion, the user may select Constant or Percolation method

Meteorologic Model Methods (see Section 3.4.2.2):

- *Precipitation: Frequency Storm*
- *Probability: Design storm probability (return period)*
  - Input/Output type: Partial duration
  - Intensity duration: Select 5 min as a default
  - Storm duration: 24 hr
  - Intensity position: 50%
  - Precipitation depths: Precipitation depths for the indicated durations from NOAA Atlas 14
- *Evapotranspiration: None*
- *Snowmelt: None*

3.4.2.1 SUHM Watershed Delineation

If the study area crosses one or more watershed divides, or includes detention[8], or is greater than 1 square mile, the study area should be subdivided into smaller, relatively homogeneous subwatersheds. If detention will be simulated, the watershed should be divided at minimum into subwatersheds upstream and downstream of the detention facility, whether detention is existing or proposed as part of the project. Subwatershed boundaries should be based on physical or topographic attributes or features of the project site, such as tributary areas, flow concentration points, channel junctions, roads, and culverts. When comparing hydrologic change among scenarios (e.g., comparing pre-project and post-project conditions), subwatershed boundaries should be consistent among scenarios to the extent possible.

3.4.2.2 SUHM Design Rainfall Event

HEC-HMS contains several methods for distributing rainfall depths over a time series to create a design rainfall hyetograph. Per Sonoma Water design criteria, use of a 24-hr storm duration is required. Sonoma Water recommends use of the “balanced frequency-based storm” or “frequency storm” method, which builds a nested and balanced hyetograph using rainfall depths for shorter duration storms.

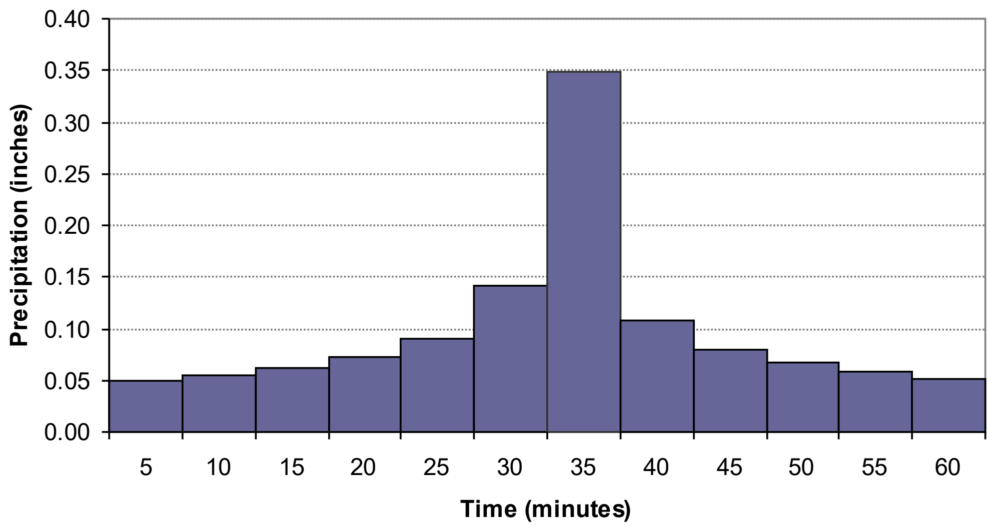
[8] For the purpose of determining the analysis method, “detention” means the capture and temporary storage of significant stormwater volumes, well in excess of the volume capture typical to an LID treatment for localized stormwater (i.e., greater than the volume generated by 1.0 inch of rain over 24 hours).

Hyetograph: Frequency Storm Method

Sonoma Water recommends use of the Frequency Storm method to identify the hyetograph for HEC-HMS modeling because it is constructed so as to include the appropriate rainfall intensities for shorter duration events and therefore can be used to appropriately assess runoff from all subbasins, regardless of size.

Using this approach and an intensity duration of 5 min for a one-hr storm, the 5-min intensity (in the example below, 0.350 inch) occurs at the central 5 min of the hyetograph, the difference between the 10-min intensity and the 5-min intensity rainfall (in the example below, 0.14 inch) occurs in the 5 min before the peak intensity, while the 15-min intensity less the 10-min intensity (in the example below, 0.11 inch) occurs in the 5 min after the peak intensity, etc.

The NOAA Atlas 14 rainfall intensities are used to construct the hyetograph for HMS simulation. A demonstration of the data entry to accomplish this is shown in Example Problem 2 in Appendix E.2.



Parameters required to define a design rainfall hyetograph in HEC-HMS include the following:

Probability: The annual probability associated with the design rainfall event frequency (e.g., 10% for the 10-year design storm). Directions on selecting an appropriate design storm probability are given in Section 3.3.4, “Design Flows”

Intensity Duration: The intensity duration (time step) for the rainfall hyetograph. Five min is an appropriate default value. The intensity duration should be no longer than one-fifth of the time of concentration, t_c, of the watershed.

Storm Duration: 24 hr.

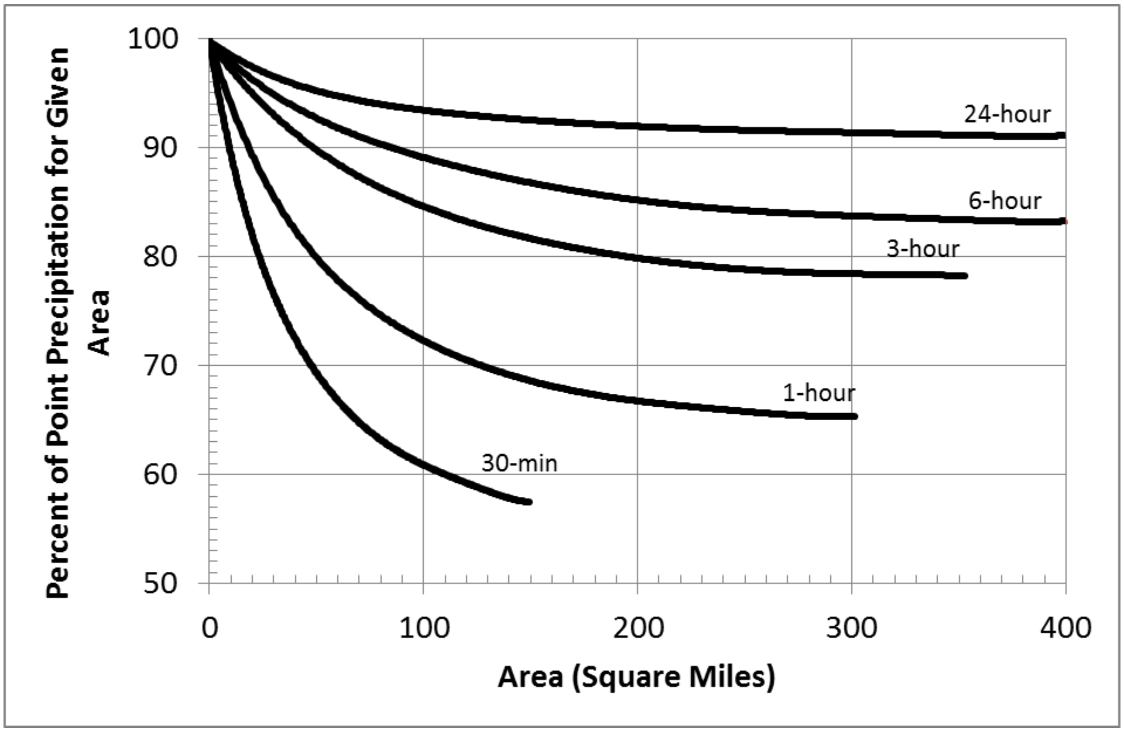
Intensity Position: Select the default at 50% (whereby the highest rainfall intensity occurs in the middle of the storm duration).

Storm Area: Enter the watershed area.

Storm Depth: The rainfall depth for each duration shown on the HEC-HMS input table. Use NOAA *Atlas 14* rainfall depth data, available from the NOAA PFDS at hdsc.nws.noaa.gov/ hdsc/pfds/pfds_map_cont.html?bkmrk=ca. Two options may be used to develop storm depth data for a given subbasin: (1) point precipitation at the centroid of the subbasin, or (2) development of an area-weighted average precipitation value for the subbasin using raster data. In either case, the precipitation data must then be adjusted by applying Areal Reduction Factors (ARFs) to translate a point precipitation value into an area precipitation value, as described in *Technical Paper No. 40 – Rainfall Frequency Atlas of the United States for Durations from 30 Minutes to 24 Hours and Return Periods from 1 to 100 Years,* referred to as TP-40 (Hershfield 1963), available at www.nws.noaa.gov/oh/hdsc/PF_documents/ TechnicalPaper_No40.pdf. ARFs are multiplied by the point precipitation values to estimate precipitation over the area of the subbasin. The ARF figure from TP-40 is reproduced below for convenience as **Figure 3-9**, "Area-Depth Relationships (Area Reduction Factors)." See Example Problem 2 in Appendix E.2 for a sample application of NOAA *Atlas 14* data to develop a hyetograph.

To retrieve rainfall depth data for a point location, select: **Data type** = *Precipitation depth*; **Units** = *English*; **Time series type** = *Partial duration* in the Data Description section. In the Select Location section, either manually enter the watershed or subwatershed latitude and longitude, or carefully select (double-click) the location on the map provided. Once the location is selected, a table will appear at the bottom of the page showing precipitation depth by event duration and frequency. To develop values for intermediate or smaller (< 30 minute) time durations, the tabular values may be interpolated or extrapolated using a linear regression of natural log-transformed values (equation of the form $y = ln(x) + b$).[9] Alternatively, raster data may be obtained by downloading precipitation data in Geographic Information System (GIS) format from the NOAA PFDS (NOAA 2017) at hdsc.nws.noaa.gov/ hdsc/pfds/pfds_gis.html.

[9] A lookup table based on this regression approach was developed to facilitate assignment of appropriate ARFs for all the curves included in Figure 3-9 as well as 5-min, 15-min, and 12-hr duration rainfall depth data. The table has been provided as Appendix C, Table C-6.



Source: Based on Technical Paper No. 40 (Hershfield 1963)

Figure 3-9. Area-Depth Relationships (Area Reduction Factors)

3.4.2.3 SUHM Watershed Losses

The NRCS Curve Number method should be used to calculate watershed rainfall losses. **Initial abstraction** and infiltration based on the curve number (*CN*) are used in this method to represent rainfall losses that do not contribute to watershed runoff.

Initial Abstraction

Initial abstraction (I_a) represents the watershed losses before runoff begins, such as rainfall that is "lost" to interception by vegetation, initial infiltration, and surface depressions. If this parameter is left blank, HEC-HMS will estimate I_a from *Equation 3.11*:

$$I_a = \frac{200}{CN} - 2 \qquad \text{(Equation 3.11)}$$

Where,

I_a is the initial abstraction (inches); and
CN is the curve number (see below).

Curve Number (Infiltration)

Curve numbers (*CN*s) are used to represent the proportion of direct runoff associated with a rainfall event as a function of land cover and soil characteristics. Appendix C, Tables C-2 through C-5 provides *CN*s by land use and hydrologic soil group.[10] A map of hydrologic soil groups is provided in Figure 3-8 above. Soils data are also available online at: websoilsurvey.sc.egov.usda.gov/App/HomePage.htm.

Composite CN

If the study area encompasses more than one soil group, each soil group is used, as shown in *Equation 3.12*, to calculate a composite *CN*, as shown in Example 3-2 in the call-out box below:

$$CN_w = \frac{\sum_{j=1}^{n} CN_j A_j}{\sum_{j=1}^{n} A_j} \qquad (\textit{Equation 3.12})$$

Where,

CN_w is the weighted curve number;
CN_j is the curve number for area *j*;
A_j is the area for land cover *j*; and
n is the number of distinct land covers.

The method described above may also be employed if the Applicant wishes to calculate a *CN* value for a given area instead of relying on the values provided in Appendix C, Table C-2. The *CN* value for impervious area should be assumed to be 0.98 for all soil types, while the *CN* value for the pervious area may be taken from the appropriate “Open Space” entry for the appropriate soil type as shown in Appendix C, Table C-2.

[10] The *CN*s provided in Appendix C are for Antecedent Runoff Condition II, which should be assumed unless calibration or validation suggests otherwise.

Example 3-2. Calculating a Composite *CN*

If a watershed comprises multiple land uses or hydrologic soil groups, a composite CN should be calculated. A composite CN is a weighted average based on the area of each land cover type. To calculate a composite CN, multiply each CN by the area of its respective land cover type. Then sum these products and divide by the total area.

For example, a 10-ac area of open space in good condition over a Group D soil has a *CN* of 80. A 2-ac area of woods in good condition over a Group C soil has a *CN* of 70. The composite *CN* is calculated as follows:

$$\frac{(80 * 10\ ac) + (70 * 2\ ac)}{12\ ac} = 78$$

Impervious Percentage

Impervious surfaces (e.g., rooftops, driveways, sidewalks, and streets) permit little or no infiltration and produce high quantities of runoff relative to the total amount of rainfall. The impervious percentage (sum of impervious surface areas divided by the total area) can help guide *CN* selection for some land use categories or can be entered directly in lieu of a composite *CN* value.

Impervious percentage is most accurately obtained directly from design drawings or aerial photographs. For larger areas, or where direct measurements are not available, impervious percentages can be estimated from Table C-2 in Appendix C.

3.4.2.4 SUHM Transform Method

The SUHM transforms a unit of excess rainfall into a flow rate. Sonoma Water recommends the use of the Snyder unit hydrograph method in HEC-HMS. Two inputs are required for transforming excess rainfall into a flow-rate time series (hydrograph) using this method: the standard lag and peaking coefficient inputs.[11]

[11] The SUHM is most accurate when variables have been calibrated for the setting in which they are applied, and this is true for the Snyder Unit Hydrograph method. While the lag equation and peaking coefficient shown here were not calibrated to Sonoma County, they are simplified approaches derived from calibration work conducted elsewhere in the region (Alameda County).

Standard Lag

The lag time is defined as the length of time between the midpoint of rainfall mass and peak flow of the resulting hydrograph. The lag time can be calculated from *Equation 3.13*:

$$t_L = 1.56 * \left(\frac{L * L_c}{\sqrt{S_o}} \right)^{0.38} \quad \text{(Equation 3.13)}$$

Where,

t_L is the lag time (hr);

L *is the length of the longest flow path (mi) (see Figure 3 10, "Watershed Lag Time Computation Components");*

L_c is the *length* (mi) along the longest flow path measured from the outlet to a point opposite (orthogonal to) the watershed centroid (see Figure 3-10); and

S_o is the average watershed slope (ft/mile), typically calculated as the difference in elevation between the end points of the principal flow path (elevation measured between the uppermost watershed divide and the watershed outlet), divided by the length of flow path.

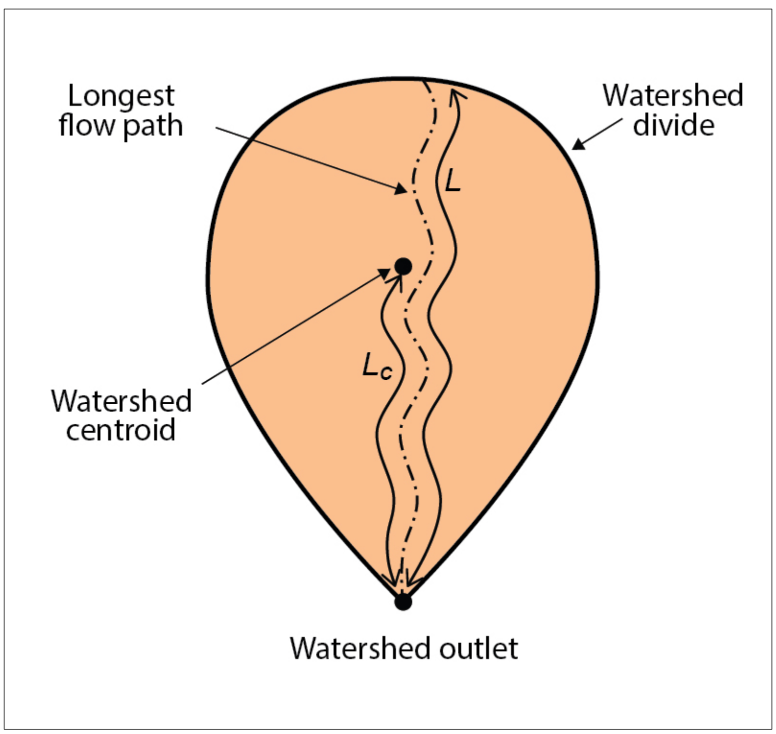


Figure 3-10. Watershed Lag Time Computation Components

Peaking Coefficient

The peaking coefficient determines the steepness of the unit hydrograph. Sonoma Water requires using a peaking coefficient of 0.75.

3.4.2.5 SUHM Hydrograph Routing

Hydrograph routing simulates the changes in a flood wave as it moves downstream. Sonoma Water recommends using the Muskingum-Cunge routing method in HEC-HMS. Manning's **roughness** values for this method can be found in Tables D.2-1 and D.2-4 in Appendix D.2.

3.4.2.6 SUHM Validation and Calibration

All SUHM modeling results must be validated for watersheds greater than 1 sq mi, as illustrated in Example Problem 2 in Appendix E.2. Validation is not required for projects with drainage areas of less than 1 sq mi.

Model validation may be completed in multiple ways; the most typical methods include comparing model results with: (1) available stream gauge data, reasonably replicating a historic flood event, ideally of similar magnitude to the event being modeled; (2) results from a flood frequency analysis of peak flow data from a nearby gauged watershed that is comparable to the study area (e.g., land cover/development, slope, mean annual precipitation), scaled to the study area basin size; (3) local planning reference documents (e.g., storm drain master plans) that provide hydrologic estimates; and (4) results obtained through use of regional regression equations. The regional regression equations developed by the U.S. Geological Survey (USGS) are appropriate to use for this purpose and may be found, together with StreamStats, an online calculator to automate their application, at streamstats.usgs.gov/ss. Use of StreamStats is demonstrated in Example Problem 2 in Appendix E.2. The regional regression equations are designed for use in rural watersheds (defined as having less than 10 percent impervious area, a parameter available on StreamStats). Methods to modify the results to reflect urban conditions are identified in the report documenting the development of the regression equations (Gotvald 2012), also available at the StreamStats website.

Appropriate stream gauge data as described in methods (1) and (2) is unlikely to be available for many streams in Sonoma County, as there are a fairly limited number of gauges in the county and many of them have only a short record. If gauge data is not available for the study area, and the local planning reference documents of method (3) are also not available, then using regional regression equations (manually or through StreamStats), as identified in method (4) above, may be the preferred option. Method (4) may be the most broadly available option for projects in Sonoma County.

If the USGS regional regression equations are used for validation, additional consultation with Sonoma Water will be required in two cases: (a) if adjustment to reflect urbanized conditions is required; and (b) if modeling results are outside of the standard error margins of the USGS regional regression equation results. In the first case, the project Applicant will consult with

Sonoma Water to review the appropriate procedure for modifying the estimation approach to reflect urban conditions. In the second case, the project Applicant is required to consult with Sonoma Water to review the appropriateness of modeling results and evaluate the potential need for additional validation checks or even model calibration to refine the model.

If the Applicant is using other validation methods, including gauge-based methods (1) or (2) above, local planning reference documents of method (3) above, or any methods other than using the regional regressions; then the Applicant must consult with Sonoma Water to discuss results that are more than 50 percent different from SUHM results for the design or 100-year flow rates. If such differences occur between the validation results and the SUHM results, then the modeling results may be subject to additional scrutiny and question at the discretion of Sonoma Water.

The validation process and results must be fully documented as part of the submitted hydrologic analysis package.

Model calibration may also be performed prior to a final validation if sufficient and appropriate data are available. Methods to calibrate hydrologic modeling results may include matching observed hydrographs by changing initial abstraction and loss values, often achieved by adjusting *CN*s. Adjustment of *CN* values may be made across the model or within specific subbasins as appropriate, and may be made as a percentage change or by applying Antecedent Runoff Condition (ARC) III *CN* values instead of the ARC II values provided in Appendix C, Tables C-2 through C-5. Changes made in the model in response to calibration should be fully documented in the submitted hydrologic analysis package.

Manual users or applicants can discuss calibration approaches, if necessary, on a case-by-case basis with Sonoma Water staff.

3.4.2.7 SUHM Hydrologic Modeling for Detention Design

While Sonoma Water has no hydrologic requirements for design of detention facilities, other agencies and jurisdictions may. Sonoma Water does require review of the hydrologic modeling for significant detention storage facilities[12]. In the interests of minimizing flood risks, Sonoma Water requires analysis of with- and without-project conditions for at least the design flow and the 100-year flow to confirm that flood risks are not aggravated by the project, including unintentional effects downstream (or upstream) of the project. At its discretion, Sonoma Water may also require analysis of additional flood conditions. The same method should be used to simulate hydrologic conditions for all analyses, and the with-and without-project models should be identical except where divergence is needed to represent the project. Both with- and without-project condition models should be provided to Sonoma Water. Lastly, Sonoma Water requires submittal documents to include the identification of the detention requirements the design is intended to address.

3.5 Submittal Requirements Summary

The items required for submittal to Sonoma Water for review of hydrologic analyses are described in Chapter 2 of this manual. The forms of the submittals presented in Appendix B and illustrated, in part, in Appendix E.1, Example Problem 1 (example of the IRM analysis), and Appendix E.2, Example Problem 2 (example of the SUHM analysis).

[12] For the purposes of this requirement, “detention” means the capture and temporary storage of significant stormwater volumes, well in excess of the volume capture typical to an LID treatment for localized stormwater (i.e., greater than the volume generated by 1.0 inch of rain over 24 hours).

Chapter 4
Flow Conveyance

4.1 Introduction

Flow conveyance systems are constructed or natural channel systems that transmit flow from one location to another. This *Flood Management Design Manual* (FMDM) focuses on conveyance systems for runoff drainage and flood management purposes. This chapter is comprised of three main sections: Open Channels (Section 4.2), Closed Conduits (Section 4.3), and Culverts (Section 4.4). Additionally, there is a brief section on Detention Facilities (Section 4.5). **Appendix E.3, Example Problem** 3, provides an example hydraulic analysis of a closed conduit design using the methods described in this chapter.

Open channels may be constructed channels or natural waterways. **Closed conduits** are closed conveyance systems located underground. Closed conduits may flow partially full, under an open channel flow condition, or may flow full under a pressure condition. **Culverts** are open-ended conduits providing flow passage under a roadway or similar feature.

All projects must be analyzed to evaluate the water surface elevations and flow paths associated with a 100-year flow event. The Sonoma County Water Agency (Sonoma Water) generally recommends that water surface elevations during this event be no higher than one foot (ft) below the finished floor elevations of buildings within or adjacent to the flow path. Applicants should be prepared to provide both maps of breakouts and the finished floor elevations of potentially-affected structures.

Sonoma Water recommends that the applicant for any project potentially affected by downstream **tailwater** conditions (starting **hydraulic grade line** [HGL]) meet with Sonoma Water and local jurisdiction early in the design process to assess potential approaches to establishing that tailwater condition and collaboratively determine the most appropriate design assumption.

It is important that all calculations be clearly and consistently presented on the same datum as the project plans. Sonoma Water requires that the applicant standardize all project calculations and design documents to use the same datum and to clearly identify that datum. Additionally, Sonoma Water recommends that the applicant reconfirm the datum of any source information relied upon in the project calculations and design.

4.2 Open Channels

This section of the FMDM provides Sonoma Water's guidance for analysis methods and design criteria for natural and constructed open channels.

4.2.1 General Considerations for Open Channels

This section introduces watershed and stream channel topics that provide a basis for the open channel design process.

4.2.1.1 Watershed Context

Natural open channels are representative of the watershed areas that drain to them. Channel designs, whether modifying existing waterways or developing new features, must therefore incorporate an understanding of the contributing watershed. The physical processes that influence stream flow and sediment transport are the primary concerns for engineers using this manual. However, other considerations beyond physical processes may influence the design of a given channel – most notably, to preserve the value of a stream corridor as a community asset and functioning ecosystem. Understanding the local policy, resource management, and community needs for a stream modification project is essential to design and implement a successful project.

In Sonoma County, many stream channels emerge from steep **headwater** areas of the Mayacamas Mountains or other coastal ranges and descend onto alluvial plains and valleys below. In the transition from mountain and canyon headwater areas to more gently sloping alluvial plains, channel slope may decrease abruptly. When this occurs, flow velocities also typically decrease and, as a result, coarse or large-grained sediments that may have been transported under steeper conditions upstream may now be deposited within the channel in the alluvial fan or plain setting. Historically, prior to channel modifications, alluvial channels aggraded over time, depositing their sediments overbank and thereby creating the surrounding floodplain areas. Channels passing through alluvial fans are particularly dynamic; they may change course abruptly during flood events as sediment is deposited.

In Sonoma County (and much of coastal California), channels traversing alluvial fans and plains have often down-cut (or incised) as a result of increased runoff and land use changes in the watershed upstream. As channels incise, additional stream bank failure and lateral erosion (bank retreat) may occur. Sediment eroded due to channel incision may be carried downstream and deposited within the channel; or carried even farther into downstream receiving waters such as San Francisco Bay, the Laguna de Santa Rosa, or the Russian River. Understanding a channel site’s position within the overall range of slope and sediment transport processes occurring in the watershed is essential to designing a channel with appropriate maintenance considerations.

4.2.1.2 Channel Corridor

A natural stream or flood control channel exists within a wider landscape corridor that both influences the channel and is affected by it. Stream corridors generally provide flood management functions, and if designed and managed appropriately can also offer potential community amenities and habitat benefits. In some jurisdictions, a regulated corridor is defined to manage activities adjacent to the channel. Whether the channel corridor is precisely defined and specifically regulated or not, it is valuable to consider any open channel as it functions within its corridor setting.

Evaluating the channel as a feature within a broader corridor is useful when evaluating flow and sediment hydraulics during a range of flood events. For example, a highly sinuous channel that is inset within a wider channel corridor might provide improved habitat conditions under low-flow conditions. When large flow events do occur at such a channel, the larger flows can occupy the wider cross-section of the entire channel (or channel corridor), which is less sinuous than the inset low-flow channel and provides greater conveyance capacity. The landscape corridor influenced by a stream channel extends beyond the low-flow channel bed or the top of its banks. A channel corridor that includes at least the 10-year inundation area (and as much as the 100-year area) should be considered during the channel design process when evaluating options such as public access and revegetation approaches.

In November 2014, Sonoma County (County) adopted zoning code changes through an ordinance, rezoning properties adjacent to waterways to include a riparian corridor setback. The purpose of the setback ordinance is to designate areas adjacent to streams and waterways as undevelopable conservation areas with the goal of creating a buffer to protect and improve water resources and water quality. Streams are designated according to Sonoma County's *General Plan 2020* (2008, amended 2016) and informed by the County's open space mapping project (prepared in 2008). Depending on the stream designation, riparian corridor setback zones may extend 50 ft, 100 ft, or 200 ft from the **top of bank** with everything in the setback boundary classified as streamside conversation area. The streamside conservation area boundary also includes the outer drip line of riparian trees where the trunks area located wholly or partially within the designated setback boundary.

Land uses and activities are limited within the streamside conservation area (setback zone). Certain activities are allowed in the streamside zone pending County review and approval, and if authorized by state and federal resource agencies. Activities that may occur within streamside setback zones include: stream maintenance and restoration carried out or overseen by Sonoma Water, **levee** maintenance, invasive plant removal, maintenance of dams and stream-related water storage systems, road and utility line crossings, fencing of existing outdoor activity areas (e.g., yards, gardens, or landscaped vegetation), approved agricultural activities, selective vegetation removal as part of an integrated pest management program, water supply wells, approved fire fuel management, pedestrian trails; seasonal floating docks, approved timber harvesting, hazardous tree removal, approved mining operations, or other activities approved under a conservation plan (Sonoma County Code, Ord. No. 6089, § I(d), Exhibit A).

4.2.1.3 Channel Stability and Sediment Processes

Sediment erosion and deposition are natural processes that occur within stream channels and are not necessarily a sign of instability. In a stable channel, such adjustments occur within a dynamic equilibrium, whereby the channel form and key parameters such as overall slope and cross-sectional area are relatively stable and do not progressively shift toward a new channel form over time. A stable channel should be the goal of any open channel design project.

When a channel's sediment transport capacity is not adequate to convey the sediment load, the channel may undergo progressive sediment deposition and bed aggradation. Progressive aggradation can result in increased maintenance costs, clogging of bridges and culverts, and reduction in flood conveyance capacity. Another form of instability is progressive channel erosion, which occurs when hydraulic forces exceed the resistance strength of the channel bed and/or banks. Erosion of the channel bed is referred to as channel incision. Stream incision can result in undercutting and destabilization of adjacent stream banks, damage to infrastructure, and degradation of riparian and aquatic habitat. Other mechanisms for stream bank failure (in addition to bed incision) include mass movement through rotational slumping and direct bank shearing.

The design for a modified or constructed channel must therefore account for sediment transport as well as flood flow conveyance to ensure satisfactory engineering performance. For example, straightening a sinuous channel can increase sediment transport capacity and cause progressive erosion; widening a stable channel may reduce sediment transport capacity such that sediment drops out of the stream flow, causing progressive aggradation and reducing flood flow conveyance capacity. Table 4-1, "Factors Influencing Sediment Transport," summarizes some of the factors that influence sediment transport, including channel form, discharge, and sediment characteristics. **Appendix D.1** of this FMDM provides a brief introduction to sediment transport assessment methodologies that may be appropriate to apply in the design of open channel systems.

Table 4-1. Factors Influencing Sediment Transport

Factor	**Relevant Characteristics**
Flow properties	Magnitude, frequency, and variability of stream discharge; magnitude and distribution of velocity and shear stress; degree of turbulence
Sediment composition	Sediment size, shape, gradation (sorting), cohesion, and stratification
Climate and hydrology	Rainfall amount, intensity, and duration; frequency and duration of freezing
Channel geometry	Width and depth of channel; height and angle of bank; bend curvature
Biology	Vegetation type, density, and root character
Anthropogenic factors	Urbanization, flood control, boating, irrigation

Source: Fischenich 2001

4.2.1.4 Habitat and Beneficial Uses

Stream channels provide important aquatic habitats, directly support adjacent riparian and terrestrial habitats, and serve as key migration corridors for fish and wildlife. The design of any proposed stream channel modification should seek to enhance and protect existing instream and riparian habitats and beneficial uses. Project designers should consult with the relevant regulatory agencies for additional guidance and policies related to stream channel design considerations with regard to habitat and beneficial uses.

4.2.1.5 Monitoring and Maintenance Needs

All conveyance systems must be maintained to preserve the design flow capacity. Monitoring and maintenance requirements, including access, monitoring methods, and regulatory requirements, should be considered in the channel design process. The party (or parties) responsible for maintenance and the associated documentation (e.g., irrevocable license, maintenance agreement) must be identified for any project. Responsible parties may include Sonoma Water (for Sonoma Water facilities), other jurisdictions (including cities and Sonoma County for facilities within their rights-of-way), or private parties.

4.2.1.6 Design Process

The successful design of open channels requires consideration of many factors, and an iterative process may be required to meet all design requirements. Some of the most fundamental considerations in designing an open channel include:

- drainage design criteria for hydrology, hydraulics, and sediment transport;
- physical opportunities and constraints at the project site on a watershed scale;
- land use designations and policies;
- protection of natural resources and regulations;
- engineering feasibility;
- economic constraints; and
- project goals.

While every design project will vary according to site-specific conditions and needs, the open channel design process will commonly include the following steps:

1. Identify channel alignment and pattern.
2. Evaluate bed material characteristics and sediment transport setting.
3. Identify the existing and target design longitudinal profile: (a) identify the equilibrium channel slope; and (b) consider the need for grade control.
4. Evaluate the need for sediment management, access, and trapping facilities.
5. Determine the characteristics of the channel and floodplain (or overbank flow zone) cross-section. Factors to consider include the overall project goals; dominant, design,

and 100-year discharges; HGL constraints; habitat conditions and needs; maintenance requirements; maintenance access; and public access opportunities.

6. Evaluate and design for channel stability, including bed and bank protection, if necessary.
7. Develop an appropriate monitoring and maintenance program, including identification of the parties responsible for funding and implementation.

Useful References for Open Channel Design:

Darby, S., and A. Simon, eds. 1999. Incised River Channels: Processes, Forms, Engineering, and Management. John Wiley & Sons.

Federal Interagency Stream Restoration Working Group (United States). 2001. Stream Corridor Restoration: Principles, Processes, and Practices. Available at: www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/stelprdb1044574.pdf.

Fischenich, C. 2002. Design of Low-flow Channels. ERDC TN-EMRRP-SR-19. USACE Environmental Laboratory, ERDC. Available at: hdl.handle.net/11681/3956

Fischenich, C., and J. V. Morrow, Jr. 2000. Reconnection of Floodplains with Incised Channels. ERDC TN-EMRRP-SR-09. USACE Environmental Laboratory, ERDC. Available at: hdl.handle.net/11681/3988

Shields, F. D. 1996. Chapter 2, "Hydraulic and Hydrologic Stability," In: A. Brookes and F. D. Shields, eds., River Channel Restoration: Guiding Principles for Sustainable Projects. John Wiley & Sons.

U.S. Army Corps of Engineers. 1994. Engineering and Design – Hydraulic Design of Flood Control Channels. EM 1110-2-1601. July 1, 1991, revised June 30, 1994. Available at: www.publications.usace.army.mil/Portals/76/Publications/EngineerManuals/EM_1110-2-1601.pdf.

4.2.2 General Design Guidance and Objectives

The following general design guidance and objectives should be applied to the design of any open channels to ensure that the open channel is as self-sustaining as possible, requires low maintenance, and does not cause excessive erosion or sedimentation either on-site or at upstream or downstream reaches. All channel designs will be reviewed by Sonoma Water for compliance with this Manual and potential approval or revision. Relevant reference documents are shown in the box above.

4.2.2.1 Channel Geometry and Roughness

The channel should be designed with a specific channel cross-sectional area that has a target size and capacity. The channel cross-section should be designed to meet conveyance requirements for different channel types per the guidance in this Manual (see more information in Section 4.2.3.1, “Design Discharge,” below). The channel design should consider allowable roughness conditions and how channel roughness changes over time with vegetation growth. The channel design should consider what maintenance activities will be required, and at what frequency, to maintain the desired channel capacity.

4.2.2.2 Channel Alignment

Constructed channels should be aligned to minimize channel degradation, erosion, or other undesirable effects. Except where bank stability, property boundary restraints, or environmental factors dictate an alternative course, constructed channels should preserve the existing channel alignment. If changes to the channel alignment are necessary, upstream and downstream transitions should be designed to match the existing channel cross-section, slope, and alignment to the extent practicable to minimize hydraulic transition effects. Abrupt transitions, including sharp channel bends (as viewed in planform), may trigger excessive erosion and should be avoided.

4.2.2.3 Stable Streambanks

Open channels should be designed to have stable streambanks. The streambank design must consider the sheer stress affecting the streambank, the sheer strength inherent to the bank materials, and what would constitute a stable streambank geometry in terms of bank height, width, and slope. These considerations to ensure a stable streambank design must also comply with the conveyance and design discharge requirements as described above. To achieve a stable streambank, measures may be required including grade control, bank revetment, rock toe slope protection, and placement of rock to redirect flows (e.g., vanes or spur dykes) away from the bank, or channel/bank regrading. If conditions are appropriate, the stable streambank design may consider including bio-technical techniques which favor vegetation and natural materials to achieve a stable streambank. To the extent feasible and practicable, the stable streambank should be designed to provide public safety, require little or low long-term maintenance, and conserve and promote aquatic and streamside habitat, including riparian vegetation where possible. The design should also consider regulatory requirements, sediment transport (see Appendix D.1), adjacent land uses, and community values.

Useful References for Streambank Design and Stabilization:

California Department of Fish and Game. 2010. *California Salmonid Stream Restoration Habitat Manual.* 4th Edition. Available at: nrm.dfg.ca.gov/FileHandler.ashx? DocumentID= 22610&inline.

California Department of Transportation. 2000. *California Bank and Shore Rock Slope Protection Design: Practitioner's Guide and Field Evaluations of Riprap Methods*. Final Report No. FHWA-CA-TL-95-10, Caltrans Study No. F90TL03. 3rd Edition. Prepared in Cooperation with the U.S. Department of Transportation. Available at: www.fs.fed.us/biology/ nsaec/fishxing/fplibrary/Racin_2000_California_bank_and_shore_rock_ slope_protection.pdf.

Fischenich, Craig. 2001. *Stability Thresholds for Stream Restoration Materials*. EMRRP Technical Notes Collection (ERDC TNEMRRP-SR-29), U.S. Army Engineer Research and Development Center, Vicksburg, MS. Available at: hdl.handle.net/11681/3947.

Gray, John, and Donald H. McCullah. 2005. Environmentally Sensitive Channel and Bank-Protection Measures. No. 544. Transportation Research Board 2005. Available at: www.trb.org/news/blurb_Detail.asp?ID=5617.

Lagasse, P. F., et al., 2009. Bridge Scour and Stream Instability Countermeasures: Experience, Selection and Design Guidance. Volumes 1 & 2. No. FHWA-NHI-09-111 & FHWA-NHI-09-112, respectively. Hydraulic Engineering Circular No. 23 (HEC-23). Available at: www.fhwa.dot.gov/engineering/hydraulics/library_arc.cfm?pub_number=23&id=142.

Lagasse, Peter Frederick, et al., 2016. Evaluation and Assessment of Environmentally Sensitive Stream Bank Protection Measures. No. Project 24-39. Available at: www.trb.org/ Main/Blurbs/174448.aspx.

Washington State Aquatic Habitat Guidelines Program. 2003. *Integrated Streambank Protection Guidelines*. Available at: wdfw.wa.gov/publications/00046.

4.2.2.4 Erosion and Sedimentation

Constructed waterways should be designed to be dynamically stable and not cause progressive erosion or sedimentation in the project reach, or upstream or downstream of the project. To the extent reasonably possible, stream junctions and culvert outlets into stream channels should be configured to minimize erosion risk through alignments that minimize the attack angle of the incoming flows on the receiving channel. See also Section 4.4.2.7, “Culvert Alignment,” for further discussion.

Sonoma Water does not have specific design criteria for erosion protection. In general, erosion protection measures should be designed to minimize adverse impacts to habitat, provide habitat enhancement where possible, and limit aesthetic impacts. References for erosion protection design approaches are provided in the call-out box above.

4.2.3 Design Criteria for Open Channels

This section describes the design criteria used by Sonoma Water in evaluating open channel flow design analysis.

4.2.3.1 Design Discharge

Flood conveyance systems in Sonoma County should be designed to convey the discharges shown in Table 4-2, "Minimum Design Flows for Conveyance Facilities."

Table 4-2. Minimum Design Flows for Conveyance Facilities

Waterway Type	Area	Design Flows
Minor waterway	1 sq mi or less	10-year peak flow
Secondary waterway	Between 1 and 4 sq mi	25-year peak flow
Major waterway	4 sq mi or more	100-year peak flow

Notes: *In all cases, the 100-year flow condition must also be analyzed to assess flood hazards.*
This table is consistent with the minimum design flows introduced in Table 3-2.

4.2.3.2 Channel Velocity

The minimum velocity design requirement for all open channels is 2.5 feet per second (ft/sec) for a 2-year storm. A minimum velocity threshold is established to help convey the sediment load through the system and prevent an abundance of instream deposition. The dominant sediment-transporting flow is typically associated with flow rates approximated by the 2-year event. For the purposes of assessing compliance with this minimum velocity criterion, it is permissible to use an estimated value based on U.S. Geological Survey (USGS) regional regression equations (Gotvald et al. 2012). For the North Coast Region, where Sonoma County is located, the 2-year peak flow may be estimated using *Equation 4.1*:

$$\text{2-year peak flow} = 1.82 * A^{0.904} * MAP^{0.983} \qquad (\textit{Equation 4.1})$$

Where,

A is the drainage area (sq mi); and
MAP is mean annual precipitation (inches [in]). [13]

[13] See Figure D.2-1 in Appendix D.2 to identify the MAP value.

Sonoma Water does not specify maximum velocity design criteria.[14]

4.2.3.3 Flow Regime (Froude Number)

Open channels should be designed for **subcritical flow**. **Supercritical flow** in open channels requires explicit approval from Sonoma Water. To prevent unstable oscillating flow, **Froude numbers**[15] between 0.8 and 1.2 should be avoided.

4.2.3.4 Freeboard

Freeboard is the vertical distance between the design-flow water surface elevation and the top of the channel bank (or levee). A minimum freeboard requirement is included in open channel design criteria to reduce the risk of overtopping due to hydrologic and hydraulic uncertainties, sediment deposition, increased friction caused by vegetation or bed forms, wave action, wind setup, survey inaccuracies, and other uncertainties or unpredictable events. Minimum freeboard requirements are summarized in Table 4-3, "Minimum Freeboard Requirements."

Table 4-3. Minimum Freeboard Requirements

Channel Type	Minimum Freeboard at Design Flow
Constructed open channel	1.5 ft from top of bank
Bridge or utility crossing	1.0 ft from low chord
Roadside ditch or V-ditch	0.2 ft from road shoulder or adjacent ground

Freeboard for Constructed Open Channels

Constructed open channels must be designed so that a minimum of 1.5 ft of freeboard is maintained between the top of bank and the design water surface. Where this requirement does not allow for adequate receiving of flows draining to the channel, the design water surface should be lowered to allow adequate gravity drainage to the channel. Channels shall be designed with proper allowances for hydraulic losses for all existing, planned, or projected crossings to maintain the required freeboard.

Upon request, Sonoma Water may reduce freeboard requirements for small natural swales and creeks through open space areas such as parks and golf courses.

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[14] As stated in Section 4.2.2, channel should be designed to avoid excessive erosion.

[15] The Froude number is proportional to the square root of the ratio of the inertial forces over the weight of fluid. The Froude number is dimensionless.

Additional freeboard may be required for leveed channels, channels with a slope between 0.7 and 1.3 times the critical slope, or where superelevation at channel bends is a concern (i.e., where waters may rise on the outer bend of a curve).

Freeboard for Bridges and Utility Crossings

Bridges and utility crossings that span major and secondary open channels and are existing, planned, or projected at the time of channel design must be designed to provide a minimum clearance of 1.0 ft between the low chord and the 100-year water surface and must preserve the required minimum freeboard in the upstream channel.

In evaluating minimum clearance and freeboard requirements, design water surface profiles must account for 2 ft of debris accumulation on either side of any piers for the full depth of the design flow. Alternatively, pier designs may include debris fins on the upstream side that follow a 2:1 slope down to the channel **invert**, in which case debris accumulation can be assumed to occur in only 25% of the submerged pier height, with a minimum debris height assumption of 2 ft.

See the following reference for design guidance on debris structures:

- Federal Highway Administration (FHWA). 2005. *Hydraulic Engineering Circular No. 9: Debris Control Structures – Evaluation and Countermeasures.* Available at: www.fhwa.dot.gov/engineering/hydraulics/pubs/04016/hec09.pdf.

Buried utility crossings must be below the anticipated stream scour depth. The stream scour depth is assumed to be 3 ft below the stream **thalweg** (low point) unless site-specific analysis shows that a lesser depth is appropriate.

Freeboard for Roadside Ditches and V-ditches

Roadside ditches and V-ditches must be designed so that the water surface for the design discharge will be at least 0.2 ft below the outside edge of the road shoulder, to ensure that no floodwater enters the normal travel-way of the road or below the adjacent ground level.

4.2.3.5 Creek Outfalls

Creek outfalls or outlets should be designed to be stable, non-erosive on-site, and to not generate erosion downstream or upstream. Where a pipe is used for a creek outfall, the requirements of Section 4.4.2.10, “Outlet Protection,” for culvert design apply. Outlet pipes should be mitered to meet the channel slope to provide a continuous channel grade or slope. The minimum size for an outlet pipe into a channel should be at least 15 inches and at least the last 20 ft must be corrugated metal pipe (CMP).

4.2.3.6 Levees and Floodwalls

Sonoma Water discourages the use of levees and floodwalls. Specific exceptions may be granted in situations where a levee or floodwall is the only feasible method for providing adequate flood protection.

4.2.3.7 Easements and Rights-of-Way

A revocable license is necessary for any work within any **easements** or property Sonoma Water manages or owns. An easement or a consent agreement is required for any permanent proposed structure or feature, including creek outlets, installed within or on property Sonoma Water manages or owns.

Regardless of the easement holder, a formal agreement between the easement holder and the project applicant that defines maintenance responsibilities associated with that structure or feature is also required.

4.2.3.8 Required Analysis Methods

Sonoma Water requires the use of either Manning's equation or the River Analysis System (RAS) computer program developed by the U.S. Army Corps of Engineers, Hydrologic Engineering Center (HEC), known as **HEC-RAS**, for hydraulic analysis of open channels, as described below in Section 4.2.4, "Analysis Methods for Open Channels," unless an exception is explicitly approved by Sonoma Water. Assumptions for channel roughness values required for the analysis are discussed in Section 4.2.4.1, "Manning's Equation," and specified in Appendix D.2, Table D.2-1.

4.2.4 Analysis Methods for Open Channels

This section provides guidance for hydraulic analysis methods to support open channel design.

4.2.4.1 Manning's Equation

Manning's equation is an empirical equation relating open channel flow velocity to flow area and channel slope. It may be used for analyzing simple design applications that do not warrant development of a hydraulic model.

The formula for Manning's equation is shown in the previously introduced *Equation 3.8*:

$$V = \left(\frac{1.49}{n}\right) * R_h^{\frac{2}{3}} * \sqrt{S}$$

(*Equation 3.8*)

Where,

V is the average horizontal velocity in the cross-section (ft/sec);
R_h is the hydraulic radius (ft);
n is the Manning's roughness coefficient or "Manning's n" (dimensionless); and
S is the friction slope (ft/ft).

Manning's *n* is an empirical coefficient that depends on a suite of factors, including surface roughness, vegetation, geomorphic irregularities, channel alignment, obstructions, stage (water elevation) and discharge, temperature, and sediment load. Manning's *n* values have been empirically estimated for many channel types and conditions. Values for use in hydraulic calculations are prescribed in the following section, "*Manning's n (Roughness)*."

The hydraulic radius (R_h) is defined as the ratio of flow area to the wetted perimeter, or the total length of the cross-section surface in contact with water. R_h can be calculated from *Equation 3.9*, provided in Chapter 3, Section 3.4.1.4, "Time of Concentration," and reprinted below:

$$R_h = \frac{A}{P_w}$$

(*Equation 3.9*)

Where,

A is the cross-sectional area (square feet [ft^2]); and
P_w is the wetted perimeter (ft).

Channel discharge can be calculated by multiplying channel velocity by the flow area, as shown in *Equation 4.2*:

$$Q = A * V$$

(*Equation 4.2*)

Where,

Q is the flow or discharge (cubic feet per second [cfs]);
A is the cross-sectional area (ft2); and
V is the flow velocity (ft/sec).

Manning's n *(Roughness)*

Channel roughness is typically characterized in hydraulic calculations through selection of an appropriate Manning's *n* value. In most applications, selection of Manning's *n*, or roughness, will significantly affect the computed water surface profile. When evaluating the appropriate roughness value to represent either a proposed design condition or an existing condition, the analysis must also consider the change that may occur in that roughness value over time. For example, vegetation can grow over time, leading to a higher roughness condition, and sediment and channel bed conditions can also change. The Applicant should consider what types and frequency of maintenance will occur and how that maintenance may affect the trajectory of change in the roughness value. Channel roughness conditions may vary by water depth depending on the variety of materials along the channel bed and bank. Also, some vegetation types behave differently under different flow depths.

Recommended References for Roughness:

Arcement, G. J., and V. R. Schneider. 1989. *Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains.* U.S. Department of the Interior, Geological Survey, 67 pp. Available at: www.fhwa.dot.gov/bridge/wsp2339.pdf or pubs.usgs.gov/wsp/2339/report.pdf.

Barnes, H. H. 1967. *Roughness Characteristics of Natural Channels.* U.S. Government Printing Office, 213 pp. Available at: pubs.usgs.gov/wsp/wsp_1849/pdf/wsp_1849_h.pdf.

Chow, V. T. 1959. O*pen Channel Hydraulics.* McGraw-Hill, New York.

Manning's roughness values for engineered channel materials are given in Appendix D.2, Table D.2-1. For other types (including natural stream channels), the Applicant can select a value for Manning's *n* using an appropriate reference, such as those provided in Table D.2-1.

4.2.4.2 Hydraulic Modeling

The U.S. Army Corps of Engineers' (USACE's) HEC-RAS program is a one-dimensional hydraulic modeling software package that can perform steady- or unsteady-state flow simulations for subcritical, critical, or mixed flow regimes. It can also be used to model sediment transport processes. HEC-RAS is required for all open channel hydraulic analyses prepared for submittal to Sonoma Water unless an exception is otherwise approved by Sonoma Water.

HEC-RAS can be used to estimate flow characteristics and physical behaviors of open channels, including stage, discharge, velocity, and sediment transport. Model output can be used to document compliance with design criteria for velocity, flow regime, and freeboard requirements, as described in Section 4.2.3, "Design Criteria for Open Channels."

One-dimensional modeling is best suited for streams with well-defined open channels that have predominantly longitudinal flow. One-dimensional hydraulic modeling is appropriate in most cases; however, depending on project conditions, Sonoma Water may require two-dimensional analyses on a case-by-case basis. The hydraulic model should be run in steady-state mode for flow regimes that vary gradually, are not dominated by significant changes in storage, and have little to no reverse flow that could affect water surface elevation profiles. Steady-state modeling is appropriate in most cases. However, depending on project conditions, Sonoma Water may require unsteady-state analyses on a case-by-case basis.

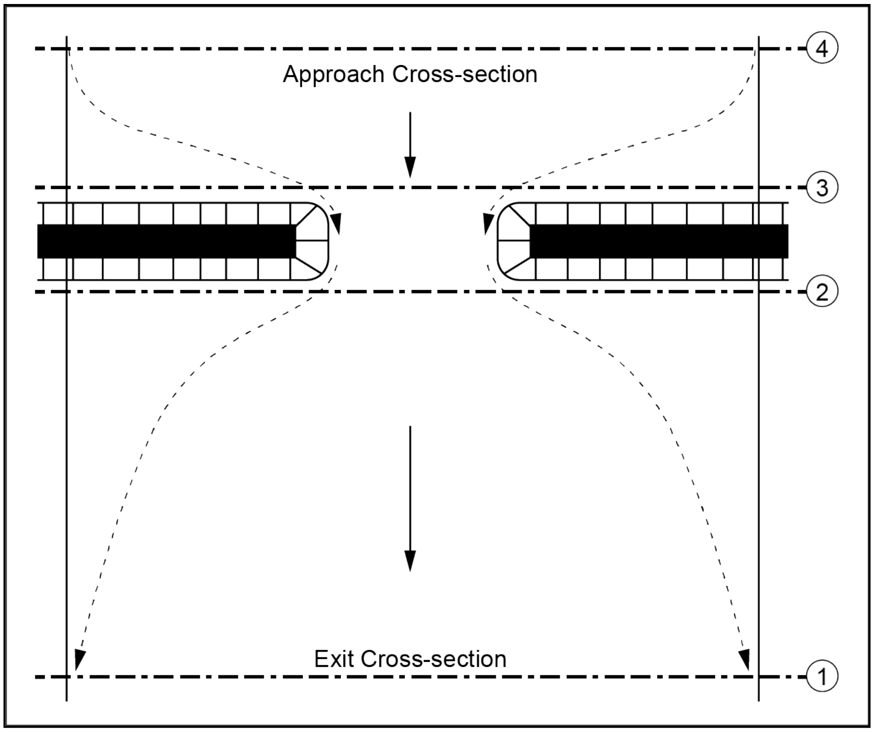
The following sections provide guidance for developing model inputs for:

- Channel geometry,
- Topographic datums,
- Loss coefficients,
- Blocked and ineffective flow areas,
- Boundary conditions, and
- Model calibration.

Channel Geometry

Channel geometry data are entered in HEC-RAS in the form of channel cross-sections. Cross-sections can be obtained from surveys conducted using conventional survey methods and equipment, or from remotely sensed topographic data such as light detection and ranging (LiDAR) technology or photogrammetry. Cross-sections must be placed perpendicular to the floodflow direction and extend far enough laterally to contain the largest flow being simulated. Cross-sections should be obtained at intervals sufficient to characterize the flow-carrying capacity of the channel and its floodplain, major breaks in the streambed profile, points of minimum and maximum cross-sectional area, and abrupt changes in channel roughness and/or shape.

Four cross-sections are recommended to define the contraction and expansion in flow induced by a hydraulic structure. Figure 4-1, “Example Cross-Section Locations at a Bridge,” demonstrates the recommended four cross-sections at a bridge. These include a cross-section upstream of the effects of flow contraction (shown as item 4 in Figure 4-1), a cross-section just upstream of the structure (item 3), a cross-section just downstream of the structure (item 2), and a cross-section downstream of the effects of flow expansion (item 1).



Source: Adapted from Bonner and Brunner 1996

Figure 4-1. Example Cross-Section Locations at a Bridge

Channel cross-sections at the upstream and downstream faces of a structure will have areas of ineffective flow (where flow is not passed toward the downstream direction), including areas on either side of a bridge or culvert opening, depending on the size of the opening and the distance between the structure and the cross-section. Blocked and ineffective flow areas are discussed in more detail below.

Channel geometry data for existing hydraulic structures can be collected using conventional survey methods or from as-built plans, if available. Dimensions should be measured for all features of the structure that may contribute to flow disruption, including piers, hand rails, blocked culverts, and bridge abutments.

Topographic Datums

It is imperative to identify the vertical and horizontal datums for all topographic data and to verify that datums are consistent among data sets. If datums vary between data sets, correction factors should be applied before the data are incorporated into the final model. The National

Geodetic Survey has developed VERTCON,[16] a publicly available (no-cost) tool to convert between the **National Geodetic Vertical Datum of 1929** (NGVD 29) and the **North American Vertical Datum of 1988** (NAVD 88). USACE provides a free tool called Corpscon[17] that enables users to convert horizontal coordinates between Geographic, State Plane, and Universal Transverse Mercator (UTM) systems, as well as between the U.S. National Grid systems on the **North American Datum of 1927** (NAD 27), the **North American Datum of 1983** (NAD 83), and High Accuracy Reference Networks (HARNs).

Loss Coefficients

Loss coefficients are used to characterize energy losses due to friction and turbulence in a channel reach or at a hydraulic structure. Contraction and expansion coefficients are used to account for energy losses derived from changes in channel shape that generate velocity gradients.

A contraction in flow area will cause velocity to increase in the downstream direction, while a sudden expansion will cause flow velocity to decrease in the downstream direction. These velocity imbalances lead to energy dissipation in the channel. An expansion in channel area typically leads to a larger energy loss than does a contraction. Typical loss coefficient values for gradual contraction and expansion in subcritical flow for a steady-state flow model are 0.1 and 0.3, respectively, at cross-sections. Typical loss coefficient values for less gradual contraction and expansion conditions, such as those that occur at bridges and other hydraulic structures, are 0.3 and 0.5, respectively (HEC 2016). Abrupt contractions or expansions may require higher loss coefficients. Contraction and expansion coefficients are generally lower for supercritical flow.

Reach losses due to friction are characterized by assigning Manning's *n* values at each cross-section. Further discussion of selecting Manning's *n* values is provided in Section 4.2.4.1, "Manning's Equation." Additional loss coefficients, such as culvert roughness coefficients, pier coefficients, and pressure and **weir** flow coefficients for high flows, may be necessary to characterize energy dissipation at hydraulic structure crossings. The *HEC-RAS Reference Manual* (HEC 2016) provides detailed instructions for selecting loss coefficients and the recommended modeling approach for various structure configurations.

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[16] The National Geodetic Survey's VERTCON is available at: www.ngs.noaa.gov/TOOLS/Vertcon/vertcon.html.

[17] USACE's Corpscon is available at: www.agc.army.mil/Missions/Corpscon.

Blocked and Ineffective Flow Areas

Areas within a cross-section that do not contribute to downstream flow should be modeled as ineffective flow areas to account for those areas where floodwaters are impounded. HEC-RAS allows for designating portions of a cross-section as ineffective flow areas to accommodate such areas. Similarly, regions within a cross-section can be modeled as blocked areas when the reach contains obstructions such as buildings or other impassable structures that provide no flow conveyance. Elevations for ineffective flow areas and blocked areas should reflect the height of these structures and allow for downstream flow conveyance if they are overtopped.

Boundary Conditions

Stage (water elevation) or discharge must be specified at the downstream and/or upstream limit of the model domain as a starting point for computation ("boundary condition"). A steady-state flow, step-backwater, subcritical, one-dimensional model will require only a downstream boundary. If the system being modeled is expected to contain a mixed flow regime of subcritical and supercritical flows, then boundary conditions must be specified for both the upstream and downstream limits of the model domain.

For steady-state flow modeling, a discharge value must be specified at the upstream end of all reaches and at subsequent flow change locations within the model domain. Unsteady-state simulations will require a flow time series (hydrograph). For further discussion of hydrologic analysis and how to estimate the design flow, refer to Chapter 3, *Hydrology,* of this manual.

Model Calibration

Where data are available, Sonoma Water may require calibration of a hydraulic model used for analysis of the 100-year peak flow. The model is calibrated using historically observed precipitation and stream discharge events. The rainfall amounts from a historic precipitation event (or multiple events) are modeled and the discharge resulting from the model simulation is compared to actual observed high water marks, recorded gauged flow or stage data, or previously calculated rating curve information. Energy loss coefficients or other appropriate model parameters can be adjusted to better reproduce the recorded data for the precipitation event and its resulting discharge. While modeling parameters may be adjusted to develop a better calibration between simulated and observed discharge conditions, adjusted input parameters should not exceed the range of typical or expected values for such parameters in similar channel and floodplain conditions. In other words, model input parameters must remain within reasonable limits following the calibration process.

In general, the most reliable calibration data are measured river stage and surveyed high water marks. Flow records, which are typically computed using recorded stage data and a rating curve, may also be used for model calibration.

4.2.4.3 Sediment Transport Analysis

Depending on project conditions, Sonoma Water may require specific channel stability or sediment transport analyses on a case-by-case basis. However, Sonoma Water has no standard requirement for such analyses. The Applicant is directed to Appendix D.1 for a review of relevant sediment transport analysis methods.

4.3 Closed Conduits

Closed conduit systems are enclosed systems, such as piped storm drains, that collect and convey runoff to receiving channels. Because **curb** and **gutter** are typically used to route flow to piped storm drains, curb and gutter design is also addressed in this section.

4.3.1 General Considerations for Closed Conduits

Specialized computer software is often used for modeling complex storm drain systems. Several commonly used software packages are noted in this section. Sonoma Water does not require the use of any specific design software. Manual calculation methods may be used for simple design applications, and guidance is provided below for manually calculating head losses associated with typical pipe configurations.

4.3.2 Design Criteria for Closed Conduits

4.3.2.1 Design Flows

Closed conduit systems should be sized using the minimum design flows shown in Table 4-2 on page 4-9. Refer to Chapter 3 for information on the calculation of runoff and design flows. For secondary and minor waterways that are placed in a closed conduit, sufficient additional surface routes for floodflows shall be made available to safely carry the 100-year discharge. If surface routes cannot be made available that will safely convey the 100-year discharge, consultation with Sonoma Water will be required to determine an appropriate strategy given the potential for debris blockage and the risks associated with floodflows.

4.3.2.2 Minimum Pipe Size

The minimum pipe size is 18 inches for open-ended **inlets**. For other inlets, the pipe should be sized to carry the necessary flow. Pipe sizes should not decrease in the downstream direction.

4.3.2.3 Minimum Velocity and Slope

Design velocities (velocity at the design flow) in closed conduits should be at least 2.5 ft/sec to prevent excess siltation. A minimum velocity threshold is established to help convey the sediment load through the system and prevent an abundance of instream deposition.

Sonoma Water does not specify maximum velocity design criteria.[18]

Appendix D.2 provides the minimum slope necessary to maintain this velocity for a range of roughness values at 80% depth (HGL = 0.8 x pipe diameter) in Table D.2-2, and at full pipe flow (HGL ≥ pipe diameter) in Table D.2-3. Figure D.2-3 in Appendix D.2 can also assist in pipe selection to meet this criterion.

4.3.2.4 Hydraulic and Energy Grade Line

Major and Secondary Waterways

If a **major** or secondary **waterway** is placed within a closed conduit, the HGL for the design flow must have a minimum of 1 ft of clearance below the top of the conduit. For circular pipes, the HGL depth must be no greater than 0.80 of the pipe diameter (Figure 4-2, "Circular Closed Conduit HGL Limitations for Major and Secondary Waterways") if downstream receiving waters allow. Figure D.2-3 in Appendix D.2 provides a useful tool for culvert selection to meet this criterion. Figure 4-2 also shows the **energy grade line** (EGL).

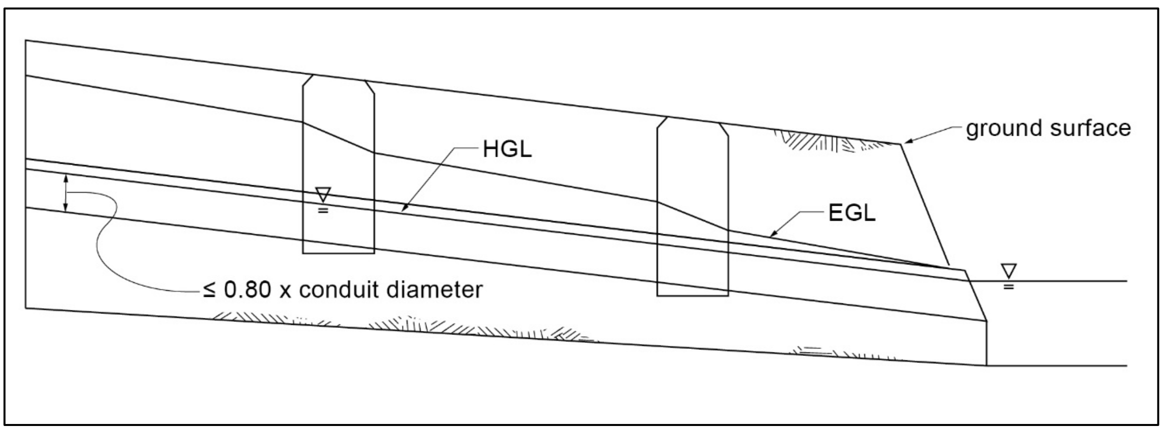


Figure 4-2. Circular Closed Conduit HGL Limitations for Major and Secondary Waterways

If the EGL is above ground level, bolted covers must be used for any manholes or access ports where that condition exists.

[18] If Sonoma Water deems a closed conduit discharging to an open channel has excessive velocity, additional erosion stabilization measures or larger pipe sizes may be required. See Section 4.4.2.10, "Outlet Protection," for more information.

Minor Waterways

Closed conduit systems for **minor waterways** must be designed so that the HGL at the design flow is not less than 1 ft below the gutter or inlet surface elevation, as shown in Figure 4-3, **"**Closed Conduit HGL Limitations for Minor Waterways." If 1 ft of clearance is not attainable using reasonable design approaches, Sonoma Water may consider allowing less freeboard on a case-by-case basis. In no case will a design be approved for which the HGL is at or above the **gutter flow line**.

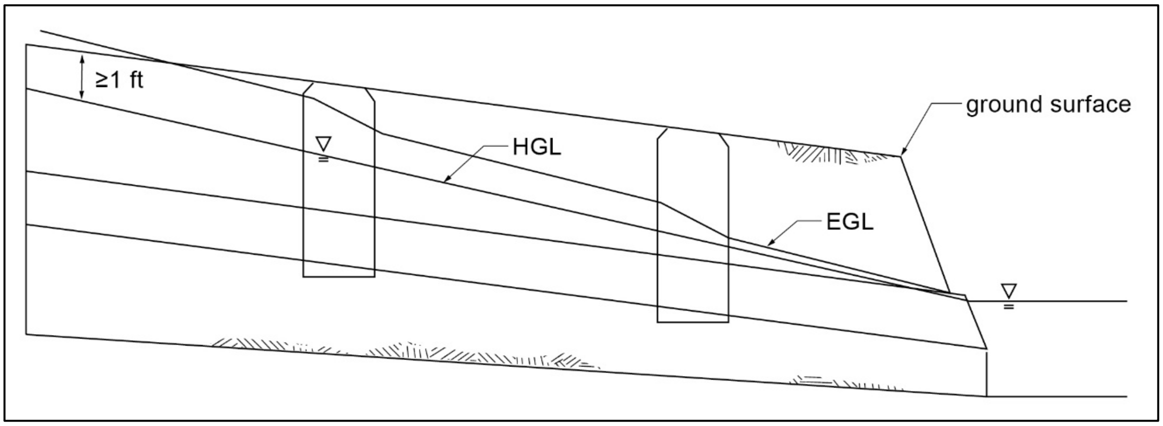


Figure 4-3. Closed Conduit HGL Limitations for Minor Waterways

If the EGL is above ground level, bolted covers must be used for any manholes or access ports where that condition exists.

If a conduit is designed for supercritical flow, the EGL must not be above ground level at inlets and non-pressure–type manholes. EGL requirements for supercritical flow are shown in Figure 4-4, "Closed Conduit EGL Limitations for Supercritical Flow."

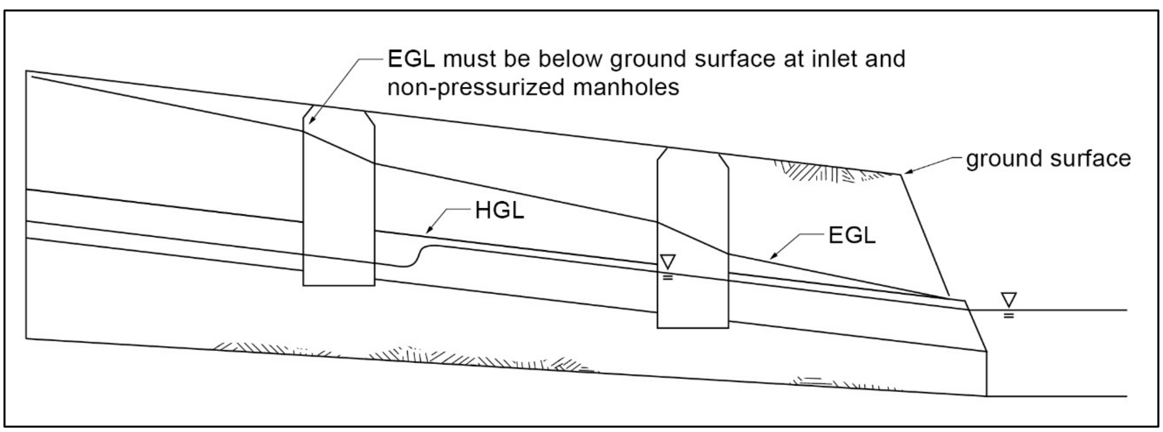


Figure 4-4. Closed Conduit EGL Limitations for Supercritical Flow

If a conduit is stubbed out for future extension, the design HGL must be low enough to allow proper drainage of the tributary area, at least 1.0 ft below existing ground level.

4.3.2.5 Tailwater

Hydraulic conditions downstream of a closed conduit must be evaluated to determine the tailwater depth for a range of discharges. At times, it may be necessary to calculate backwater curves to establish the tailwater depth conditions. The Applicant should consider the following conditions:

(1) If the outlet is operating with a **free outfall**, the Applicant should determine the **critical depth** and equivalent HGL. Graphs of critical depths for circular pipe are provided in Appendix D.2 as Figure D.2-3. *Equation 4.3* can be iteratively applied to determine critical depth in a cross-section of any shape:

$$\frac{Q^2}{g} = \frac{A_c^3}{T_c} \qquad \text{(Equation 4.3)}$$

Where,

Q is discharge (cfs);
g is standard gravity (ft/sec2);
A_c is the area for critical flow (ft^2); and
T_c is the water surface width for critical flow (ft).

(2) For conduits that discharge to an open channel, lake, pond, or other major water body, the design tailwater, or starting HGL, should be calculated based on Table 4-4, "Tailwater Elevations for Closed Conduit Design."

Table 4-4. Tailwater Elevations for Closed Conduit Design

Waterway Type	Area	Tailwater Elevation
Minor waterway	1 sq mi or less	10-year peak elevation
Secondary waterway	Between 1 and 4 sq mi	25-year peak elevation
Major waterway	4 sq mi or more	100-year peak elevation

4.3.2.6 Conduit Layout

Closed conduits should be straight between manholes where possible. If curves are necessary to conform to street layout, the minimum radius of curvature is 100 ft.

4.3.2.7 Cover and Clearance Requirements

Closed conduits should have a minimum of 2 ft of cover between the top of the conduit and the ground surface. There should be a minimum of 1 ft of vertical clearance and 5 ft of horizontal clearance from the conduit line and other utilities and conduits.

4.3.2.8 Manholes

Manholes should be placed at a maximum interval of 500 ft along conduits. Manholes or other acceptable access points should be placed at all junctions, significant reductions in slope, bends, and other points where access to the system is critical.

4.3.2.9 Conduit Materials

Conduits may be constructed out of any material appropriate to their setting and use, provided that the 20-foot section closest to the exposed end of the conduit is a material that is both heat-resistant and non-combustible, such as CMP, to avoid damage in the event of wildfire.

4.3.2.10 Outlet Protection

Outlet protection is necessary for closed conduits discharging to unlined channels. See Section 4.4.2.10, “Outlet Protection,” for outlet protection requirements.

4.3.2.11 Flap Gates

If the outlet to the closed conduit system is below the design water surface elevation of the receiving waters, a flap gate may be required to prevent flow backup in the system. In this case, coordination with Sonoma Water is necessary to determine flap gate requirements.

4.3.2.12 Gutters

Design depth of flow in gutters should not exceed 0.4 ft for the 10-year flow. To calculate the depth of flow for a uniform cross-section (see Figure 4-5, “Uniform Gutter Cross-section”), Manning’s equation can be written in terms of flow depth as shown in *Equation 4.4*.

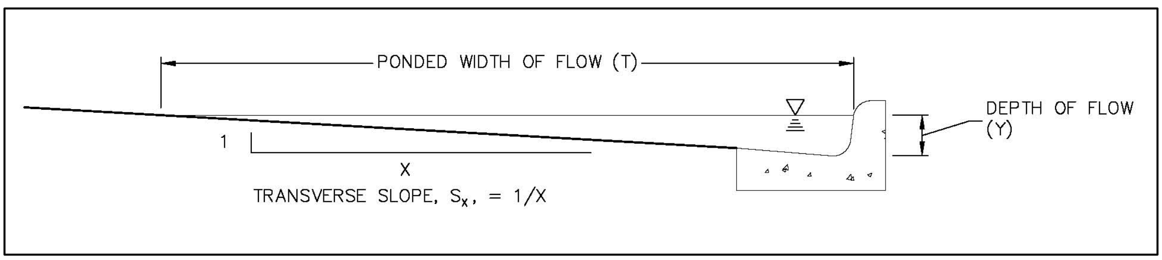


Figure 4-5. Uniform Gutter Cross-section

$$Y = 1.24\left(\frac{Q\,n\,S_x}{S_L^{1/2}}\right)^{3/8} \qquad (\textit{Equation 4.4})$$

Where,

Y is the depth of flow at the curb (ft);
Q is the discharge rate (cfs);
n is the Manning's roughness coefficient (see Appendix D.2, Table D.2-4);
S_x is the street cross-slope (ft/ft); and
S_L is the longitudinal slope (ft/ft).

4.3.2.13 Inlets

Inlets should be sized and located to limit the spread of runoff onto traffic lanes. They should be located so that concentrated flow and sheet flow in gutters will not cross traffic lanes. Sonoma Water has not established criteria for spacing of drainage inlets. However, inlets should be used at locations of sag points, superelevation reversal, street intersections, upstream from crosswalks, and upstream from curbed median openings.

4.3.2.14 Easements and Rights-of-Way

A revocable license is necessary for any work within any easements or property Sonoma Water manages or owns. An easement or a consent agreement is required for any permanent proposed structure or feature installed within or on property Sonoma Water manages or owns.

Regardless of the easement holder, a formal agreement between the easement holder and the project Applicant that defines maintenance responsibilities associated with that structure or feature is also required.

4.3.2.15 Required Analysis Methods

There is no method of analysis explicitly required to be used for closed conduits. Specific options for analysis assumptions (i.e., values, equations) are detailed in Section 4.3.3, "Analysis Options for Closed Conduits."

4.3.3 Analysis Options for Closed Conduits

In general, closed conduits are expected to be designed using computer software for ease of computation, although no specific design software package is required to be used. For simple design applications, manual calculation methods may be used. Appendix E.3, Example Problem 3 provides an example hydraulic analysis of a closed-conduit design using the methods described in this section.

4.3.3.1 Design Software

Several computer programs can be used to conduct the storm drain analysis and design process. These software programs also provide graphical user interfaces (GUIs) that greatly facilitate the analysis process. These programs include, but are not limited to, the following:

- **StormCAD –** www.bentley.com/en-US/Products/StormCAD
- **HydroCAD –** www.hydrocad.net
- **XPstorm –** innovyze.com/products/xpstorm

Other software design tools may also be used with approval of Sonoma Water.

4.3.3.2 Manual Calculation Methods

For simple design applications, manual analysis methods may be used for closed conduit analyses. The following sections provide guidance for manually calculating losses for closed conduit analyses.

Friction Losses

Energy losses occur due to friction as flow travels through a closed conduit. The difference in elevation of the EGL due to friction loss over a specified distance is the friction slope. Manning's equation for full pipe flow in a circular culvert is used to calculate the friction slope, as shown in *Equation 4.5*:

$$S_f = 4.66 * \frac{n^2 * Q^2}{D^{5.33}} \qquad (\textit{Equation 4.5})$$

Where,

S_f is the friction slope (ft/ft);
Q is the discharge rate (cfs);
n is the Manning's roughness coefficient (see Appendix D.2, Table D.2-4); and
D is the pipe diameter (ft).

For pipes flowing less than full, or for non-circular closed conduits, use the form of Manning's equation shown in *Equation 4.6* to calculate friction slope:

$$S_f = \left(\frac{Q * n}{1.49 * A * R^{2/3}} \right)^2$$ (*Equation 4.6*)

Where,

- S_f is the friction slope (ft/ft);
- Q is the discharge rate (cfs);
- A is the cross-sectional area of conduit at a given flow rate (ft^2);
- R is the hydraulic radius (ft); and
- n is the Manning's roughness coefficient (see Appendix D.2, Table D.2-4).

Once the friction slope is calculated, the head loss due to friction across a closed conduit is simply the product of friction slope and conduit length. This relationship is given in *Equation 4.7:*

$$h_f = S_f * L$$ (*Equation 4.7*)

Where,

- h_f is the friction head loss (ft);
- S_f is the friction slope (ft/ft); and
- L is the length of conduit (ft).

Minor Losses

Local head losses occur at changes in pipe section, such as at manholes, bends, valves, and fittings. These localized head losses, termed minor losses, can be significant in closed conduit performance.

Minor losses are the sum of expansion losses, contraction losses, bend losses, junction losses, losses from changes at a manhole, and outlet losses, as represented by *Equation 4.8:*

$$h_m = h_e + h_c + h_b + h_j + h_{mh} + h_o$$ (*Equation 4.8*)

Where,

h_m is the sum of minor losses (ft);
h_e is the expansion loss (ft);
h_c is the contraction loss (ft);
h_b is the bend loss (ft);
h_j is the junction loss (ft);
h_{mh} is the manhole loss (ft); and
h_o is the outlet loss (ft).

Each of these losses is explained below. The discussion begins with manhole losses, since Sonoma Water typically requires manholes at all junctions and transitions. The presentation of other loss components follows.

Manhole Losses

As flow moves through a manhole structure, head losses occur at the expansion as flow enters a manhole, at changes in flow inside the structure, and at the contraction as flow exits the manhole. Head loss across a manhole can be estimated by summing the head losses for expansion head loss as the flow discharges to the manhole structure, the contraction head loss as flow leaves the manhole structure, and an additional term that quantifies head loss due to change in flow direction, as shown in *Equation 4.9*:

$$h_{mh} = h_e + h_{fd} + h_c$$ (*Equation 4.9*)

Where,

h_e is the head loss due to expansion (ft);
h_{fd} is the head loss in the manhole due to change in direction (ft); and
h_c is the head loss due to contraction (ft).

Manhole losses due to a change in flow direction are addressed in this section. Because contraction and expansion loss terms that apply at manholes also apply elsewhere, separate sections addressing these losses follow.

Manhole Losses due to Change in Flow Direction

For losses at a change in flow direction at a manhole, as shown in Figure 4-6, "Manhole with Change in Flow Direction," *Equation 4.10* should be used.

Table 4-5, **"**Values of k_{fd} for Determining Loss of Head due to Change in Flow Direction," gives values for loss coefficients due to flow direction changes in manholes.

$$h_{fd} = k_{fd} * \frac{V^2}{2g} \qquad (\textit{Equation 4.10})$$

Where,

h_{fd} is the head loss due to change in flow direction at a manhole (ft);
k_{fd} is the loss coefficient due to change in flow direction (dimensionless);
V is the flow velocity in the conduit (ft/sec); and
g *is standard gravity (ft/sec^2).*

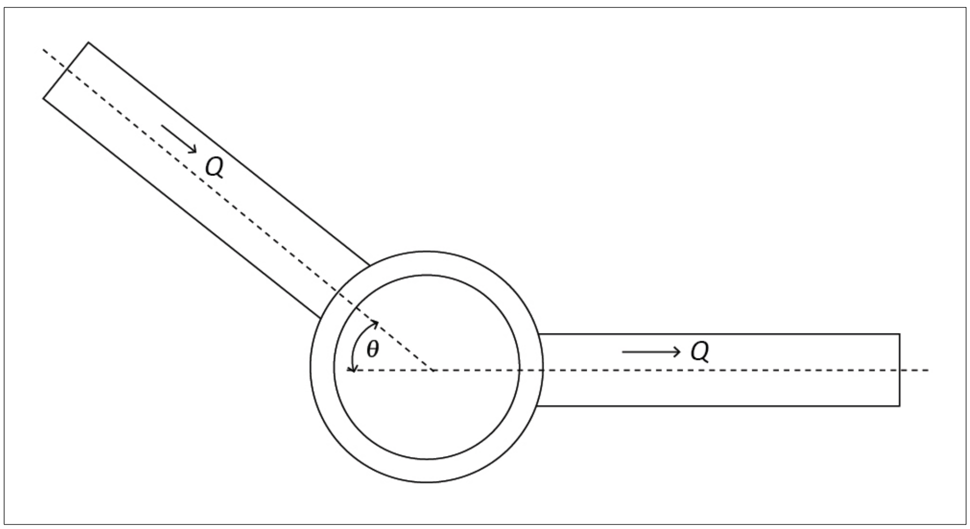


Figure 4-6. Manhole with Change in Flow Direction

Table 4-5. Values of k_{fd} for Determining Loss of Head due to Change in Flow Direction

Change in Flow Direction, θ (Degrees)	**k_{fd}**
0	0.00
15	0.19
30	0.35
45	0.47
60	0.56
75	0.64
≥ 90	0.70

Expansion Losses

Expansion losses occur during a change in the size of the conduit. In general, the Applicant should avoid transitioning between conduit sizes at locations other than manholes. *Equation 4.11* gives the relationship for calculating expansion losses.

$$h_e = k_e * \left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g} \right) \qquad \text{(Equation 4.11)}$$

Where,

h_e is the head loss due to expansion (ft);
k_e is the expansion loss coefficient (dimensionless);
V_1 is the velocity upstream of the expansion (ft/sec);
V_2 is the velocity downstream of the expansion (ft/sec); and
g *is standard gravity (ft/sec^2).*

The expansion loss coefficient (k_e) is dependent on whether the closed conduit system experiences a sudden or gradual expansion. Sudden expansion (depicted in Figure 4-7, “Sudden Expansion for Conduit Flow”) is defined as an abrupt change from one pipe diameter to another. Gradual transition (depicted in Figure 4-8, “Gradual Expansion for Conduit Flow with Angle of Cone”) reduces losses by gradually changing from one diameter to another. Expansion losses at the entrance to manholes should be estimated using the coefficient for sudden expansion.

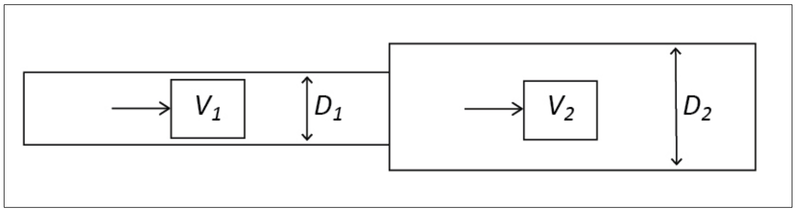
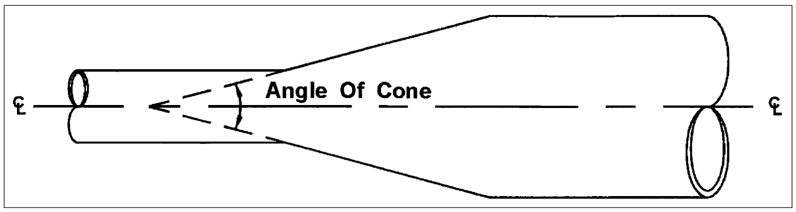


Figure 4-7. Sudden Expansion for Conduit Flow



Source: FHWA 2009

Figure 4-8. Gradual Expansion for Conduit Flow with Angle of Cone

Loss Coefficients for Sudden Expansions

Using Table D.2-5 in Appendix D.2, the Applicant can select an appropriate expansion loss coefficient (k_e) for sudden expansions. In the case of expansion losses at the entrance to manholes, the loss coefficient for the infinite expansion should be used as a conservative estimate.

Loss Coefficients for Gradual Expansions

Using Table D.2-6 in Appendix D.2, the Applicant can select an appropriate expansion loss coefficient (k_e) for gradual expansions.

Contraction Losses

Contraction losses occur in transitioning from a larger conduit to a smaller conduit (contraction) and at the exit pipe of a manhole structure. Transitioning from a larger to a smaller conduit is not recommended. However, when this is unavoidable, losses due to contraction should be calculated as shown in *Equation 4.12*:

$$h_c = k_c * \left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g} \right)$$ (*Equation 4.12*)

Where,

V_1 is the velocity upstream of the contraction (ft/sec);
V_2 is the velocity downstream of the contraction (ft/sec); and
g *is standard gravity (ft/sec^2)*.

As with expansion losses, selection of a contraction loss coefficient is dependent on whether the contraction occurs at a sudden or gradual transition in conduit diameter. Contraction losses at manholes should be estimated as a sudden contraction.

Loss Coefficients for Sudden Contractions

Loss coefficients for sudden contractions should be selected using Table D.2-7 in Appendix D.2. Loss coefficients for the exit pipe from a manhole should be estimated using the contraction loss coefficient based upon the ratio of the manhole diameter and pipe diameter.

Gradual Contractions

For gradual contractions, it has been observed that the contraction loss coefficient is equal to half the equivalent expansion loss coefficient. For gradual contractions, Sonoma Water recommends using an analogous expansion loss coefficient (k_e) from Table D.2-6 in Appendix D.2. *Equation 4.13* can be used to calculate losses due to gradual contraction:

$$h_c = 0.5k_e * \left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g} \right) \qquad \text{(Equation 4.13)}$$

Where,

h_c is the head loss due to gradual contraction (ft);
k_e is the equivalent expansion loss coefficient (dimensionless);
V_1 is the velocity upstream of the contraction (ft/sec);
V_2 is the velocity downstream of the contraction (ft/sec); and
g *is standard gravity (ft/sec^2)*.

Bend Losses

Bend losses are caused by the curvature of a conduit at locations with no manhole. Bend losses are a function of the angle of curvature and are calculated using *Equation 4.14*:

$$h_b = 0.0033\Delta \frac{V^2}{2g} \qquad \text{(Equation 4.14)}$$

Where,

h_b is the head loss due to bend (ft);
Δ is the angle of curvature (degrees);
V is the velocity (ft/sec); and
g *is standard gravity (ft/sec^2)*.

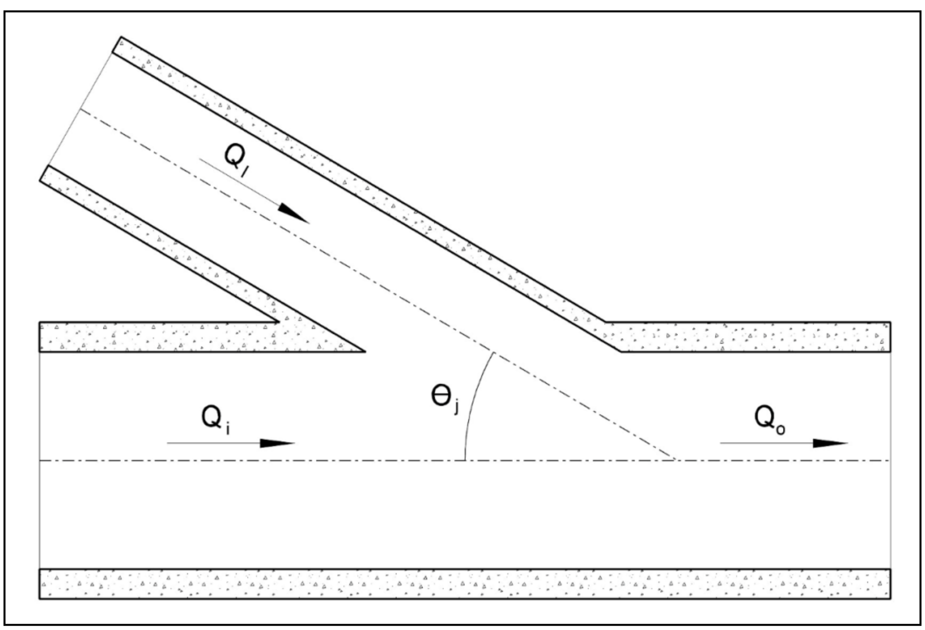
Junction Losses

Junction losses refer to losses at a conduit junction without the use of a manhole, as shown in Figure 4-9, "Conduit Junction and Interior Angle Definition." Sonoma Water typically requires manholes at all junctions; however, if a manhole is infeasible at a junction, junction losses can be calculated using *Equation 4.15*:

$$h_j = \frac{\left(Q_o * V_o\right) - \left(Q_i * V_i\right) - \left(Q_l * V_l * \cos\theta_j\right)}{0.5g*(A_1 + A_2)} + \frac{V_i^2 - V_o^2}{2g} \qquad (\textit{Equation 4.15})$$

Where,

- h_j is the head loss due to junction (ft);
- Q_o is the outlet flow (cfs);
- Q_i is the inlet flow (cfs);
- Q_l is the lateral flow (cfs);
- V_o is the outlet velocity (ft/sec);
- V_i is the inlet velocity (ft/sec);
- V_l is the lateral velocity (ft/sec);
- A_o is the outlet cross-sectional area (ft^2);
- A_i is the inlet cross-sectional area (ft^2);
- θ_j is the angle between main conduit and lateral (degrees); and
- g *is standard gravity (ft/sec^2)*.



Source: FHWA 2009

Figure 4-9. Conduit Junction and Interior Angle Definition

Outlet Losses

Outlet losses occur when a conduit discharges to a receiving channel or body of water. Outlet losses are calculated using *Equation 4.16*:

$$h_o = 1.0 * \frac{V^2}{2g} \qquad \text{(Equation 4.16)}$$

Where,

h_o is the head loss due to outlet discharge (ft);
V is the velocity (ft/sec); and
g is standard gravity (ft/sec2).

4.4 Culverts

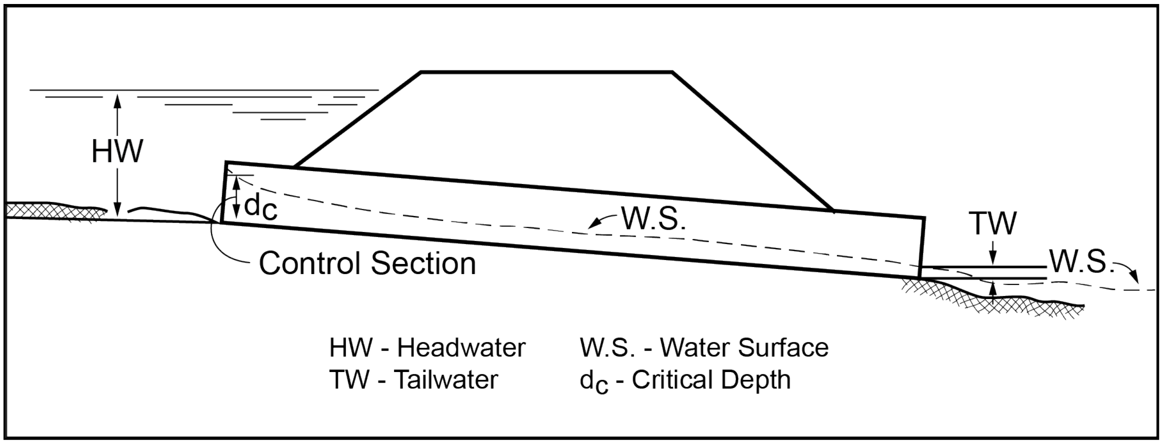
For the purposes of this FMDM, a culvert is defined as a conduit with open ends, such as might be used to provide flow passage under a roadway or similar feature. This section describes general analysis and design considerations for culverts. Other kinds of closed conduits (e.g., pipes) are addressed in Section 4.3.

4.4.1 General Considerations for Culverts

While serving as conveyance elements, culverts also act as grade and flow control structures. Culverts regulate flow through head adjustments across their inlets and outlets. Flow control in a culvert may occur at either the inlet section or the outlet section, depending on slope and other factors. When designing a culvert, calculations are made assuming both inlet and outlet control and comparing the headwater depth under each condition. The condition causing the greater headwater depth is assumed.

4.4.1.1 Inlet Control

For culverts with steeper slopes, inlet conditions such as depth of headwater and entrance geometry generally control the capacity of the culvert. A typical inlet-controlled culvert is shown in Figure 4-10, "Typical Inlet-controlled Culvert." During inlet flow control conditions, supercritical flow usually develops in the culvert barrel.

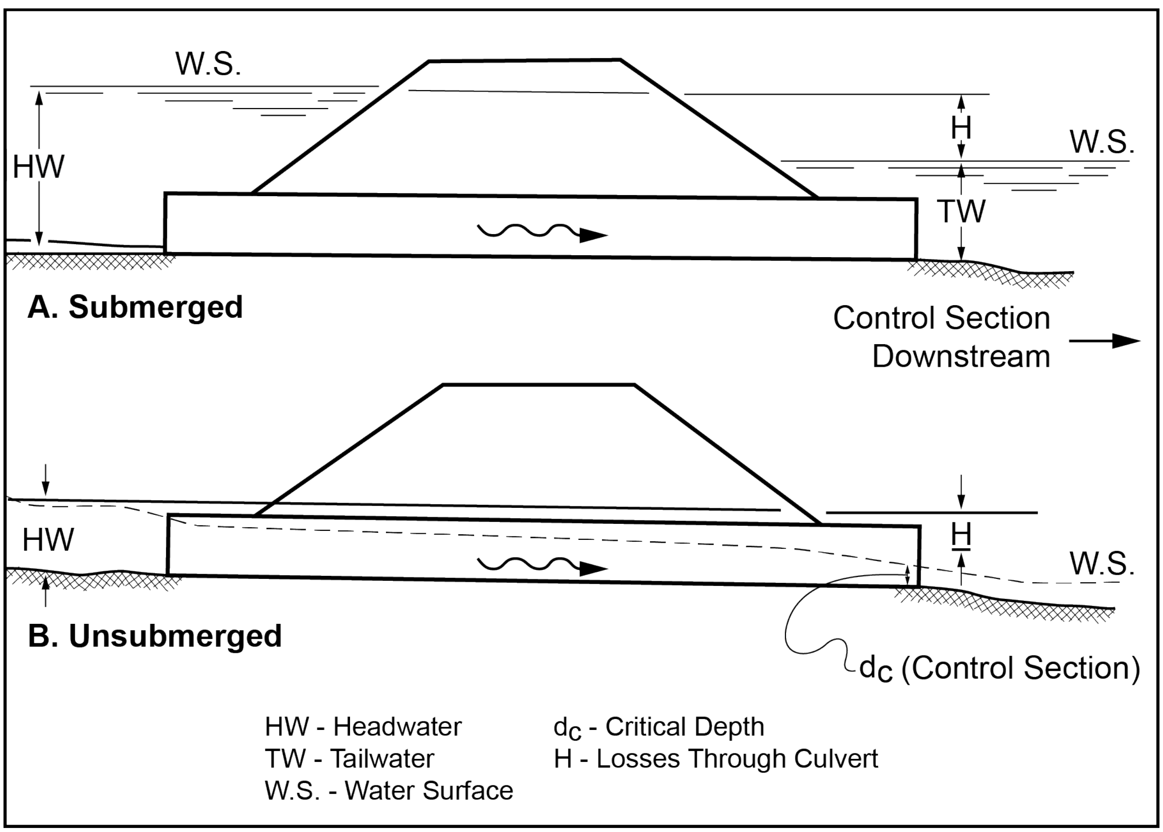


Source: FHWA 2012

Figure 4-10. Typical Inlet-controlled Culvert

4.4.1.2 Outlet Control

Culverts that have a mild slope usually operate under outlet control. A typical outlet-controlled culvert is shown in Figure 4-11, "Typical Outlet-controlled Culvert." Under outlet flow control situations, the typical flow regime condition is subcritical flow within the culvert.



Source: FHWA 2012

Figure 4-11. Typical Outlet-controlled Culvert

4.4.2 Design Criteria for Culverts

4.4.2.1 Design Flows

Culverts must be sized using, at a minimum, the design flows shown in Table 4-2 above. See Chapter 3, *Hydrology,* for information on the calculation of runoff and design flows. Culverts may be designed to permit roadway overtopping for flows greater than the design flow; however, the 100-year water surface upstream and downstream of the culvert must be analyzed to ensure that the culvert will not cause upstream and downstream flood damages (see Section 4.1, “Introduction,” of this chapter).

4.4.2.2 Freeboard

Culverts must be designed so that headwater during the design flow is at least 1 ft below the lowest point in the roadway. If the roadway is designed to be overtopped during storm events larger than the design flow, adequate channel freeboard must be provided upstream and downstream of the culvert. See Section 4.2.3.4, “Freeboard,” for more information regarding freeboard requirements. Figure D.2-4 in Appendix D.2 can be helpful in selecting a culvert to meet this criterion.

During floods, debris can accumulate at culvert inlets and can significantly reduce the culvert’s performance. In evaluating minimum clearance and freeboard requirements, design water surface profiles must account for 2 ft of debris accumulation on either side of any obstruction. Multiple-box culverts with interior walls or other mid-channel obstructions should be designed assuming debris obstruction in one of two configurations:

- In most instances, a 2-ft debris allowance should be assumed on each side of the obstruction for the full height of the obstruction, or
- If debris fins are located in the stream channel upstream of interior culvert walls, the debris allowance may be limited to the upper 2 ft of the water column at the design flow. Debris fins should be designed so that debris passes through the culvert without accumulating at the inlet. Debris fin length should be 1.5 to 2 times the culvert height. Refer to *Hydraulic Engineering Circular No. 9: Debris Control Structures – Evaluation and Countermeasures* (FHWA 2005), available at www.fhwa.dot.gov/engineering/hydraulics/pubs/04016/hec09.pdf, for more design guidance.

4.4.2.3 Minimum Velocity

To minimize sediment deposition within culverts, a minimum velocity of 2.5 ft/sec must be maintained during the 2-year flow event. The dominant sediment-transporting flow is typically associated with flow rates approximated by the 2-year event. See Chapter 3, *Hydrology*, for detailed instructions on estimating the 2-year flow. Alternatively, for the purposes of assessing compliance with this minimum velocity criterion, it is permissible to use an estimated value based on USGS regional regression equations (Gotvald et al. 2012). For the North Coast Region, where Sonoma County is located, the 2-year peak flow may be estimated using *Equation 4.1*, provided in Section 4.2.3.2, "Channel Velocity."

Sonoma Water does not specify maximum velocity design criteria.

Tables D.2-2 and D.2-3 in Appendix D.2 provide the slopes necessary for various pipe materials to maintain this minimum velocity. Figure D.2-2 in Appendix D.2 can also assist in culvert selection to meet this criterion.

4.4.2.4 Minimum Size

The minimum size for culverts is 18 inches (diameter) with no debris control structures, and 15 inches if debris control structures are used. For non-circular culvert shapes, the minimum flow area is 2.2 ft^2.

4.4.2.5 Spacing of Multiple Culverts

When multiple culverts are installed, the spacing between the outside surfaces of each pipe should be at least half the nominal diameter with a minimum of 2 ft between barrels. Note that additional clearance between pipes will be required to accommodate flared end sections.

4.4.2.6 Design Flow Conditions

Culverts conveying the flow of major or secondary waterways must provide at least 1 ft of clearance between the design water surface and the **soffit** (underside ceiling) of the culvert. If a circular culvert is used, the design depth must not exceed 80% of the culvert diameter. Figure D.2-2 in Appendix D.2 provides a useful tool for culvert selection to meet this criterion. Culverts conveying minor waterways may be designed to flow full.

Culverts that have a shape other than circular or rectangular (e.g., concrete-span arches) must maintain an equivalent area for freeboard. See Figure 4-12, "Examples of Culvert Freeboard."

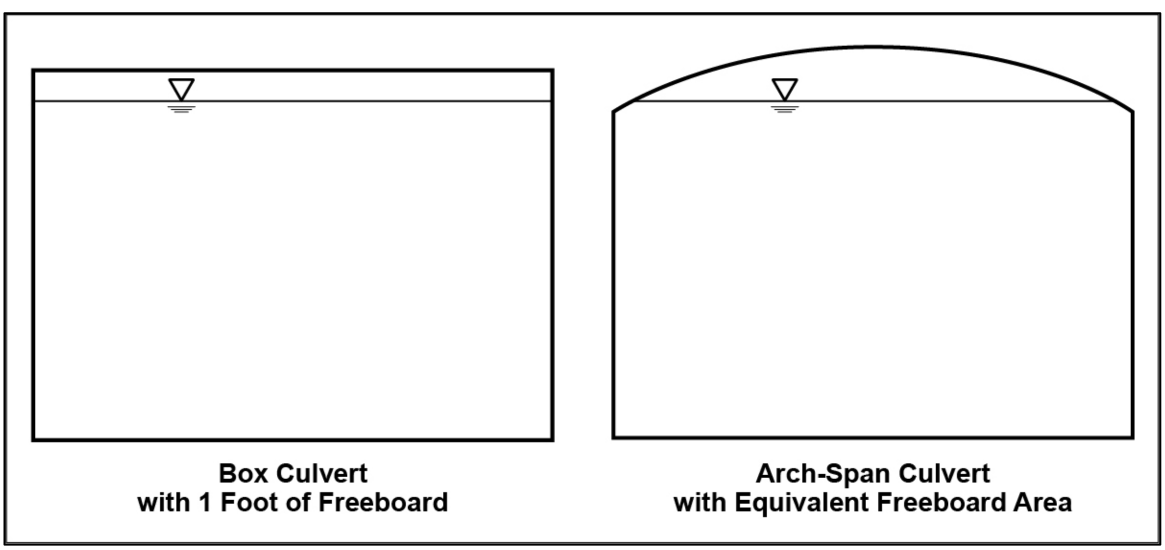


Figure 4-12. Examples of Culvert Freeboard

4.4.2.7 Culvert Alignment

Whenever possible, culverts should be aligned with the natural channel. Where culverts cross roadways, skew should be no more than 45 degrees as measured from a line perpendicular to the road centerline.

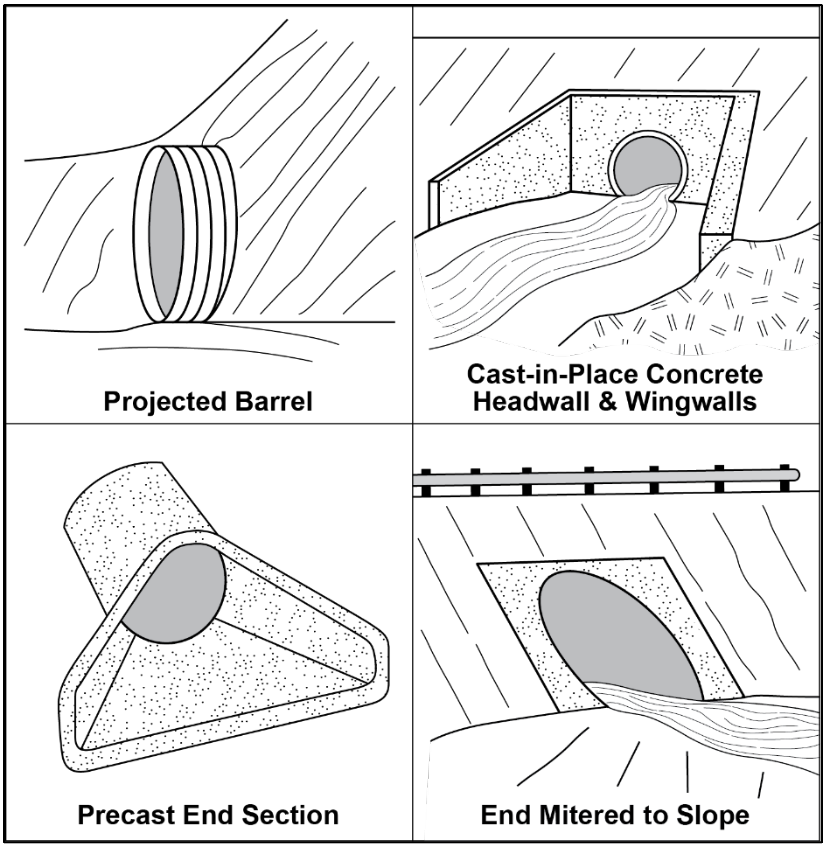
4.4.2.8 Culvert Materials

Culverts may be constructed out of any material appropriate to their setting and use, provided that the 20-foot section closest to the exposed end of the culvert is a material that is both heat-resistant and non-combustible, such as CMP, to avoid damage in the event of wildfire.

4.4.2.9 Culvert End Treatments

Flow acceleration and deceleration typically occur at the entrance and exit of culverts. This dynamic condition can cause channel and embankment erosion. End treatments provide erosion protection and also help stabilize the embankment slopes around culvert entrances and exits. End treatments may also increase the hydraulic efficiency through the culvert, thereby increasing the conveyance capacity and reducing the headwater elevation.

Typical end treatments are shown in Figure 4-13, “Typical Culvert End Treatments.” The same end treatment should be used at the entrance and exit to a culvert.

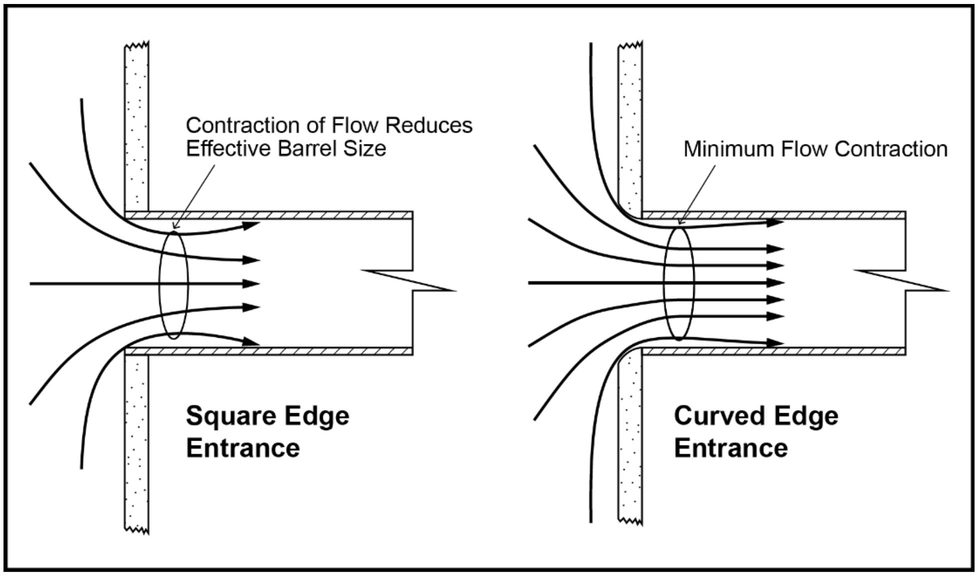


Source: FHWA 2012

Figure 4-13. Typical Culvert End Treatments

Mitering the edges of the entrance to the culvert may improve its hydraulic efficiency. With a square-edged entrance, the effective culvert size is decreased because, as water enters the culvert, flow streamlines contract. As shown in Figure 4-14, "Culvert Entrance with Square Edge and Curved Edge," rounding the edges reduces the flow contraction effect, resulting in a larger entrance flow area. When designing a culvert for flow detention, a less efficient (i.e., slower) flow opening may be desired.

See also Section 4.2.3.5, "Creek Outfalls," for specific requirements for a culvert discharging flow to a creek.



Source: FHWA 2012

Figure 4-14. Culvert Entrance with Square Edge and Curved Edge

4.4.2.10 Outlet Protection

Due to the high velocities and turbulence at culvert inlets and outlets, protection against erosion is almost always required. Culvert inlets and outlets should include an end treatment, such as a headwall with wing walls or a flared end section, to minimize the potential for culvert discharge to cause excess erosion at the outlet (see Section 4.4.2.9, "Culvert End Treatments"). The culvert outlet may require additional protection, such as riprap or an **energy dissipator**. Table 4-6, "Outlet Erosion Protection for Discharges to Unlined Channels," provides guidelines for outlet erosion protection for discharges to channels with unlined bottoms. Permitting or regulatory agencies may have additional requirements for outlet protection.

Table 4-6. Outlet Erosion Protection for Discharges to Unlined Channels

Outlet Velocity	Recommended Outlet Protection
Less than 5 ft/sec	Minimum riprap protection
Between 5 and 15 ft/sec	Scour-hole riprap protection or energy dissipator
Greater than 15 ft/sec	Energy dissipator

Outlet protection design should conform to methods described in *Hydraulic Engineering Circular No. 14: Hydraulic Design of Energy Dissipators for Culverts and Channels* (FHWA 2006). When the culvert diameter is less than 24 in, the outlet must be at least 2 ft above the revetment on the bed of the channel of the receiving waters. When the culvert diameter is equal to or greater than 24 in, the outlet elevation must be at least 1 ft above the revetment on the bed of the receiving waters channel. Documentation should be provided that the design will meet Sonoma Water requirements to prevent excessive erosion and off-site impacts.

4.4.2.11 Fish Passage Requirements

Many of the streams and channels in Sonoma County support fish populations, including habitat for federally listed and state-listed protected species. Culverts designed for use in channels that support fish populations may be required to comply with fish passage requirements. Sonoma Water does not review culvert design to determine adequacy for fish passage. Designers should contact the relevant resource agencies, including the National Marine Fisheries Service (NMFS) and the California Department of Fish and Wildlife (CDFW), for fish passage design guidance and requirements.

4.4.2.12 Detention Behind Culverts

Although the primary purpose of a culvert is to convey flow, a culvert may also be used to restrict flow. Designing the culvert to restrict flow and detain water may reduce the impact of flooding downstream. When designing a culvert for detention, sufficient detention volume must be available at the upstream face of the culvert and all headwater and freeboard requirements must be met.

4.4.2.13 Trash and Safety Racks

The use of trash and safety racks in culvert installations will be considered by Sonoma Water on a case-by-case basis. In general, trash and safety racks are not needed for most culvert installations. However, if a public safety hazard is identified with the culvert installation, trash and safety racks should be included. When designing trash and safety racks, the location of the installation should be considered, including the proximity of residents, access for trash removal and maintenance, the length and size of the culvert, and other factors affecting safety and culvert capacity.

Head loss at culvert entrances increases with the inclusion of trash and safety racks. This additional head loss must be included in design calculations. Refer to *Hydraulic Design Series No. 5: Hydraulic Design of Highway Culverts* (FHWA 2012), available at www.fhwa.dot.gov/engineering/hydraulics/pubs/12026/hif12026.pdf, for guidance on how to account for head losses at trash and safety racks.

4.4.2.14 Easements and Rights-of-Way

A revocable license is necessary for any work within any easements or property Sonoma Water manages or owns. An easement or a consent agreement is required for any permanent proposed structure or feature installed within or on property Sonoma Water manages or owns.

Regardless of the easement holder, a formal agreement between the easement holder and the Applicant that defines maintenance responsibilities associated with that structure or feature is also required.

4.4.2.15 Required Analysis Methods

Sonoma Water requires that culverts be designed with appropriate software packages (e.g., HY-8 or HEC-RAS) or nomographs, alignment charts that use diagrams, or graphs to compute a mathematical function.

4.4.3 Analysis Methods for Culverts

Two general procedures are available for the analysis and design of culverts: design software and hydraulic nomographs.

4.4.3.1 Design Software

In general, culverts are expected to be designed using computer software for ease of computation, although no specific design software package is required to be used. For simple design applications, manual calculation methods may be used.

It is recommended that design software be used for most culvert design applications. Sonoma Water specifically recommends using either HY-8 (developed by FHWA) or HEC-RAS (developed by USACE's Hydrologic Engineering Center). These two software packages are recommended for hydraulic analysis and culvert design in Sonoma County because of their relative ease of use and ability to route hydrographs and analyze road overtopping. Both packages are public-domain software and are available for download free of charge:

- HY-8 is available from the FHWA website: www.fhwa.dot.gov/engineering/hydraulics/software/hy8
- HEC-RAS is available from the HEC website: www.hec.usace.army.mil/

To design a culvert using hydraulic software, the Applicant should follow the user's manual provided with the software package. Values for entrance losses, Manning's n, loss coefficients, and other parameters are specified in Section 4.3.3.2, "Manual Calculation Methods," and Appendix D.2, Tables D.2-4 through D.2-8. Hydraulic analysis using design software should use solution criteria that solve for the highest upstream energy gradient. When culvert design software is used, the Sonoma Water submittal must include all input and output files, including a performance curve for each culvert (refer to submittal requirements described in Chapter 2, *Flood Management Design Review Process*).

4.4.3.2 Culvert Nomographs

Culvert nomographs may also be used for simple design applications. Using design nomographs involves an iterative solution process that sequentially solves for design parameters, including culvert size, flow rate, and hydraulic head. Nomographs do not directly incorporate hydrograph routing, road overtopping, or velocity calculations. If the road overtopping flow calculation is required, it should be analyzed as a weir and the weir flow calculation procedure should be used as presented in the FHWA guidance document (FHWA 2012). For convenience, values for discharge coefficients are provided in Figure D.2-5 in Appendix D.2.

To design a culvert using nomographs, the Applicant should use the detailed guidance developed by FHWA (FHWA 2012). Values for entrance losses, Manning's *n*, contraction and expansion coefficients, and other parameters are specified in Section 4.3.3.2, "*Manual Calculation Methods*," and Appendix D.2, Tables D.2-4 through D.2-8. Guidance for evaluating headwater and tailwater conditions is provided below. When using culvert nomographs for design, the Sonoma Water submittal must include marked copies of all calculations, including nomographs used, and a performance curve for each culvert (refer to submittal requirements described in Chapter 2, *Flood Management Design Review Process*).

4.4.3.3 Headwater

Refer to FHWA guidelines (FHWA 2012) for guidance on evaluating headwater conditions.

4.4.3.4 Tailwater

Hydraulic conditions downstream of a culvert must be evaluated to determine the tailwater depth for a range of discharges. At times, it may be necessary to calculate backwater curves to establish the tailwater depth conditions. The Applicant should consider the following conditions:

(1) If the culvert outlet is operating with a free outfall, the Applicant should determine the critical depth and equivalent HGL. Graphs of critical depths for circular pipe are provided in Appendix D.2 as Figure D.2-3. As previously described, *Equation 4.3* can be iteratively applied to determine critical depth in a cross-section of any shape:

$$\frac{Q^2}{g} = \frac{A_c^3}{T_c} \qquad \text{(\textit{Equation 4.3})}$$

Where,

- Q is discharge (cfs);
- g *is standard gravity (ft/sec^2);*
- A_c is the area for critical flow (ft^2); and
- T_c is the water surface width for critical flow (ft).

(2) For culverts that discharge to an open channel, lake, pond, or other major water body, the Applicant should use the tailwater elevation, or starting HGL, assumptions shown in Table 4-7, "Tailwater Elevations for Culvert Design" (identical to Table 4-4, "Tailwater Elevations for Closed Conduit Design").

Table 4-7. Tailwater Elevations for Culvert Design

Waterway Type	Area	Tailwater Elevation
Minor waterway	1 sq mi or less	10-year peak elevation
Secondary waterway	Between 1 and 4 sq mi	25-year peak elevation
Major waterway	4 sq mi or more	100-year peak elevation

(3) If an upstream culvert outlet is located near a downstream culvert inlet, the headwater elevation of the downstream culvert may establish tailwater depth for the upstream culvert.

4.5 Detention Facilities

While Sonoma Water does not generally have hydraulic design requirements for detention facilities, two risk-related design criteria do apply to detention[19] facilities, as described below.

4.5.1 Design Criteria for Detention Facilities – Emergency Overflow Structure

Any detention facility must be designed with an emergency overflow structure, such that water in excess of the facility's design capacity is routed to a suitable overtopping and conveyance feature.

4.5.2 Design Criteria for Detention Facilities – High Flow Conveyance Route

Every detention facility design must provide a reasonably safe evacuation route for floodwaters in the event that the capacity of the facility is exceeded. At minimum, conveyance of a 100-year design event with minimal risk is required for any facility that has a design capacity for a smaller event, and analysis must be conducted to justify the design. If the detention facility is designed to contain a 100-year event, consultation with Sonoma Water will be necessary to identify an appropriate design event for analysis, which will be determined in part by the public safety and

[19] For the purposes of these requirements, "detention" means capture and temporary storage of significant stormwater volumes, those well in excess of volume capture as an LID approach for Clean Water Act compliance (i.e., greater than the volume generated by 1.0 inch of rain over 24 hours).

assets at risk. Sonoma Water will also make a determination of what constitutes a “reasonably safe” evacuation route for floodwaters based on a site-specific assessment of risk.

4.6 Submittal Requirement Summary

The items required for submittal to Sonoma Water for review of hydraulic analyses are described in Chapter 2 of this manual.

Chapter 5
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