APPENDICES

Appendix A.
Glossary

Base flow	Flow in a channel that is contributed from groundwater.
Calibration	Hydrologic and hydraulic models involve many parameters to simulate physical hydrologic and hydraulic processes. Model calibration is a process whereby estimated input parameters are adjusted to better replicate actual conditions. Model calibration is typically achieved through comparing the simulated model results to actual observed data, such as comparing hydrographs produced by the model for a known rainfall-runoff event to observed streamflow data from the same event.
Catchment	The area drained by a river or body of water. Also called *catchment basin*.
Channel flow travel time	The length of time during which flow is present in a clearly defined channel.
Closed conduit	A closed conveyance system located underground. Closed conduits may flow partially full, under an open channel flow condition, or may flow full under a pressure condition.
Compliance Letter	Hard-copy letter provided to the Applicant that verifies the project is compliant with the FMDM.
Conduit	A passage (pipe or tunnel) through which water can pass.
Critical depth	The depth at which the specific energy of a given flow rate is at its minimum level. For a given discharge and cross-section geometry, there is only one critical depth.
Cross-section	The two-dimensional area of flowing fluid measured perpendicular to the direction of flow.
Culvert	A structure used to convey surface runoff or a watercourse through an embankment. Culverts, as distinguished from bridges, are usually covered with embankment and composed of structural material around the entire perimeter, although some are supported on spread footings with the streambed serving as the bottom of the culvert.
Curb	An enclosing border or edging; raised strips of concrete along the edges of streets or parking lots.
Curve number (*CN*)	A watershed parameter that represents the impermeability of the land in a watershed.
Design rainfall	The rainfall used to design drainage or flood control improvements.
Design storm	A selected storm event for which drainage or flood control improvements are designed; typically described in terms of frequency, or the probability of occurrence in a given year. For example, the 100-year design storm is the storm with a 1% (1-in-100) chance of occurring in any given year.

Design storm frequency

The expected frequency for a given storm event is the chance that it will be equaled or exceeded in a given year. The frequency is the reciprocal of the probability. The probability is the likelihood of the event. The magnitude of probability is between 0 (cannot possibly happen) and 1 (must happen). In hydrology, engineers are interested in the consequences of exceeding system capacity. Thus, engineers specify exceedance probability. The following table lists standard design frequencies and probabilities of flooding.

Frequency (years)	Flooding Probability (%)
2	50
5	20
10	10
25	4
50	2
100	1

Detention

Detention is the temporary holding of runoff to protect an area from flooding. For the purposes of this Manual, 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).

Detention facility

A facility or basin that is specifically designed and engineered to receive and temporarily hold runoff to protect an area from flooding. Detention facilities are typically used to mitigate the effects of flooding caused by development, which increases impervious surfaces and thus increases the quantity of runoff.

Easement

A right held by a party to use land belonging to another party for a specific purpose. Mains, drains, water pipes, and other conveyances are often allowed to travel across private property by means of easements.

Energy dissipator

A device to dissipate the energy of flow in a channel or culvert. Energy dissipators can be constructed in several ways and from different materials, including rocks, logs, steel baffles, and concrete blocks.

Energy grade line (EGL)

The line above a datum used to express the total energy of a flow.

Federal Emergency Management Agency (FEMA)

The federal agency within the U.S. Department of Homeland Security that is tasked with responding to, planning for, recovering from, and mitigating against human-made and natural disasters.

Flow conveyance system

A constructed or natural channel system that transmits flow from one location to another.

Flood hazard

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.

Flood Insurance Rate Map (FIRM)	An official map of a community within the United States that displays the floodplains, special hazard areas, and risk premium zones, as delineated by FEMA.
Floodplain	An area of land adjacent to a river that is susceptible to being inundated by water.
Free outfall	A culvert or conduit that has a tailwater at a depth equal to or lower than critical depth. For culverts having free outfalls, lowering of the tailwater has no effect on the discharge or the backwater profile upstream of the tailwater.
Freeboard	The vertical distance between the design water surface elevation and the top of the channel (or levee).
Froude number	A dimensionless parameter in the context of flow regime that characterizes the state of flow as subcritical, critical, or supercritical.
Gutter	The edge of the street below the curb designed to drain water from streets, driveways, and parking lots into catchment basins.
Gutter flow line	A line representing the lowest point in the bottom of the gutter.
Headwater	The depth of water at the inlet end of a pipe, culvert, or bridge waterway.
HEC-HMS	A Windows-based computer program developed by the U.S. Army Corps of Engineers, Hydrologic Engineering Center (HEC) and used for flood hydrograph computations.
HEC-RAS	The River Analysis System (RAS) computer program developed by the U.S. Army Corps of Engineers, Hydrologic Engineering Center (HEC) and used for hydraulic modeling computations. HEC-RAS contains one-dimensional river analysis components for (1) steady-flow water surface profile computations; (2) unsteady-flow simulation; (3) movable boundary sediment transport computations; and (4) water quality analysis.
HY-8	A software package for hydraulic analysis and culvert developed by FHWA.
Hydraulic grade line (HGL)	A line whose plotted ordinate position represents the sum of pressure head plus elevation head for the various positions along a given fluid flow path, such as along a pipeline or a groundwater streamline.
Hydraulic radius	In viewing the channel cross-section, the hydraulic radius is the area of the cross-section occupied by flow/discharge, divided by the wetted perimeter, or the total length of the cross-section surface in contact with water.
Hydrograph	The rate of flow (discharge) versus time at a specific point in a river, channel, or conduit carrying flow. The rate of flow is typically expressed in cubic meters per second (cms) or cubic feet per second (cfs).
Hydrograph method	The hydrograph method for estimating stormwater runoff is used for relatively large watersheds where runoff characteristics are more complex than those for which the rational method is applicable. The hydrograph method should also be used when storage must be evaluated.

Hyetograph	A graphical representation of rainfall distribution, the amount of precipitation that falls over time.
Infiltration	Rainfall that seeps into a pervious surface.
Initial abstractions	All rainfall losses occurring before the beginning of surface runoff, including interception, infiltration, and depression storage.
Inlet	A structure that intercepts stormwater on the ground surface or street gutter and conveys it into the storm drain system.
Invert	When used in relation to *culvert*, the flow line of the culvert (inside bottom). However, if the culvert is buried to accommodate fish passage, then the invert should be considered the streambed.
Lag time	Lag time has a variety of slightly different definitions, but is typically the difference in time from the center of mass of excess precipitation that generates runoff to the peak in the hydrograph for that same precipitation event.
Levee	A natural, modified, or engineered raised embankment along the edge of a river channel. Levees can develop naturally through the overbank deposition of sediment, be modified or created embankments that are not engineered features, or be engineered features designed to contain a specific flow condition.
Low Impact Development	(Also known as green infrastructure.) A planning and design strategy used to avoid and reduce potential harmful water resource impacts associated with residential and commercial development. Most commonly, this approach is focused toward preventing excess runoff generation and water quality impacts associated with development and urban stormwater.
Major waterway	Major waterways are sized to carry a 100-year flood event and have a drainage area of 4 square miles or more. These facilities should be designed to prevent property damage and loss of life.
Manning’s equation	An empirical equation relating open channel flow velocity to flow area and channel slope.
Minor waterway	A minor waterway is designed to handle a 10-year flood event and have a drainage area of 1 square mile or less. These facilities reduce the frequency of street flooding and provide protection against regularly occurring flooding. A minor waterway cannot convey major runoff events.
Model calibration	The process of determining the degree to which the model accurately reflects or represents the actual physical system.
Model validation	The process of adjusting model parameters to improve model results that better represent the actual physical system.
National Flood Insurance Program (NFIP)	The federal regulatory program under which flood-prone areas are identified and flood insurance is made available to residents of participating communities.
Nomograph	A chart relating three or more scales across which a straight-edge can be placed to provide a graphical solution for a particular problem.

North American Datum of 1927 (NAD 27)	The North American Datum of 1927 (NAD 27) is a unified horizontal or geometric datum computed in 1927 by the United States Coast and Geodetic Survey (see below).
North American Datum of 1983 (NAD 83)	The North American Datum of 1983 (NAD 83) is a unified horizontal or geometric datum and successor to NAD27 created by the National Geodetic Survey (see above).
North American Vertical Datum of 1988 (NAVD 88)	A geodetic reference for elevations, created by the National Geodetic Survey in 1988 to replace the NGVD 29 (see below).
National Geodetic Vertical Datum of 1929 (NGVD 29)	A geodetic reference for elevations, completed and adjusted in 1929. These elevations were used to define the mean sea level datum until 1988, when it was replaced by NAVD 88 (see above).
Open channel	A constructed channel or natural waterway.
Overland flow	Overland flow is the flow of water across the land surface that occurs when excess runoff, stormwater, meltwater, or other water sources exceed the ability for the land to infiltrate.
Permeability	Penetrability; the quality of being penetrable. The ability for runoff to infiltrate into the soil or substrate is a function of both permeability and porosity (the area of voids in the soil or substrate). Hardened land covers such as concrete, asphalt, or compacted gravel are typically impervious and cause water to run off.
Rational method	A method to estimate stormwater runoff for relatively small drainage areas using the rational formula.
Retention facility	Retention facilities store stormwater permanently or indefinitely. Water is stored until it is absorbed through percolation, taken in by plants, or lost through evaporation. Retention facilities should only be constructed where the groundwater table is shown to accommodate such percolation.
Revision Letter	Hard-copy letter issued by Sonoma Water to the Applicant if a project submittal requires revision. The Revision Letter will identify comments, questions, or other requests to revise or clarify the submittal, and will enumerate each specific comment or question that the Applicant is required to address.
Roughness	Roughness is used to represent the amount of frictional resistance water experiences when flowing. Roughness contributors include bed and bank material, vegetation, transport of sediment, and hydraulic turbulence. Manning's roughness coefficient, n, is used to represent the degree of roughness in channels. Flow velocity is strongly dependent on slope and roughness (the resistance to flow).
Runoff	That part of the precipitation that runs off the surface of a drainage area after all the abstractions are accounted for.
Soffit	The underside ceiling of a bridge, culvert, or storm drain pipe.

Steady-state flow The condition where the fluid properties at any single point in the system do not change over time. These fluid properties include temperature, pressure, and velocity. One of the most significant properties that is constant in a steady-state flow system is the system mass flow rate (discharge).

Streamflow The volume of water in a surface stream passing a given point per unit of time.

Subcritical flow A flow condition in which the velocity is less than the critical velocity and the depth is greater than the critical depth.

Submerged inlet A submerged inlet occurs when the headwater is greater than 1.2 times the culvert diameter or barrel height.

Submerged outlet A submerged outlet occurs when the tailwater elevation is higher than the crown of the culvert.

Supercritical flow A flow condition in which the velocity is greater than the critical velocity and the depth is less than the critical depth.

Swale A depression in the land surface that is at least seasonally wet, is usually vegetated, and normally does not contain flowing water. Swales conduct stormwater into primary drainage channels and may provide some groundwater recharge.

Tailwater The water surface elevation at the downstream side of a hydraulic structure such as a culvert, dam, or bridge.

Thalweg The line of maximum depth in a stream. The thalweg is the part of a stream that has the maximum velocity and causes cutbanks and channel migration.

Time of concentration (t_c) The time required for runoff to travel from the hydraulically most distant point of the watershed to a point of interest within the watershed.

Top of bank The bank that is at or above the elevation of the adjacent natural ground outside of the waterway.

Unsteady-state flow The condition where the fluid properties at any single point in the system can change over time. These fluid properties include temperature, pressure, and velocity. One of the most significant properties that can change in an unsteady-state flow system is the system mass flow rate (discharge).

Validation The process of testing a hydrologic or hydraulic model by comparing model results with data that are obtained independent of the modeling process. This testing and evaluation process provides a measure of the reliability of the model.

Watershed A land area that drains to a single point of discharge.

Weir A depressed channel in a dam providing an outlet for the overflow of water when the water level exceeds a desired height.

Wetted perimeter That portion of the perimeter of a stream channel cross-section that is in contact with the water.

Appendix B.

Flood Management Design Review Process Forms and Materials

Appendix B.1

Development Submittal Information Form

Sonoma Water

Development Submittal Information

Please type or print the following information:

Development or Project: ______________________________

Property Address, City, Zip: ______________________________

Nearest Cross-Street: ______________________________

Assessor's Parcel No.(s): ______________________________

Design Engineer: ______________________________

Contact Name: ________________ E-mail: ________________

Address, City, Zip: ______________________________

Phone: ________________ Fax: ________________

Developer: ______________________________

Contact Name: ________________ E-mail: ________________

Address, City, Zip: ______________________________

Phone: ________________ Fax: ________________

Fee based on: __________ Minimum = ________________

__________ Lots @________per lot = ________________

__________ Acres @________per acre = ________________

Minor Subdivision = ________________

To be completed by Sonoma Water

File/Unique No: Quad Maps:

	Amount	Date	Receipt No.
Fee Paid	____________	____________	____________

Review Engineer: ________________________ **Final Letter Date:** ____________

Comments:

Project Task # ____________

Appendix B.2

Flood Management Design Review Plan Submittal Checklist

Flood Management Design Review Plan Submittal Checklist

Project Name: ________________________________ Date: __________

Project Address: ____

<u>All of the following items</u> must be submitted before a flood management design review can be completed.

Please submit the following items or indicate why they are not necessary.

ID #	Page #	General
______	______	☐ Transmittal Letter
______	______	☐ Applicant Information Sheet
______	______	☐ Plan Submittal Log
______	______	☐ Plan Check Fee

ID #	Page #	Plan
______	______	☐ Civil Improvement Plans
______	______	☐ Assessor's Parcel Map with site outlined
______	______	☐ Final Map or Parcel Map, as applicable
______	______	☐ Conditions of Approval

ID #	Page #	Hydrology
______	______	☐ Hydrology Map, identifying area of each drainage or tributary to each structure
______	______	☐ Coefficients and measured or calculated parameters used in analysis (For IRM, include at least Runoff Coefficients for each subbasin, with slopes, soil types; for SUHM, include at least Curve Numbers for each subbasin, soil types, ARF value, storm duration, time of concentration, peaking coefficient, intensity position)
______	______	☐ Hydrology Calculations: Design flow (include calculations if completed by hand; model results and model files if completed using software package)
______	______	☐ Hydrology Calculations: 100-yr flow (include calculations if completed by hand; model results and model files if completed using software package)

ID #	Page #	Flow Conveyance (Hydraulics)
_______	_______	☐ Coefficients and measured or calculated parameters used in analysis (summary)
_______	_______	☐ Values and source(s) of starting HGLs
_______	_______	☐ Hydraulic calculations: Open channel flow (Section 4.2; include calculations if completed by hand; model results and model files if completed using software package)
_______	_______	☐ Hydraulic calculations: Conduits flow (Section 4.3; include calculations if completed by hand; model results and model files if completed using software package)
_______	_______	☐ Hydraulic calculations: Culverts flow (Section 4.4; include calculations if completed by hand; model results and model files if completed using software package)
_______	_______	☐ EGL and HGL plots for design flow
_______	_______	☐ Hydraulic calculations: Gutter water depth (*Section 4.3.2.11*)
_______	_______	☐ Hydraulic calculations: Inlet capacity (*Section 4.3.2.12*)
_______	_______	☐ 100-yr flow routing map
_______	_______	☐ 100-year storm elevations vs. finished floor elevations
_______	_______	☐ FEMA Flood Insurance Rate Maps, if applicable

Appendix B.3

Applicant Submittal Log

Sonoma Water

Submittal Log

Project Name: Big Creek Subdivision Date: 6 / 15 / 2018

Project Address: 1234 Big Creek Drive, Santa Rosa, CA 95409

EXAMPLE: SUBMITTAL LOG

Submittal ID #	Submittal Name	Submittal Date	Subject(s) (General, Plan, Hydrology, Hydraulics, Other)	Format (P-paper or D-digital: report[1], map, drawing, data, calculations[2])
1	Parcel Map with site outline	6/1/2018	General	P-map
2	Improvement plans	6/1/2018	Plan	P-drawing
3	Hydrology map	~~6/1/2018~~ 6/15/2018	Hydrology	~~P-map~~ P-updated map
4	Hydrologic input parameters	6/1/2018	Hydrology	P-report
5	Hydrologic calculations: Design Event	6/1/2018	Hydrology	P-calculations
6	Conditions of approval	6/1/2018	Plan	P-report
7	Hydraulic parameters	6/1/2018	Hydraulics	P-report
8	Hydraulic calculations	6/1/2018	Hydraulics	P-calculations
	ETC.			

[1] Any text-focused document, letter, memorandum, etc.
[2] Including any model files or modeling results.

Sonoma Water

Submittal Log

Project Name: ____ Date: _____/_____/20______

Project Address: ____

SUBMITTAL LOG

Submittal ID #	Submittal Name	Submittal Date	Subject(s) (General, Plan, Hydrology, Hydraulics, Other)	Format (P-paper or D-digital: report[1], map, drawing, data, calculations[2])

[1] Any text-focused document, letter, memorandum, etc.
[2] Including any model files or modeling results.

P.O. Box 11628, Santa Rosa, CA 95406 Phone: (707) 547-1900 FAX: (707) 524-3782

Appendix C.

Chapter 3, *Hydrology* Reference Materials

Table C-1. Runoff Coefficients (*Cs*) (Incremental Rational Method)

Land Use	Lot Size (acres)	Impervious Fraction	Average Slope (%)			
			0-2	>2-6	>6-12	>12
Soil Type A						
Residential[1]						
Rural		0.03	0.24	0.28	0.34	0.38
Very low density	2	0.11	0.29	0.34	0.38	0.42
	1	0.24	0.38	0.42	0.46	0.49
Low density	1/2	0.32	0.43	0.47	0.50	0.53
	1/3	0.41	0.50	0.53	0.56	0.58
Medium-low density	1/4	0.49	0.55	0.58	0.60	0.62
Medium density	1/8	0.70	0.70	0.71	0.73	0.74
Medium-high density	1/18	1	0.90	0.90	0.90	0.90
Business, commercial, etc.		1	0.90	0.90	0.90	0.90
General industrial		1	0.90	0.90	0.90	0.90
Parks and recreation		0.05	0.25	0.25	0.30	0.35
Ag and open space		0.02	0.23	0.23	0.28	0.33
Soil Type B						
Residential[1]						
Rural		0.03	0.28	0.33	0.39	0.43
Very low density	2	0.11	0.34	0.38	0.43	0.47
	1	0.24	0.42	0.45	0.50	0.53
Low density	1/2	0.32	0.47	0.50	0.54	0.57
	1/3	0.41	0.53	0.56	0.59	0.61
Medium-low density	1/4	0.49	0.58	0.60	0.63	0.65
Medium density	1/8	0.70	0.71	0.73	0.74	0.76
Medium-high density	1/18	1	0.90	0.90	0.90	0.90
Business, commercial, etc.		1	0.90	0.90	0.90	0.90
General industrial		1	0.90	0.90	0.90	0.90
Parks and recreation		0.05	0.25	0.30	0.34	0.40
Ag and open space		0.02	0.23	0.28	0.33	0.38
Soil Type C						
Residential[1]						
Rural		0.03	0.33	0.38	0.43	0.47
Very low density	2	0.11	0.38	0.42	0.47	0.51
	1	0.24	0.45	0.49	0.53	0.57
Low density	1/2	0.32	0.50	0.53	0.57	0.60

Land Use	Lot Size (acres)	Impervious Fraction	Average Slope (%)			
			0-2	>2-6	>6-12	>12
	1/3	0.41	0.56	0.59	0.62	0.64
Medium-low density	1/4	0.49	0.60	0.63	0.65	0.68
Medium density	1/8	0.70	0.73	0.74	0.76	0.77
Medium-high density	1/18	1	0.90	0.90	0.90	0.90
Business, commercial, etc.		1	0.90	0.90	0.90	0.90
General industrial		1	0.90	0.90	0.90	0.90
Parks and recreation		0.05	0.34	0.39	0.44	0.48
Ag and open space		0.02	0.33	0.38	0.43	0.47
Soil Type D						
Residential[1]						
Rural		0.03	0.38	0.43	0.48	0.52
Very low density	2	0.11	0.42	0.47	0.52	0.55
	1	0.24	0.49	0.53	0.57	0.60
Low density	1/2	0.32	0.54	0.57	0.61	0.63
	1/3	0.41	0.59	0.62	0.65	0.67
Medium-low density	1/4	0.49	0.63	0.65	0.68	0.70
Medium density	1/8	0.70	0.74	0.76	0.77	0.78
Medium-high density	1/18	1	0.90	0.90	0.90	0.90
Business, commercial		1	0.90	0.90	0.90	0.90
General industrial		1	0.90	0.90	0.90	0.90
Parks and recreation		0.05	0.39	0.44	0.49	0.53
Ag and open space		0.02	0.38	0.42	0.48	0.52

[1] *Percent impervious values are based on analysis conducted by ESA for Sonoma County Water Agency (Sonoma Water) in 2014, using a sample of existing developed areas.*

[2] *For residential areas, composite* C *values were developed as follows:* C *values for soil type from Los Angeles County Hydrology Manual (1991) were modified for slope using the vegetated areas curve from Plate B-1 of SCWA (1983) for pervious areas within a given slope range and a* C *of 0.90 for all impervious areas.*

Source: Approach adapted from McCuen 1989

Table C-2. Runoff Curve Numbers (CNs) for Urban Areas (Synthetic Unit Hydrograph Method)

Cover Type and Hydrologic Condition[1]	Average Percent Impervious Area[2]	Curve Numbers for Hydrologic Soil Group			
		A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.)[3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only)[4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:[5]					
1/8 acre or less (town houses)	70	81	87	91	93
1/4 acre	49	68	79	86	89
1/3 acre	41	63	76	84	87
1/2 acre	32	58	73	82	86
1 acre	24	53	70	80	84
2 acres	11	46	65	77	82
Developing urban areas, newly graded areas (pervious areas only, no vegetation)[6]		77	86	91	94
Idle land (*CN*s are determined using cover types in Table C-3)					

[1] *Average runoff condition, and $I_a = 0.2S$.*

[2] *The average percent impervious area shown was used to develop the composite* CNs. *Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a* CN *of 98, and pervious areas are considered equivalent to open space in good hydrologic condition.*

[3] CN*s shown are equivalent to those of pasture. Composite* CN*s may be computed for other combinations of open space cover type.*

[4] *Composite* CN*s for natural desert landscaping should be computed based on the impervious area percentage (*CN *= 98) and the pervious area* CN*. The pervious area* CN*s are assumed to be equivalent to desert shrub in poor hydrologic condition.*

[5] *Percent impervious based on analysis of Sonoma County conditions conducted by ESA for Sonoma Water in 2014.* CN*s have been recalculated to reflect these assumptions.*

[6] *Composite* CN*s for the design of temporary measures during grading and construction should be computed based on the degree of development (impervious area percentage) and the* CN*s for the newly graded pervious areas.*

Source: Adapted from SCS 1986

Table C-3. Runoff Curve Numbers (CNs) for Cultivated Agricultural Lands (Synthetic Unit Hydrograph Method)

Cover Type	Treatment[1]	Hydrologic Condition[2]	Curve Numbers for Hydrologic Soil Group[3]			
			A	B	C	D
Fallow	Bare soil	—	77	86	91	94
	Crop residue cover (CR)	Poor	76	85	90	93
		Good	74	83	88	90
Row crops	Straight row (SR)	Poor	72	81	88	91
		Good	67	78	85	89
	SR + CR	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured (C)	Poor	70	79	84	88
		Good	65	75	82	86
	C + CR	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured & terraced (C&T)	Poor	66	74	80	82
		Good	62	71	78	81
	C&T+ CR	Poor	65	73	79	81
		Good	61	70	77	80
Small grain	SR	Poor	65	76	84	88
		Good	63	75	83	87
	SR + CR	Poor	64	75	83	86
		Good	60	72	80	84
	C	Poor	63	74	82	85
		Good	61	73	81	84
	C + CR	Poor	62	73	81	84
		Good	60	72	80	83
	C&T	Poor	61	72	79	82
		Good	59	70	78	81
	C&T+ CR	Poor	60	71	78	81
		Good	58	69	77	80
Close-seeded or broadcast legumes or rotation meadow	SR	Poor	66	77	85	89
		Good	58	72	81	85
	C	Poor	64	75	83	85
		Good	55	69	78	83
	C&T	Poor	63	73	80	83
		Good	51	67	76	80

[1] *Crop residue cover applies only if residue is on at least 5% of the surface throughout the year.*

[2] *Hydrologic condition is based on a combination of factors that affect infiltration and runoff, including (a) density and canopy of vegetative areas, (b) amount of year-round cover, (c) amount of grass or close-seeded legumes, (d) percent of residue cover on the land surface (good ≥ 20%), and (e) degree of surface roughness.*

Poor: Factors impair infiltration and tend to increase runoff.

Good: Factors encourage average or better-than-average infiltration and tend to decrease runoff.

[3] *Average runoff condition, and* I_a*=0.2S*

Source: SCS 1986

Table C-4. Runoff Curve Numbers (CNs) for Other Agricultural Lands (Synthetic Unit Hydrograph Method)

Cover Type	Hydrologic Condition	Curve Numbers for Hydrologic Soil Group[1]			
		A	B	C	D
Pasture, grassland, or range—continuous forage for grazing[2]	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow—continuous grass, protected from grazing and generally mowed for hay	—	30	58	71	78
Brush—brush-weed-grass mixture with brush the major element[3]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	30 [4]	48	65	73
Woods—grass combination (orchard or tree farm)[5]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30[4]	55	70	77
Farmsteads—buildings, lanes, driveways, and surrounding lots	—	59	74	82	86

[1] *Average runoff condition, and $I_a = 0.2S$.*

[2] *Poor: < 50% ground cover or heavily grazed with no mulch.*
Fair: 50% to 75% ground cover and not heavily grazed.
Good: > 75% ground cover and lightly or only occasionally grazed.

[3] *Poor: < 50% ground cover.*
Fair: 50% to 75% ground cover.
Good: > 75% ground cover.

[4] *Actual curve number is less than 30; use* CN *= 30 for runoff computations.*

[5] CN*s shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from the* CN*s for woods and pasture.*
Poor: Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.
Fair: Woods are grazed but not burned, and some forest litter covers the soil.
Good: Woods are protected from grazing, and litter and brush adequately cover the soil.

Source: Adapted from SCS 1986

Table C-5. Runoff Curve Numbers (CNs) for Arid and Semiarid Rangelands (Synthetic Unit Hydrograph Method)

Cover Type	Hydrologic Condition[1]	Curve Numbers for Hydrologic Soil Group[2]			
		A[3]	B	C	D
Herbaceous—mixture of grass, weeds, and low-growing brush, with brush the minor element	Poor		80	87	93
	Fair		71	81	89
	Good		62	74	85
Oak-aspen—mountain brush mixture of oak brush, aspen, mountain mahogany, bitter brush, maple, and other brush	Poor		66	74	79
	Fair		48	57	63
	Good		30	41	48
Pinyon-juniper—pinyon, juniper, or both; grass understory	Poor		75	85	89
	Fair		58	73	80
	Good		41	61	71
Sagebrush with grass understory	Poor		67	80	85
	Fair		51	63	70
	Good		35	47	55
Desert shrub—saltbush, greasewood, creosotebush, blackbrush, bursage, palo verde, mesquite, and cactus	Poor	63	77	85	88
	Fair	55	72	81	86
	Good	49	68	79	84

1 *Poor: < 30% ground cover (litter, grass, and brush overstory).*
Fair: 30% to 70% ground cover.
Good: > 70% ground cover.

2 *Average runoff condition and I_a, = 0.2S.*

3 CN*s for Group A have been developed only for desert shrub.*

Source: SCS 1986

Note: *TR-55 presents* CN*s for three different antecedent moisture conditions (I, II, and III). All tables shown in this manual are for Antecedent Moisture Condition II. This condition should be used for all design analyses unless otherwise indicated and documented.*

Table C-6. Areal Reduction Factors

Area (sq-mi)	Storm duration (% reduction from point precipitation to areal precipitation)							
	5 minutes	**15 minutes**	**30 minutes**	**1 hour**	**3 hour**	**6 hour**	**12 hour**	**24 hour**
0	100%	100%	100%	100%	100%	100%	100%	100%
5	89.3%	92.2%	93.4%	96.7%	98.3%	98.9%	99.0%	99.2%
10	82.0%	86.8%	89.1%	93.8%	97.1%	97.9%	98.1%	98.6%
15	76.0%	82.3%	85.5%	91.3%	95.9%	97.1%	97.4%	97.9%
20	70.5%	78.1%	82.1%	89.1%	94.8%	96.2%	96.6%	97.3%
25	65.5%	74.4%	79.0%	87.2%	93.9%	95.5%	96.0%	96.9%
30	61.4%	71.3%	76.5%	85.4%	93.0%	94.9%	95.5%	96.5%
35	57.9%	68.6%	74.3%	83.7%	92.1%	94.2%	95.0%	96.1%
40	55.1%	66.4%	72.4%	82.4%	91.3%	93.7%	94.5%	95.8%
45	52.4%	64.3%	70.7%	81.1%	90.5%	93.2%	94.1%	95.5%
50	50.1%	62.5%	69.2%	79.9%	89.8%	92.6%	93.7%	95.2%
55	48.1%	60.9%	67.9%	78.8%	89.2%	92.2%	93.4%	95.0%
60	46.4%	59.5%	66.7%	77.9%	88.5%	91.8%	93.0%	94.8%
65	45.0%	58.4%	65.7%	77.0%	87.9%	91.4%	92.7%	94.5%
70	43.6%	57.3%	64.8%	76.2%	87.4%	91.0%	92.4%	94.4%
75	42.6%	56.3%	64.0%	75.4%	86.8%	90.7%	92.1%	94.2%
80	41.5%	55.4%	63.2%	74.7%	86.4%	90.4%	91.9%	94.0%
85	40.6%	54.7%	62.6%	74.0%	85.9%	90.1%	91.6%	93.8%
90	39.8%	54.0%	62.0%	73.4%	85.4%	89.7%	91.4%	93.7%
95	39.0%	53.3%	61.4%	72.8%	85.0%	89.4%	91.2%	93.6%
100	38.4%	52.7%	60.9%	72.2%	84.6%	89.2%	90.9%	93.4%
105	37.8%	52.2%	60.4%	71.7%	84.2%	88.9%	90.7%	93.3%
110	37.3%	51.7%	60.0%	71.3%	83.8%	88.6%	90.6%	93.2%
115	36.8%	51.3%	59.6%	70.9%	83.5%	88.3%	90.4%	93.1%
120	36.3%	50.9%	59.2%	70.5%	83.2%	88.1%	90.2%	93.0%
125	35.9%	50.4%	58.8%	70.1%	82.9%	87.8%	90.0%	92.9%
130	35.4%	50.0%	58.5%	69.8%	82.6%	87.6%	89.9%	92.8%
135	35.1%	49.7%	58.2%	69.5%	82.3%	87.4%	89.7%	92.7%
140	34.7%	49.4%	57.9%	69.2%	82.1%	87.2%	89.6%	92.7%
145	34.3%	49.1%	57.6%	68.9%	81.9%	86.9%	89.5%	92.6%

Area (sq-mi)	Storm duration (% reduction from point precipitation to areal precipitation)							
	5 minutes	**15 minutes**	**30 minutes**	**1 hour**	**3 hour**	**6 hour**	**12 hour**	**24 hour**
150	34.0%	48.7%	57.3%	68.6%	81.7%	86.7%	89.3%	92.5%
155				68.4%	81.4%	86.5%	89.2%	92.4%
160				68.1%	81.3%	86.3%	89.1%	92.4%
165				67.9%	81.0%	86.2%	88.9%	92.3%
170				67.7%	80.8%	86.0%	88.8%	92.3%
175				67.5%	80.7%	85.9%	88.7%	92.2%
180				67.3%	80.5%	85.7%	88.6%	92.2%
185				67.2%	80.3%	85.6%	88.5%	92.1%
190				67.0%	80.1%	85.4%	88.5%	92.1%
195				66.9%	80.0%	85.3%	88.4%	92.0%
200				66.7%	79.8%	85.2%	88.3%	92.0%
205				66.6%	79.7%	85.1%	88.2%	91.9%
210				66.5%	79.6%	85.0%	88.1%	91.9%
215				66.4%	79.4%	84.9%	88.0%	91.8%
220				66.3%	79.3%	84.8%	88.0%	91.8%
225				66.2%	79.2%	84.7%	87.9%	91.7%
230				66.1%	79.2%	84.6%	87.9%	91.7%
235				66.0%	79.1%	84.5%	87.8%	91.7%
240				65.9%	79.0%	84.5%	87.8%	91.6%
245				65.9%	78.9%	84.3%	87.7%	91.6%
250				65.8%	78.9%	84.3%	87.7%	91.6%
255				65.7%	78.8%	84.2%	87.6%	91.6%
260				65.6%	78.7%	84.1%	87.6%	91.6%
265				65.6%	78.6%	84.0%	87.6%	91.5%
270				65.5%	78.6%	84.0%	87.5%	91.5%
275				65.5%	78.5%	83.9%	87.5%	91.5%
280				65.4%	78.5%	83.9%	87.4%	91.5%
285				65.4%	78.5%	83.8%	87.4%	91.4%
290				65.4%	78.4%	83.8%	87.4%	91.4%
295				65.3%	78.4%	83.7%	87.3%	91.4%
300				65.3%	78.4%	83.7%	87.3%	91.3%
305					78.4%	83.6%	87.3%	91.3%

Area (sq-mi)	Storm duration (% reduction from point precipitation to areal precipitation)							
	5 minutes	**15 minutes**	**30 minutes**	**1 hour**	**3 hour**	**6 hour**	**12 hour**	**24 hour**
310					78.4%	83.6%	87.2%	91.3%
315					78.3%	83.5%	87.2%	91.3%
320					78.3%	83.5%	87.2%	91.2%
325					78.3%	83.5%	87.1%	91.2%
330					78.3%	83.5%	87.1%	91.2%
335					78.3%	83.4%	87.1%	91.2%
340					78.3%	83.4%	87.1%	91.2%
345					78.2%	83.4%	87.1%	91.1%
350					78.2%	83.4%	87.1%	91.1%
355						83.4%	87.2%	91.1%
360						83.3%	87.2%	91.0%
365						83.3%	87.2%	91.0%
370						83.3%	87.2%	91.0%
375						83.3%	87.1%	91.0%
380						83.2%	87.1%	91.0%
385						83.2%	87.1%	91.0%
390						83.2%	87.1%	91.0%
395						83.1%	87.1%	91.0%
400						83.2%	87.1%	91.0%

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Appendix D.

Chapter 4, *Flow Conveyance* Reference Materials

Appendix D.1.

Sediment Transport and Channel Stability

Appendix D.1. Sediment Transport and Channel Stability

A variety of methods can be used to assess channel stability and sediment transport in an open channel. Sediment transport studies should be used to select appropriate designs for channel geometry, identify areas that may be subject to channel aggradation or degradation, or otherwise refine the channel design.

A wide range of channel assessment methods (both field-based and desktop) are available to evaluate channel stability. These methods include relatively simple erodibility assessments and more detailed sediment transport simulation models. Table D.1-1., "Suitable Sediment Transport Assessment Methodologies," provides recommended approaches to sediment transport studies for various open-channel project types. Final decisions regarding the suitability of a particular method should be made on a case-by-case basis and reviewed and coordinated with Sonoma County Water Agency (Sonoma Water).

Table D.1-1. Suitable Sediment Transport Assessment Methodologies

Project Type	Site/Watershed Assessment	Risk to Life, Property, or Project Investment	Suitable Sediment Analysis and Impact Assessment
Bank stabilization project with no significant change to cross-section, slope, or channel plan form	Relatively stable watershed and site	Low	Expert or professional judgment to assess significance of change in the local hydraulic conditions from pre- to post-project
		Moderate	Assess stable channel grade at design flows; field-check indications of future evolutionary change in channel
		High	Conduct comparison of rating curves above and through site
Channel modification with small change to cross-section, slope, or channel plan form	Moderately active watershed and site	Low	Conduct comparison of rating curves above and through site, as well as pre- and post-project evaluations
Channel modification with significant change to cross-section, slope, or channel plan form	Moderately active watershed and site	Moderate	Develop sediment budget analysis with HEC-RAS stable channel analysis
		High	Conduct sediment transport analysis using long-term numerical simulation with HEC-RAS or other hydraulic model

Source: NRCS 2007

Sediment transport and channel stability analysis can be complex; any analysis beyond the simple erodibility assessment methods described below should be undertaken by a professional engineer or geomorphologist with experience in sediment transport and fluvial geomorphology.

Simplified Methods

Simplified methods for channel stability assessment rely on comparing the erosive forces of streamflow acting on the channel and its banks to the ability of the channel boundary materials to resist these erosive forces.

Critical Velocity

Velocity can be used to represent the erosive force of streamflow for the purposes of stream stability assessment. Critical velocity for a given channel is defined as the streamflow velocity at which erosion begins to take place. Comparing average channel velocity for the design flow to critical velocity can be a simple way to assess channel stability: If average velocity is greater than critical velocity, the channel is likely to be erosive; if it is less than critical velocity, the channel may be stable or depositional. Table D.1-2, "Typical Velocities Required to Entrain Channel Bed and Bank Materials," indicates typical flow velocities required to entrain various channel bed and bank materials.

This method assumes uniform velocity distribution across a cross-section. In reality, velocities vary across a cross-section depending on channel shape, roughness elements, and converging and diverging flow. In addition, initial sediment entrainment velocities can vary significantly for different channel bed and bank materials. For these reasons, comparing average velocity to critical velocity provides only a cursory assessment of channel stability.

Table D.1-2 provides entrainment velocities for various bed and bank materials. This table may be used to evaluate critical velocity for initial screening of alternatives or for qualitative assessments of bed and bank stability at design flows.

Table D.1-2. Typical Velocities Required to Entrain Channel Bed and Bank Materials

Channel Bed or Bank Material Type	**Flow Velocity Required to Entrain Materials**				
	0-2 ft/sec	**2-4 ft/sec**	**4-6 ft/sec**	**6-8 ft/sec**	**> 8 ft/sec**
Sandy soils					
Firm loam					
Mixed gravel and cobbles					
Average turf					
Degradable rolled erosion control products (RECPs)					
Bioengineering					

Channel Bed or Bank Material Type	**Flow Velocity Required to Entrain Materials**				
	0-2 ft/sec	**2-4 ft/sec**	**4-6 ft/sec**	**6-8 ft/sec**	**> 8 ft/sec**
Good turf					
Permanent RECPs					
Armoring					
Gabions					
Riprap					
Concrete					

RECP = rolled erosion control products

Key:

	Appropriate
	Use Caution
	Not Appropriate

Source: Adapted from Fischenich 2001

Critical Shear Stress

Shear stress is the fluid force action on a bed or bank boundary. Similar to the concept of critical velocity, critical shear stress for a given channel is defined as the shear stress at which erosion begins to take place. Comparing the shear stress exerted by the design flow to critical shear stress is a more direct method for simplified assessment of bed and bank stability at design flows.

An estimate of the average boundary shear stress is given by *Equation D.1-1*.

$$\tau_o = \gamma * R * S_f \qquad \text{(\textit{Equation D.1-1})}$$

Where,

τ_o is the average boundary shear stress (pounds per square foot [lb/ft^2]);
γ is the specific weight of water (usually 62.4 pounds per cubic foot [lb/ft^3]);
R is the Hydraulic radius (ft); and
S_f is the Friction slope (ft/ft).

Average cross-section boundary shear stress is also calculated by HEC-RAS and other one-dimensional hydraulic models. In evaluating channel stability, average channel shear stress should be adjusted to account for local variability and instantaneous values that are higher than the average. Where no other information exists, *Equation D.1-2* and *Equation D.1-3* can be used to estimate local maximum shear stress from average boundary shear stress. For straight channels, the local maximum shear stress can be computed from *Equation D.1-2*:

$$\tau_{max} = 1.5 * \tau_O \qquad \text{(Equation D.1-2)}$$

Where,

τ_{max} is the maximum shear stress (lb/ft^2); and

τ_O is the average boundary shear stress (lb/ft^2).

For sinuous channel, the maximum shear stress can be calculated from *Equation D.1-3*:

$$\tau_{max} = 2.65 * \tau_O * \left(\frac{R_c}{W}\right)^{-0.5} \qquad \text{(Equation D.1-3)}$$

Where,

τ_{max} is the maximum shear stress (lb/ft^2);

τ_O is the average boundary shear stress (lb/ft^2);

R_c is the radius of curvature; and

W is the top width of the channel (ft).

Critical shear stress depends on the properties of the channel bed and banks, and varies by material type. When designing an open channel, the design flow maximum shear stress at a cross-section should not exceed the boundary's critical shear stress. Actual boundary shear stresses can vary widely compared to those computed for an average cross-section. Therefore, an appropriate factor of safety should be used for design, in addition to the shear stress adjustments described in *Eqns. D.1-1* through *D.1-4*. The factor of safety used is dependent on the type of channel, proximity to infrastructure or buildings, and maintenance concerns. For typical projects, a factor of safety between 1.2 and 1.5 is usually appropriate. When evaluating the stability of an open channel for channel design purposes, use *Equation D.1-4* with an appropriate factor of safety.

$$\tau_{cr} \geq FS * \tau_{max} \qquad \text{(Equation D.1-4)}$$

Where,

τ_{cr} is the critical shear stress (lb/ft^2);

τ_{max} is the maximum shear stress (lb/ft^2);and

FS is the factor of safety.

Table D.1-3, "Critical Shear Stress for Selected Channel Materials," provides critical shear stress values for various bed and bank materials.

Table D.1-3. Critical Shear Stress for Selected Channel Materials

Material Category	Material Type	Critical Shear Stress (τ_{cr}) (lb/ft^2)
Soils	Fine colloidal sand	0.02
	Sandy loam (noncolloidal)	0.03
	Alluvial silt (noncolloidal)	0.045
	Silty loam (noncolloidal)	0.045
	Firm loam	0.075
	Fine gravels	0.075
	Stiff clay	0.26
	Alluvial silt (colloidal)	0.26
	Graded loam to cobbles	0.38
	Graded silts to cobbles	0.43
	Shales and hardpan	0.67
Gravel/Cobble	1-in	0.33
	2-in	0.67
	6-in	2
	12-in	4
Vegetation	Class A turf*	3.7
	Class B turf*	2.1
	Class C turf*	1
	Long native grasses	1.2
	Short native and bunch grass	0.7
	Reed plantings	0.1
	Hardwood tree plantings	0.41
Temporary Degradable RECPs	Jute net	0.45
	Straw with net	1.5
	Coconut fiber with net	2.25
	Fiberglass roving	2
Non-degradable RECPs	Unvegetated	3
	Partially established	4
	Fully vegetated	8

Material Category	**Material Type**	**Critical Shear Stress (τ_{cr}) (lb/ft^2)**
Riprap	6-in. d50	2.5
	9-in. d50	3.8
	12-in. d50	5.1
	18-in. d50	7.6
	24-in. d50	10.1
Soil Bioengineering	Wattles	0.2
	Reed fascine	0.6
	Coir roll	3-5
	Vegetated coir mat	4-8
	Live brush mattress (initial)	0.4
	Live brush mattress (grown)	3.90
	Brush layering (initial/grown)	0.4 – 6.25
	Live fascine	1.25
	Live willow stakes	2.10
Hard Surfacing	Gabions	10
	Concrete	12.5

** Turf classes are defined in Table 8-7 of NRCS 2007.*

Notes:
RECPs = rolled erosion control products.
d50 = median grain size (50th percentile) whereby half of the sediments have a diameter smaller than this size, and half a diameter greater.

Source: Adapted from Fischenich 2001; see also NRCS 2007.

Appendix D.2.

Figures and Tables

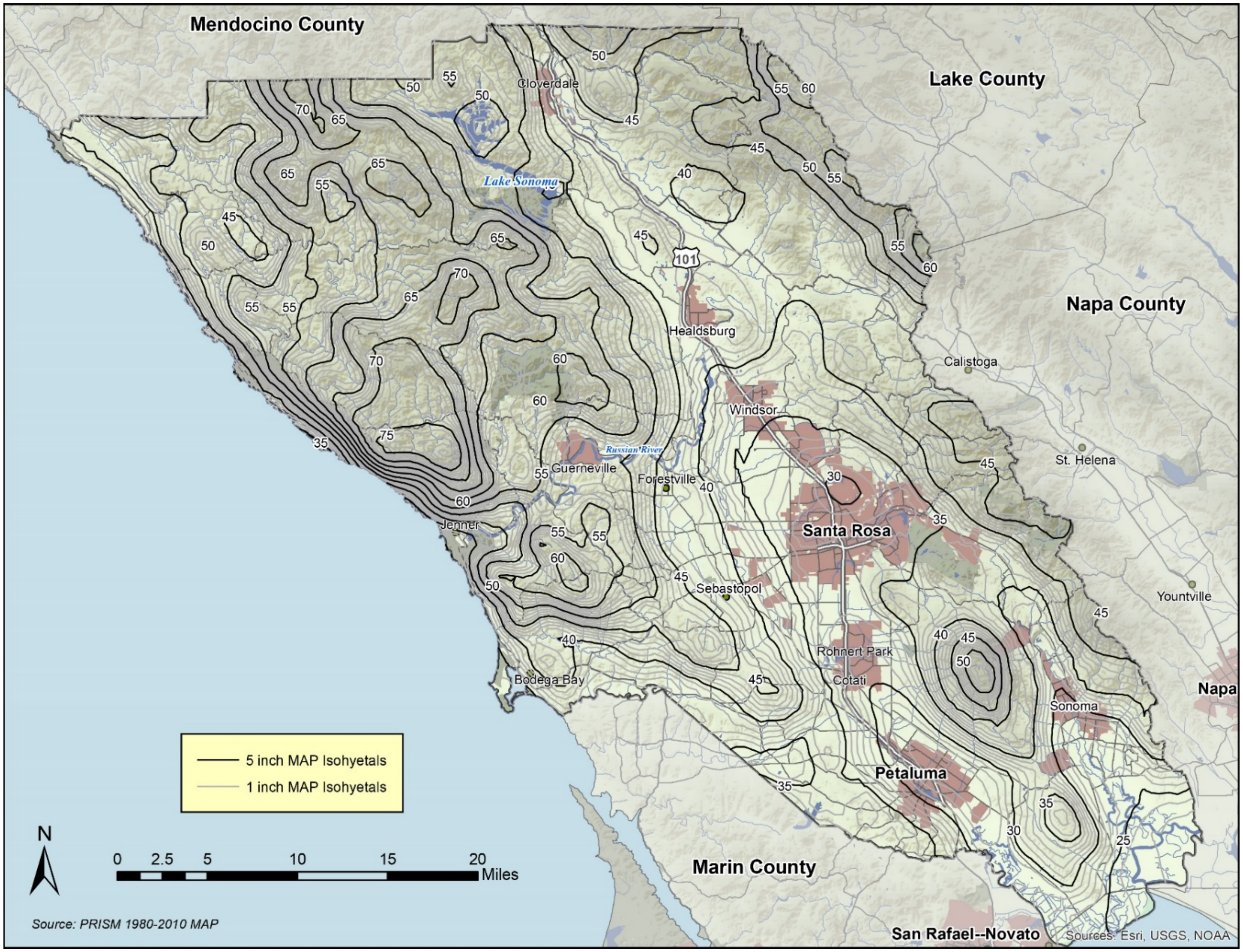
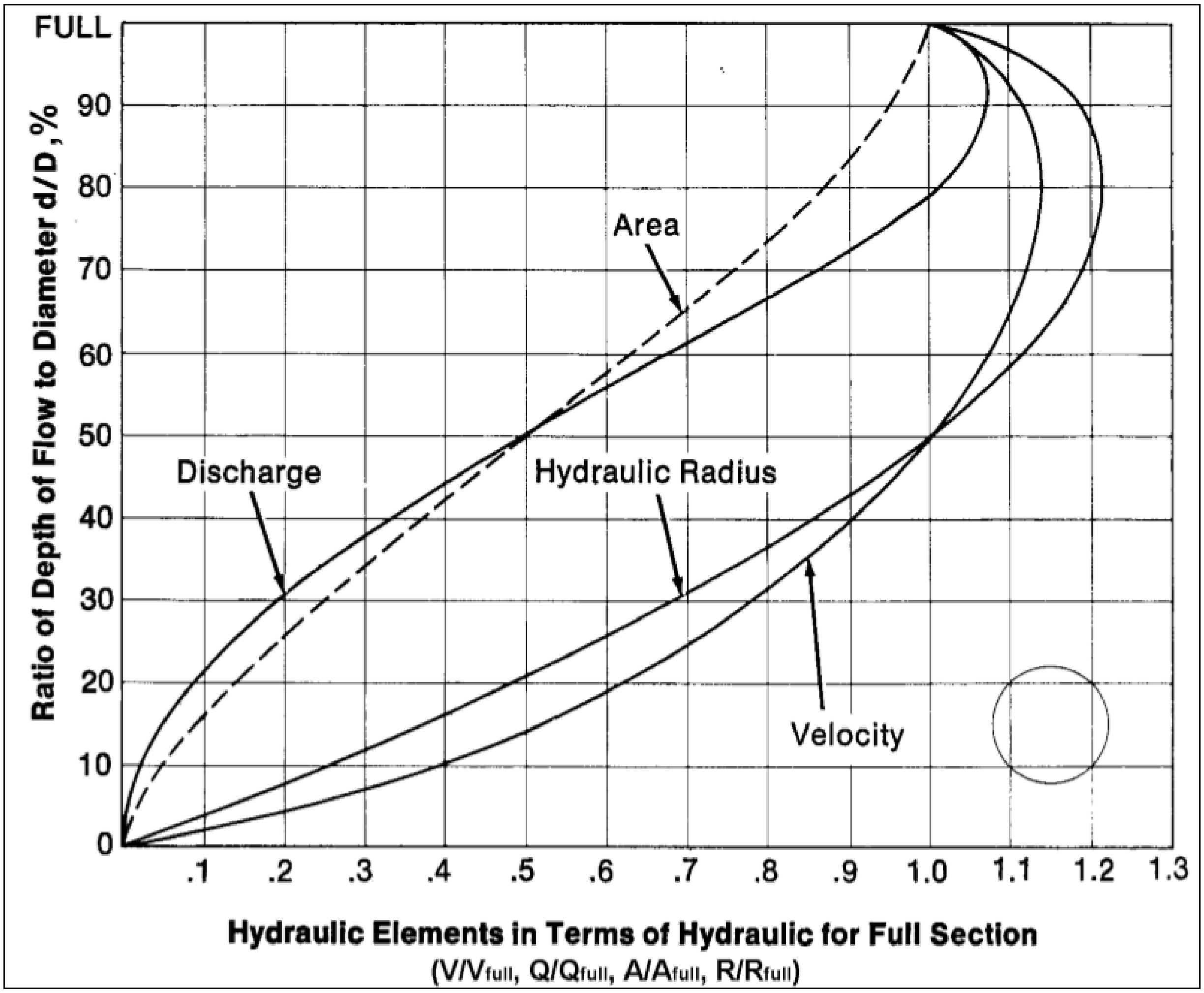
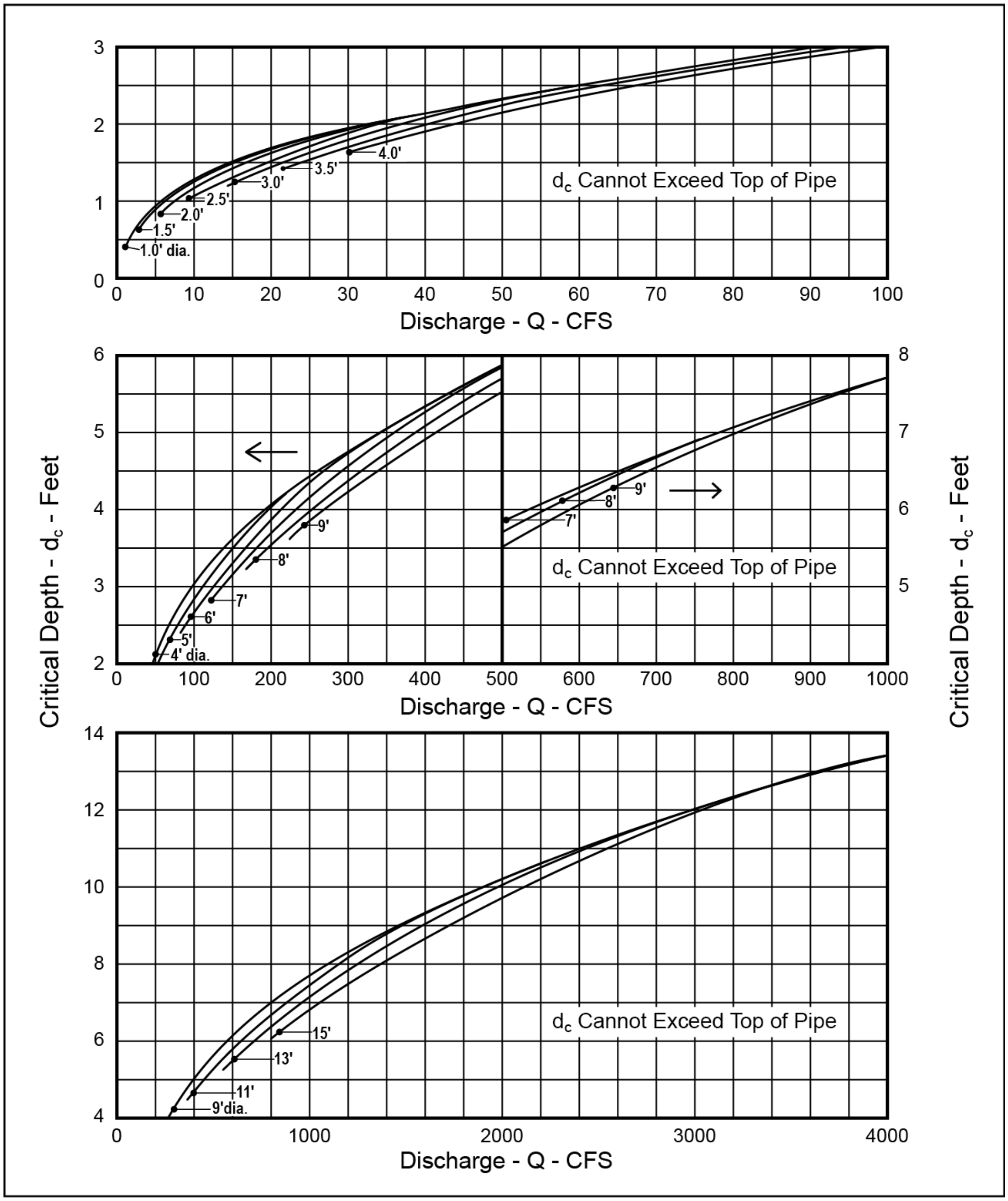


Figure D.2-1. Mean Annual Precipitation



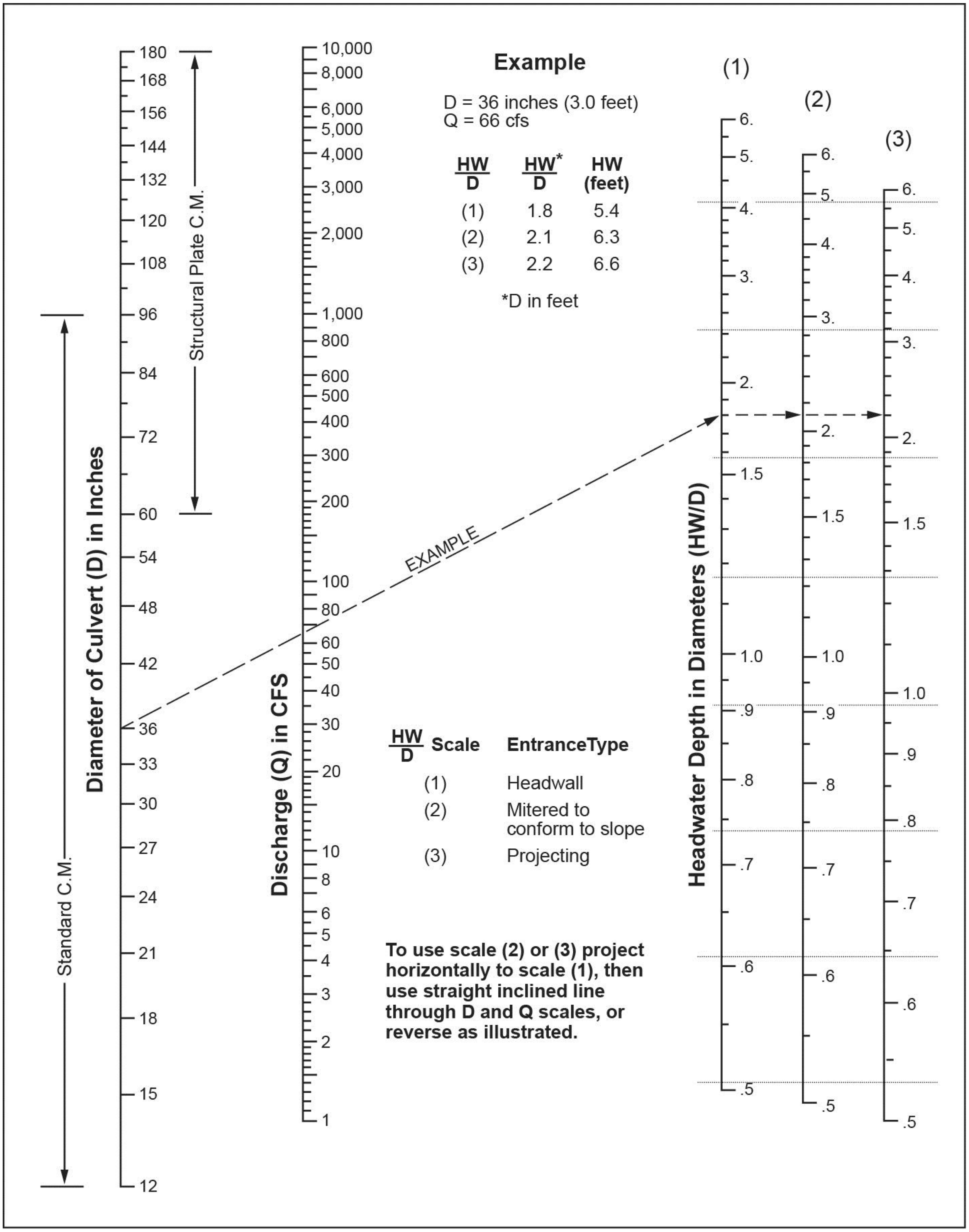
Source: FHWA 2008

Figure D.2-2 Hydraulic Elements for Circular Pipe



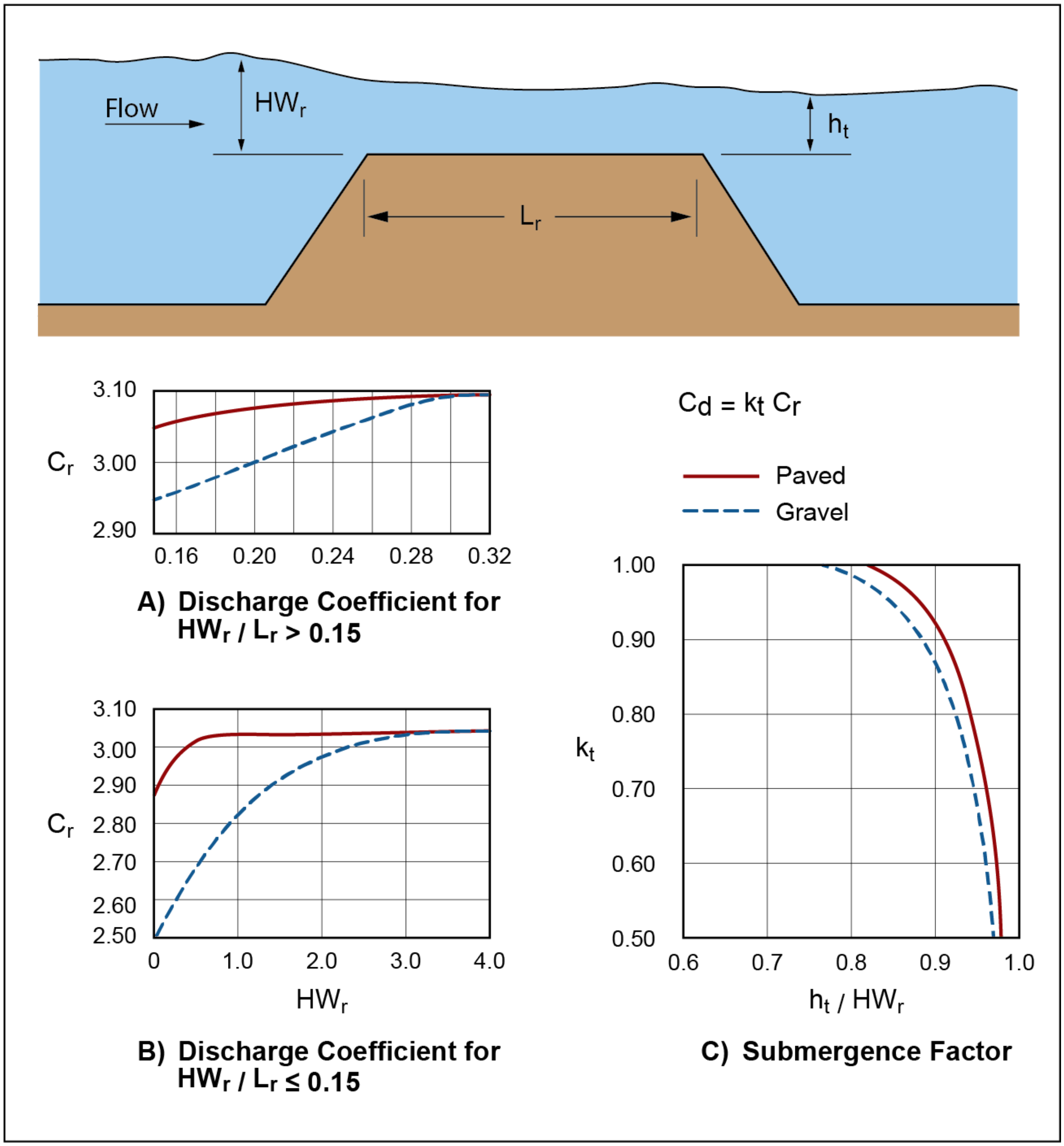
Source: FHWA 2012, based on Bureau of Public Roads 1964

Figure D.2-3. Critical Depth – Circular Pipe



Source: FHWA 2008, Hydraulic Design of Highway Culverts 3rd Ed. FHWA-HIF-12-026 HDS 5 April 2012 Chart 2B

Figure D.2-4 Hydraulic Elements for Circular Pipe



Source: FHWA 2012

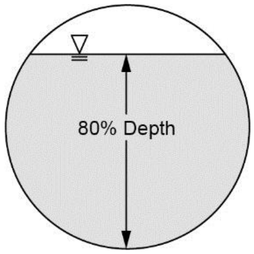
Figure D.2-5 Discharge Coefficients for Roadway Overtopping (English units)

Table D.2-1. Manning's Roughness Coefficients for Channels

Channel Material	Manning's Roughness (*n*)
Concrete, steel troweled, or smooth-form finish	0.014
Concrete, wood float or broomed finish, including pneumatically applied mortar	0.015
Asphaltic concrete	0.017
Sack concrete riprap	0.030
Grouted rock riprap	0.030
Loose rock riprap	0.035
Grass channels	0.035
Constructed natural waterways	0.050 minimum, under typical conditions*
Other natural channels	See references: 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, US Geological Survey, 67 pp. Available at: pubs.er.usgs.gov/publication/wsp2339. 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.

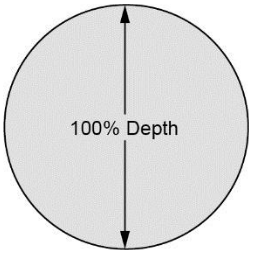
* Sonoma Water may be willing to allow values as low as 0.035 for channels that are well maintained and hydraulically smooth.

Table D.2-2. Slopes Necessary to Maintain Minimum Velocity for 80% Depth Flow (V = 2.5 ft/sec)



Pipe Diameter (inches)	Slope, %		
	n = 0.012	**n = 0.014**	**n = 0.024**
36	0.046	0.063	0.184
42	0.037	0.051	0.150
48	0.031	0.043	0.125
54	0.027	0.036	0.107
60	0.023	0.032	0.093
66	0.021	0.028	0.082
72	0.018	0.025	0.073
84	0.015	0.020	0.060

Table D.2-3. Slopes Necessary to Maintain Minimum Velocity for Full Pipe Flow (V = 2.5 ft/sec)



Pipe Diameter (inches)	Slope, %		
	n = 0.012	n = 0.014	n = 0.024
3	1.643	2.237	6.573
4	1.120	1.524	4.479
6	0.652	0.888	2.608
8	0.444	0.605	1.777
10	0.330	0.449	1.320
12	0.259	0.352	1.035
15	0.192	0.262	0.769
18	0.151	0.205	0.603
24	0.103	0.140	0.411
30	0.076	0.104	0.305
36	0.060	0.081	0.239
42	0.049	0.066	0.195
48	0.041	0.055	0.163
54	0.035	0.047	0.139
60	0.030	0.041	0.121
66	0.027	0.036	0.107
72	0.024	0.032	0.095
84	0.019	0.026	0.077

Table D.2-4. Manning's Roughness Coefficients for Closed Conduits and Culverts

Conduit/Culvert Material	**Manning's *n***
Concrete	
Smooth forms	0.014
Rough	0.015-0.017
Corrugated-metal pipe -- (2-1/2 in x 1/2 in corrugations)	
Plain	0.022-0.026
Paved invert	0.018-0.022
Spun asphalt lined	0.012-0.015
Plastic pipe (smooth)	0.012-0.015
Corrugated-metal pipe -- (2-2/3 in x 1/2 in annular)	0.022-0.027
Corrugated-metal pipe -- (2-2/3 in x 1/2 in helical)	0.012-0.023
Corrugated-metal pipe -- (6 in x 1 in helical)	0.022-0.025
Corrugated-metal pipe -- (5 in x 1 in helical)	0.025–0.026
Corrugated-metal pipe -- (3 in x 1 in helical)	0.027–0.028
Corrugated-metal pipe -- (6 in x 2 in structural plate)	0.033–0.035
Corrugated-metal pipe -- (9 in x 2-1/2 in structural plate)	0.033–0.037
Polyethylene	
Smooth	0.012-0.015
Corrugated	0.018–0.025
Spiral rib metal pipe (smooth)	0.012-0.013
Polyvinyl chloride (PVC) (smooth)	0.012-0.011

Sources: ASCE 1982, FHWA 2001; modified to show minimum values of 0.012-0.014.

Table D.2-5. Values of k_e for Estimating Head Loss due to Sudden Pipe Enlargement

D_2/D_1	Velocity (V_1), ft/sec												
	2.0	**3.0**	**4.0**	**5.0**	**6.0**	**7.0**	**8.0**	**10**	**12**	**15**	**20**	**30**	**40**
1.2	0.11	0.10	0.10	0.10	0.10	0.10	0.10	0.09	0.09	0.09	0.09	0.09	0.08
1.4	0.26	0.26	0.25	0.24	0.24	0.24	0.24	0.23	0.23	0.22	0.22	0.21	0.20
1.6	0.40	0.39	0.38	0.37	0.37	0.36	0.36	0.35	0.35	0.34	0.33	0.32	0.32
1.8	0.51	0.49	0.48	0.47	0.47	0.46	0.46	0.45	0.44	0.43	0.42	0.41	0.40
2.0	0.60	0.58	0.56	0.55	0.55	0.54	0.53	0.52	0.52	0.51	0.50	0.48	0.47
2.5	0.74	0.72	0.70	0.69	0.68	0.67	0.66	0.65	0.64	0.63	0.62	0.60	0.58
3.0	0.83	0.80	0.78	0.77	0.76	0.75	0.74	0.73	0.72	0.70	0.69	0.67	0.65
4.0	0.92	0.89	0.87	0.85	0.84	0.83	0.82	0.80	0.79	0.78	0.76	0.74	0.72
5.0	0.96	0.93	0.91	0.89	0.88	0.87	0.86	0.84	0.83	0.82	0.80	0.77	0.75
10.0	1.00	0.99	0.96	0.95	0.93	0.92	0.91	0.89	0.88	0.86	0.84	0.82	0.80
∞	1.00	1.00	0.98	0.96	0.95	0.94	0.93	0.91	0.90	0.88	0.86	0.83	0.81

D_2/D_1 = ratio of diameter of larger pipe to smaller pipe

V_1 = velocity in smaller pipe (upstream of transition)

Source: ASCE 1992

Table D.2-6. Values of k_e for Gradual Enlargement of Pipes in Non-pressure Flow

D_2/D_1	Angle of Cone						
	10°	**20°**	**45°**	**60°**	**90°**	**120°**	**180°**
1.5	0.17	0.40	1.06	1.21	1.14	1.07	1.00
3	0.17	0.40	0.86	1.02	1.06	1.04	1.00

Source: FHWA 2009

Table D.2-7. Values of k_c for Determining Loss of Head due to Sudden Contraction

D_2/D_1	Velocity (V_1), ft/sec												
	2.0	**3.0**	**4.0**	**5.0**	**6.0**	**7.0**	**8.0**	**10.0**	**12.0**	**15.0**	**20.0**	**30.0**	**40.0**
1.1	0.03	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.05	0.05	0.06
1.2	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.09	0.10	0.11
1.4	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.18	0.18	0.18	0.18	0.19	0.20
1.6	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.25	0.25	0.24
1.8	0.34	0.34	0.34	0.34	0.34	0.34	0.33	0.33	0.32	0.32	0.32	0.29	0.27
2.0	0.38	0.38	0.37	0.37	0.37	0.37	0.36	0.36	0.35	0.34	0.33	0.31	0.29
2.2	0.40	0.40	0.40	0.39	0.39	0.39	0.39	0.38	0.37	0.37	0.35	0.33	0.30
2.5	0.42	0.42	0.42	0.41	0.41	0.41	0.40	0.40	0.39	0.38	0.37	0.34	0.31
3.0	0.44	0.44	0.44	0.43	0.43	0.43	0.42	0.42	0.41	0.40	0.39	0.36	0.33
4.0	0.47	0.46	0.46	0.46	0.45	0.45	0.45	0.44	0.43	0.42	0.41	0.37	0.34
5.0	0.48	0.48	0.47	0.47	0.47	0.46	0.46	0.45	0.45	0.44	0.42	0.38	0.35
10.0	0.49	0.48	0.48	0.48	0.48	0.47	0.47	0.46	0.46	0.45	0.43	0.40	0.36
∞	0.49	0.49	0.48	0.48	0.48	0.47	0.47	0.47	0.46	0.45	0.44	0.41	0.38

D_2/D_1 = ratio of diameter of larger pipe to smaller pipe
V_1 = velocity in smaller pipe (downstream of transition)
Source: ASCE 1992

Table D.2-8. Entrance Loss Coefficients

Channel Material	K_e
Pipe, concrete	
Projecting from fill, socket end	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove end)	0.2
Square-edge	0.5
Rounded (radius = 1/12 barrel dimension)	0.2
Mitered to conform to fill slope	0.7
End section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° levels	0.2
Side- or slope-tapered inlet	0.2
Pipe, or pipe-arch, corrugated metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls, square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
End section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° levels	0.2
Side- or slope-tapered inlet	0.2
Box, reinforced concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of 1/12 barrel dimension or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of 1/12 barrel dimension, or beveled top edge	0.2
Wingwall at 10° to 25° to barrel, square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

Source: FHWA 2012

Appendix E.
Example Problems

Introduction

The following example design problems are associated with the development of the Blue Heron Subdivision. Two drainages flow through the development, Big Creek from the west and Rock Creek from the north.

Example Problem 1 Size a culvert for Rock Creek using the Incremental Rational Method – page E.1-1

Example Problem 2 Size a culvert for Big Creek using a Synthetic Unit Hydrograph – page E.2-1

Example Problem 3 Design new stormwater conveyance pipe for Blue Heron (South) using full-flowing pipes – page E.3-1

Each example problem is followed by attachments depicting the figures and calculations that should be included in a complete submittal to the Sonoma County Water Agency (Sonoma Water).

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Appendix E.1

Example Problem 1

Example Problem 1
Design a Culvert for Rock Creek Using the Incremental Rational Method

The Rock Creek watershed encompasses 41.6 acres (0.07 square mile [mi^2]) and drains into a culvert located at the edge of a residential development as shown in Attachment E.1-1, located at the end of this narrative. This example problem determines the culvert diameter required to contain the minimum design flow, as well as the 100-year peak flow, using the Incremental Rational Method. Tabulated calculations are included in Attachments E.1-2 and E.1-3, at the end of Appendix E.1.

Table 3-2, "Minimum Design Flows," in Chapter 3 is used to determine the minimum design flow based on the watershed area. Based on drainage area of less than 1 mi^2, the minimum design flow is the 10-year peak flow.

Step 1. Delineate the drainage area of Rock Creek.

The Rock Creek drainage area or watershed was hand delineated from the topographic contours as shown in Attachment E.1-1.

Step 2. Delineate the flow path(s).

The flow path is delineated in Attachment E.1-1.

Step 3. Subdivide the total drainage area into sub-basins.

The watershed was divided into four distinct sub-basins as shown in Attachment E.1-1. The most upstream sub-basin is assumed to be characterized by overland flow and therefore should be a maximum of 2 acres in size, based on Table 3-3, "Estimated Overland Flow Travel Time," in Chapter 3.

Determine the following characteristics for the flow path segment in each sub-basin: length, upstream elevation, and downstream elevation.

Determine the slope of the flow path in each sub-basin using the flow path length and elevations found at the upstream and downstream limits of the flow path in each sub-basin:

$$S = \frac{US - DS}{L}$$

Where,

S is the slope of the flow path (feet/foot [ft/ft]);
US is the upstream elevation of the flow path (ft);
DS is the downstream elevation of the flow path (ft);
L is the length of flow path (ft).

A summary of these sub-basin characteristics and estimated flow path slope is included in Table E.1-1, “Rock Creek Drainage Basin Characteristics,” presented below.

Table E.1-1. Rock Creek Drainage Basin Characteristics

Sub-Basin	Area (acres)	Flow Path Length (ft)	Flow Path Upstream Elevation	Flow Path Downstream Elevation	Flow Path Slope (ft/ft)
Sub-Basin A	1.9	420	475	442	0.0786
Sub-Basin B	11.7	770	442	418	0.0312
Sub-Basin C	14.1	560	418	398	0.0357
Sub-Basin D	13.9	760	398	388	0.0132

Step 4. Identify the time of concentration for the most upstream contributing area, Sub-Basin A.

Hydrology in the most upstream contributing area, in this example, Sub-Basin A, is typically characterized by overland flow. Table 3-3 indicates that for areas greater than ½ acre and less than 2 acres, the time of concentration (t_c) is assumed to be 15 minutes (min); therefore, the time of concentration in Sub-Basin A is 15 min.

t_A = 15 min

Step 5. Select the appropriate Runoff Coefficient for Sub-Basin A and calculate the peak flow for computation Point 1.

Select the appropriate runoff coefficient *C* value using Table C-1, “Runoff Coefficients (*Cs*) (Incremental Rational Method),” found in Appendix C.

<u>Sub-Basin A</u>

1.9 acres Type C Soils – Ag and open space with an average 8 percent slope

Based on Table C-1:

C_A = 0.43

To calculate the peak flow at Point 1, located at the downstream limit of Sub-Basin A, it is necessary to identify the rainfall intensity (inches/hour [hr]) for the design rainfall event.

Rainfall intensity (inches/hr) is determined using the precipitation data source, NOAA Atlas 14, described in Chapter 3, Section 3.3.3 and accessed at: hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=ca.

Rainfall intensity data for a watershed can be found by supplying a latitude and longitude, the name of a nearby meteorological station, or a street address. Additional information about rainfall data access is attached in Attachment E.1-4.

The minimum design flow and the 100-year flow must be included in every project submittal. According to Table 3-2, the minimum design flow for this example's watershed is the 10-year peak flow, which is assumed to be produced by a storm with an average recurrence interval of 10 years. For this example, calculations for only the minimum design flow (the 10-year peak flow) will be presented, however the tabulated values for the 100-year flow can be found in Attachment E.1-3. The NOAA rainfall data for Rock Creek project site is included in Table E.1-2, "Rock Creek NOAA Atlas 14 Rainfall Data," and Attachment E.1-4.

Table E.1-2. Rock Creek NOAA Atlas 14 Rainfall Data

Rainfall Duration (min)	Rainfall Intensity for 10 Year Recurrence Interval (inches/hr)	Rainfall Intensity for 100 Year Recurrence Interval (inches/hr)
5	3.32	5.26
10	2.39	3.77
15	1.92	3.04
30	1.35	2.13
60	0.96	1.52

The time of concentration (t_c) for Sub-Basin A was identified in **Step 4** as 15 min. The time of concentration (t_c) is assumed to be equal to the rainfall duration; therefore, the precipitation intensity for a recurrence interval of 10 years is 1.92 inches/hr (Attachment E.1-2).

The peak flow for Point 1 can be calculated using the rational formula (*Equation 3.1*)

$$Q = C * I * A \qquad (\textit{Equation 3.1})$$

$$Q_1 = C_A * I_A * A_A$$

Where,

Q_1 is the peak flow rate for Point 1 (cubic feet/second [cfs]);
C_A is the runoff coefficient for sub-basin A (dimensionless);
A_A is the watershed or sub-watershed area (acres) for sub-basin A; and
I_A is the precipitation intensity (inches/hr) for the design rainfall event for sub-basin A.

C_A = 0.43
I_A = 1.92 inches/hr

A_A = 1.9 acres

Q_1 = 0.43 * 1.92 * 1.9
Q_1 = 1.6 cfs

Step 6. Estimate travel time through Sub-Basin B, the time of concentration at computation Point 2 and the corresponding rainfall intensity.

In Sub-Basin B, there is no defined channel visible in Attachment E.1-1, "Rock Creek Watershed Sub-Basins." Due to the lack of defined channel, we assume that water travels across Sub-Basin B as shallow concentrated flow. The velocity of shallow concentrated flow over unpaved ground is calculated using *Equation 3.6*.

$$V = 16.13 * S^{0.5} \qquad \text{(\textit{Equation 3.6})}$$

Where,

V is the velocity of the flow (ft/sec);
S is the slope of the flow path (ft/ft).

$$V_B = 16.13 * S_B^{0.5}$$

S_B = 0.0312 ft/ft (calculated in **Step 3**)

V_B = 16.13 * (0. 0312)$^{0.5}$
V_B = 2.8 ft/sec

Travel time is calculated with *Equation 3.10*.

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

Where,

t_p is the travel time (min);
L_p is the flow path length (ft); and
V is the flow velocity (ft/sec).

$$t_B = \frac{L_B}{V_B * 60}$$

L_B = 770 ft

V_B = 2.8 ft/sec

t_B = 770 / (3.1 * 60)

t_B = 4.5 min

The time of concentration for Point 2 (t_2), located at the downstream limit of Sub-Basin B, is the sum of the travel time in Sub-Basin A (t_A) found in **Step 4** and the travel time in Sub-Basin B (t_B).

$$t_2 = t_A + t_B$$

t_A = 15.0 min

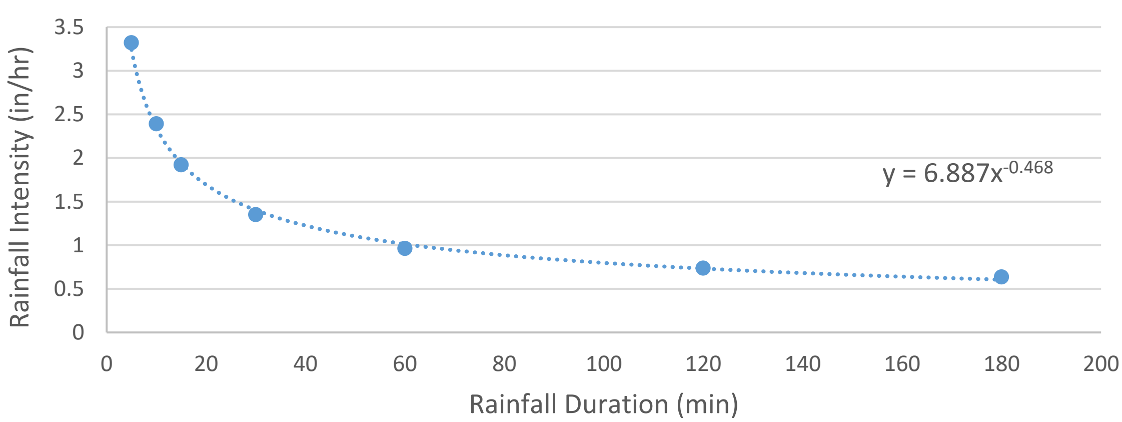
t_B = 4.5 min

t_2 = 15.0 + 4.5

t_2 = 19.5 min

The time of concentration for Point 2 (t_2) is used to identify the rainfall intensity (inches/hr) for the Sub-Basin B. It may be necessary to interpolate rainfall intensity from the tabulated values included in Table E.1-2. Using Microsoft Excel or a similar program, plot rainfall duration and intensity for the 10-year recurrence interval. Insert a power trendline (line of best fit) and display the equation on the chart, as is shown in Figure E.1-1 below.

Figure E.1-1. Trendline of Rainfall Data from Attachment E.1-4.



For this example problem, the equation of the power trendline is:

$$I = 6.887 * t_c^{-0.468}$$

Where,

I is the precipitation intensity (inches/hr); and
t_c is the time of concentration, which is equivalent to rainfall duration (min).

$$I_B = 6.887 * t_2^{-0.468}$$

t_2 = 19.5 min

$I_B = 6.887 * 19.5^{-0.468}$
I_B = 1.71 inches/hr

Step 7. Select an appropriate *C* value for Sub-Basin B and estimate the cumulative *C* * *A* value for computation Point 2.

Select the appropriate *C* value using Table C-1.

Sub-Basin B – 11.7 acres total

4.8 acres Type C Soils – Ag and open space with an average 12.5 percent slope
6.9 acres Type C Soils – Ag and open space with an average 9 percent slope

Based on Table C-1:

4.8 acres with a C = 0.47
6.9 acres with a C = 0.43

Calculate the Composite *C* for the sub-basin from *Equation 3.2*.

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

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.

Sub-Basin B

$$C_B = \frac{(4.8*0.47)+(6.9*0.43)}{11.7}$$

C_B = 0.45

The C * A value of Point 2 is found using information from **Step 5**.

$$C_2 * A_2 = (C_A * A_A) + (C_B * A_B)$$

C_A = 0.43
A_A = 1.9 acres
C_B = 0.45
A_B = 11.7 acres

C_2 * A_2 = 0.43 * 1.9 + 0.45 * 11.7
C_2 * A_2 = 0.8 + 5.3
C_2 * A_2 = 6.1

Step 8. Calculate the peak flow for computation Point 2.

To calculate the peak flow for Point 2 using the rational formula (*Equation 3.1*), utilize C_2 * A_2 determined in **Step 7** and the rainfall intensity determined in **Step 6.**

$$Q_2 = C_2 * I_2 * A_2$$

Where,

Q_2 is the peak flow rate at Point 2 (cfs);
C_2 is the runoff coefficient (dimensionless);
I_2 is the rainfall intensity (inches/hr) for the design rainfall event; and
A_2 is the watershed or sub-watershed area (acres).

$$Q_2 = (C_2 * A_2) * I_2$$

C_2 * A_2 = 6.1
I_2 = 1.71 inches/hr

Q_2 = 6.1 * 1.71
Q_2 = 10.4 cfs

Step 9. Repeat Steps 6, 7, and 8 for the downstream sub-basins.

A defined channel forms downstream of Point 2, as shown in Attachment E.1-1. When a channel is visible on an aerial photograph or topographic map, Manning's equation, *Equation 3.8*, should be used to calculate flow velocity.

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

(*Equation 3.8*)

Where,

V is the velocity (ft/sec);
n is the Manning's roughness coefficient (dimensionless);
R_h is the hydraulic radius (ft); and
S is the pipe slope(ft/ft).

The hydraulic radius (R_h) is defined by *Equation 3.9*.

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

(*Equation 3.9*)

Where,

A is the area of the channel (ft^2); and
P_w is the wetted perimeter of the channel (ft).

Assume that Sub-Basin C and D channel geometry and characteristics have been verified through field observations and flow travels across both sub-basins in a trapezoidal channel with a 5-ft bottom width, 2H:1V side slopes, and a Manning's n of 0.05, which is representative of a constructed natural waterway (Appendix D.2, Table D.2-1).

Select an estimate of water depth and calculate the associated flow. Repeat the procedure to find the water depth that produces a discharge that most closely approximates the flow calculated at Point 2 (10.8 cfs).

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

n = 0.05 (from Table D.2-1)
S = 0.0357 ft/ft (from Table E.1-1 above and Attachment E.1-2)

If assumed flow depth = 6 inches:

A = 3.0 ft^2
P_w = 7.2 ft

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

R_h = 0.41 ft

Q = 3.0 * (1.49/0.05) * 0.41$^{2/3}$ * 0.0357$^{1/2}$
Q = 9.3 cfs

If assumed flow depth = 7 inches:

A = 3.6 ft^2

P_w = 7.6 ft

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

R_h = 0.47 ft

Q = 3.6 * (1.49/0.05) * 0.47$^{2/3}$ * 0.0357$^{1/2}$
Q = 12.3 cfs

A flow depth between 6 and 7 inches results in a range of flows (9.3 to 12.3 cfs) that contains the flow calculated at Point 2 (10.4 cfs).

Calculate the velocity of water using *Equation 3.8* and the conservative end of the range of flow depths found above (7 inches).

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

R_h = 0.47 ft

V_C = (1.49/0.05) * 0.47$^{2/3}$* 0.0357$^{1/2}$
V_C = 3.4 ft/sec

Determine the travel time across Sub-Basin C using the velocity and the flow path length (*Equation 3.10*).

$$t_C = \frac{L_C}{V_C * 60}$$ (*Equation 3.10*)

L_C = 575 ft
V_C = 3.4 ft/sec

t_C = 560 / (3.4* 60)
t_C = 2.7 min

Calculate the time of concentration to Point 3 by adding the upstream travel times.

$$t_3 = t_2 + t_C$$

t_2 = 19.5 min
t_C = 2.7 min

t_3 = 19.5 + 2.7
t_3 = 22.2 min

Use the trendline equation found in **Step 6** to interpolate between tabulated rainfall duration and intensity data (Table E.1-2).

$$I_C = 6.887 * t_3^{-0.468}$$

t_3 = 22.2 min

$I_C = 6.887(22.2)^{-0.468}$
I_C = 1.61 inches/hr

Select the appropriate C value using Table C-1, "Runoff Coefficients for Incremental Rational Formula," found in Appendix C.

Sub-Basin C – 14.1 acres total

2.5 acres Type C Soils – Ag and open Space with an average 7.5 percent slope
8.6 acres Type C Soils – Ag and open Space with an average 5 percent slope
3.0 acres Type B Soils – Ag and open Space with an average 5 percent slope

Based on Table C-1:

2.5 acres with a C = 0.43
8.6 acres with a C = 0.38

3.0 acres with a $C = 0.28$

Calculate the Composite C for the sub-basin from *Equation 3.2*.

$$C_c = \frac{(2.5 * 0.43) + (8.6 * 0.38) + (3.0 * 0.28)}{14.1}$$

$C_C = 0.37$

The $C * A$ value for Point 3 is the sum of the $C * A$ value for Point 2 and the $C * A$ value for Sub-Basin C.

$$C_3 * A_3 = (C_2 * A_2) + (C_C * A_C)$$

$C_2 * A_2 = 6.1$ acres
$C_C = 0.37$
$A_C = 14.1$ acres

$C_3 * A_3 = 6.1 + (0.37 * 14.1)$
$C_3 * A_3 = 6.1 + 5.2$
$C_3 * A_3 = 11.3$

Peak flow at Point 3 is found using *Equation 3.1*.

$$Q_3 = (C_3 * A_3) * I_3$$

$C_3 * A_3 = 11.3$
$I_3 = 1.61$ inches/hr

$Q_3 = 11.3 * 1.61$
$Q_3 = 18.2$ cfs

The Sub-Basin D channel has the same geometry and Manning's value as the channel in Sub-Basin C.

Find the water depth that produces a value for discharge that most closely approximates the discharge calculated at Point 3 (18.6 cfs).

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

$n = 0.05$ (from Table D.2-1)
$S = 0.0132$ ft/ft (from Table E.1-1 and Attachment E.1-2)

Assumed flow depth = 11 inches:

$A = 6.3\ ft^2$
$P_w = 9.1$ ft
$R_h = 0.69$ ft

$Q = 6.3 * (1.49/0.05) * 0.69^{2/3} * 0.0132^{1/2}$
$Q = 16.8$ cfs

Assumed flow depth =12 inches:

$A = 7.0\ ft^2$
$P_w = 9.5$ ft
$R_h = 0.74$ ft

$Q = 7.0 * (1.49/0.05) * 0.74^{2/3} * 0.0132^{1/2}$
$Q = 19.6$ cfs

A flow depth between 11 and 12 inches results in a range of flows (16.8 to 19.6 cfs) that contains the flow calculated at Point 3 (18.2 cfs).

Calculate the flow velocity using the conservative end of the range of flow depths found above (12 inches).

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

$n = 0.05$
$S = 0.0132$ ft/ft (**Step 3**)

$V_D = (1.49/0.05) * 0.74^{2/3} * 0.0132^{1/2}$
$V_D = 2.8$ ft/sec

Determine the travel time using the velocity and the flow path length (*Equation 3.10*). Calculate the time of concentration by summing the travel times to the point of interest.

$$t_D = \frac{L_D}{V_D * 60}$$

$L_D = 760$ ft
$V_D = 2.8$ ft/sec

t_D = 760 / (2.8 * 60)
t_D = 4.5 min

Calculate the time of concentration to Point 4 by adding the upstream travel times.

$$t_4 = t_3 + t_D$$

t_3 = 22.2 min
t_D = 4.5 min

t_4 = 22.2 + 4.5
t_4 = 26.8 min

Use the trendline equation found in **Step 6** to interpolate between tabulated rainfall duration and intensity data (Figure E.1-1).

$$I_D = 6.887 * t_4^{-0.468}$$

t_4 = 26.8 min

$I_D = 6.887(26.8)^{-0.468}$
I_D = 1.48 inches/hr

Sub-Basin D – 13.9 acres total

3.3 acres Type C Soils – Ag and open Space with an average 1.5 percent slope
9.1 acres Type C Soils – Ag and open Space with an average 5 percent slope
1.5 acres Type B Soils – Ag and open Space with an average 4 percent slope

Based on Table C-1:

3.3 acres with a C = 0.33
9.1 acres with a C = 0.38
1.5 acres with a C = 0.28

Calculate the Composite C for the sub-basin from *Equation 3.2*.

$$C_D = \frac{(3.3 * 0.33) + (9.1 * 0.38) + (1.5 * 0.28)}{13.9}$$

C_D = 0.36

The $C * A$ value for Point 4 is the sum of the $C * A$ value for Point 3 and the $C * A$ value for Sub-Basin D.

$$C_4 * A_4 = (C_3 * A_3) + (C_D * A_D)$$

$C_3 * A_3 = 11.3$
$C_D = 0.36$
$A_D = 13.9$ acres

$C_4 * A_4 = 11.3 + 0.36 * 13.9$
$C_4 * A_4 = 11.3 + 5.0$
$C_4 * A_4 = 16.3$

Peak flow at Point 4 is found using *Equation 3.1*.

$$Q_4 = (C_4 * A_4) * I_3$$

$C_4 * A_4 = 16.3$
$I_4 = 1.48$ inches/hr

$Q_4 = 16.3 * 1.48$
$Q_4 = 24.1$ cfs

The 10-year peak flow at Point 4, located at the entrance of the culvert shown in Attachment E.1-1 at downstream extent of Sub-Basin D, is 24.1 cfs. Tabulated calculations are shown in Attachment E.1-2.

The 100-year peak flow at Point 4 is 38.2 cfs. Calculations are shown in Attachment E.1-3.

Step 10. Size the culvert for the peak flow.

For this example, assume the culvert is a 3-ft corrugated metal pipe (CMP) with a Manning's Roughness Coefficient of 0.024 (from Appendix D.2, Table D.2-4), at a 3 percent slope with no tail water.

Calculate the full capacity of the pipe, using a modification of the previously defined Manning's Equation (*Equation 3.8*).

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

(*Equation 3.8*)

Flow (Q) is the product of velocity and area (Q = V * A). By multiplying both sides of *Equation 3.8* by the area of the pipe, we can solve for flow.

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

n = 0.024
A = 7.1 ft^2
P_w = 9.4 ft
R_h = 0.76 ft
S = 0.03 ft/ft

Q = 7.1 * (1.49/0.024) * $(0.76)^{2/3}$ * $(0.03)^{1/2}$
Q = 63.7 cfs

A 3-ft CMP culvert at 3 percent slope can carry 63.7 cfs when flowing full, which is greater than the 10-year peak flow (24.1 cfs) and the 100-year peak flow at Point 4 (38.2 cfs). Therefore, the 3-ft culvert has adequate pipe capacity to convey the 10-year and 100-year peak flows at the entrance to the culvert.

The designer should evaluate the inlet losses for this culvert to estimate the headwater conditions and that the design depth (for 10-year flow) is at least 1 ft below the lowest roadway elevation at the culvert (Section 4.4.2 in Chapter 4).

Headwater depth can be evaluated using the nomographs found in the U.S. Department of Transportation Federal Highway Administration (FHWA) Hydraulic Design of Highway Culverts (see Figure D.2-3 in Appendix D.2 for a reproduction).

For this example problem, the roadway surface is located 5 ft above the culvert invert and the culvert is mitered to the roadway embankment slope. The relevant FHWA nomograph, for an inlet control CMP culvert, is reproduced in Attachment E.1-6.

To use the FHWA nomograph, project a straight line from the diameter of the culvert in inches, 36 inches, through the discharge for 10-year flow and 100-year flows, 24.1 cfs and 38.2 cfs, respectively. Where the projected line intersects the first Headwater Depth Diameters (HW/D) column, draw a horizontal line across all columns. The HW/D columns 1 through 3 reflect different culvert entrance types: column 1 reflects a culvert with a headwall; column 2 reflects a mitered culvert; and column 3 reflects a projecting culvert.

Headwater (HW) is measured from the invert of the culvert and is calculated from the resultant nomograph ratio (HW/D).

$$HW = D * \frac{HW}{D}$$

Where,

D is culvert diameter in ft;
HW/D is Headwater Depth in Diameters or nomograph ratio from the FHWA nomograph (Attachment E.1-6)

$$HW = D * \frac{HW}{D}$$

D = 3 ft
HW/D = 0.79 for the 10-year design flow

HW = 3 * 0.79
HW = 2.4 ft

The 10-year design flow is associated with a headwater depth of approximately 2.4 ft, measured from the invert of the culvert, which meets the required 1-ft clearance (Section 4.4.2).

Some roadway surfaces should not be overtopped during the 100-year event. Calculate the headwater depth associated with the 100-year flow.

$$HW = D * \frac{HW}{D}$$

D = 3 ft
HW/D = 1.1 for the 100-year flow

HW = 3 * 1.1
HW = 3.3 ft

The 100-year flow is associated with a headwater depth of approximately 3.3 ft, measured from the invert of the culvert, which does not overtop the roadway, located 5 ft above the culvert inlet.

Section 4.4.2, "Design Criteria for Culverts," requires culverts to have a minimum velocity of 2.5 ft/sec during the 2-year flow event. To verify the 2-year velocity, use the 2-year flow calculated in Attachment E.1-7 (16.0 cfs), which corresponds to a flow depth of approximately 12.5 inches.

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

n = 0.024
A = 2.2 ft^2
P_w = 3.8 ft
R_h = 0.58 ft
S = 0.03 ft/ft

V = (1.49/0.024) * (0.58)$^{2/3}$ * (0.03)$^{1/2}$

V = 7.5 ft/sec

The calculated velocity is greater than the required minimum velocity.

The figures and calculations included in a submittal to Sonoma Water for this example would include all of the information types listed in the Flood Management Design Review Plan Submittal Checklist (Appendix B.2), as indicated in parentheses below and shown in Appendix E.1:

Attachment E.1-1. Rock Creek Watershed Sub-Basins (Hydrology Map)
Attachment E.1-2. 10-Yr Peak Flow from Rock Creek Using Incremental Rational Method (Hydrology Calculations, partial)
Attachment E.1-3. 100-Yr Peak Flow from Rock Creek Using Incremental Rational Method (Hydrology Calculations, partial)
Attachment E.1-4. NOAA Atlas 14 map and rainfall data (Hydrologic Coefficients and Parameters, partial)
Attachment E.1-5. Rock Creek Culvert Input and Results Summary (Hydrologic Coefficients and Parameters, partial)
Attachment E.1-6. FHWA Hydraulic Design of Culverts Nomograph (Hydraulic Calculations, partial)
Attachment E.1-7. 2-Yr Peak Flow from Rock Creek Using Incremental Rational Method (Hydrology Calculations, partial)

A clear and complete submittal will expedite the review process.

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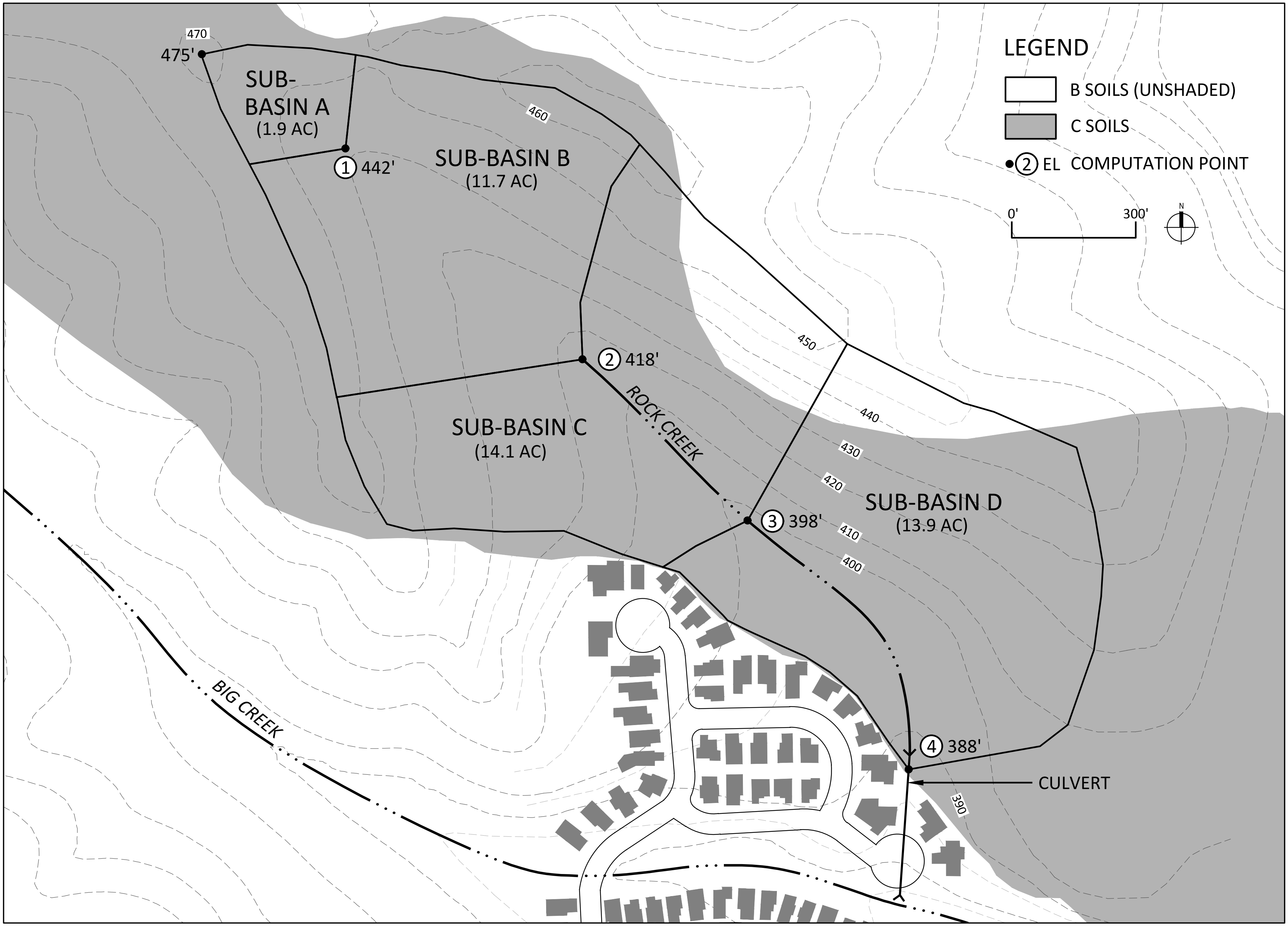
Attachment E.1

Attachment E.1-1
Rock Creek Watershed Basins



Attachment E.1-2
10-Yr Peak Flow From Rock Creek Using Incremental Rational Method

Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6	Col. 7	Col. 8	Col. 9	Col. 10	Col. 11	Col. 12	Col. 13	Col. 14	Col. 15	Col. 16	Col. 17
Sub-Basin	**Comp. Pt.**	**Area**	**ΣArea**	**Flow Path Length**	**US Elev.**	**DS Elev.**	**Slope**	**Runoff Type**	**Velocity**	**Travel Time**	**Time of Conc.**	**10-Yr Rainfall Intensity**	**C**	**CA**	**ΣCA**	**Peak Flow**
		(acres)	(acres)	(ft)	(ft)	(ft)	(ft/ft)		(ft/sec)	(min)	(min)	(in/hr)				(cfs)
A	1	1.9	1.9	420	475	442	0.0786	Overland Flow		15.0	15.0	1.92	0.43	0.8	0.8	1.6
B	2	11.7	13.6	770	442	418	0.0312	Shallow Conc. Flow	2.8	4.5	19.5	1.71	0.45	5.3	6.1	10.4
C	3	14.1	27.7	560	418	398	0.0357	Open Channel Flow	3.4	2.7	22.2	1.61	0.37	5.2	11.3	18.2
D	4	13.9	41.6	760	398	388	0.0132	Open Channel Flow	2.8	4.5	26.8	1.48	0.36	5.0	16.3	24.1

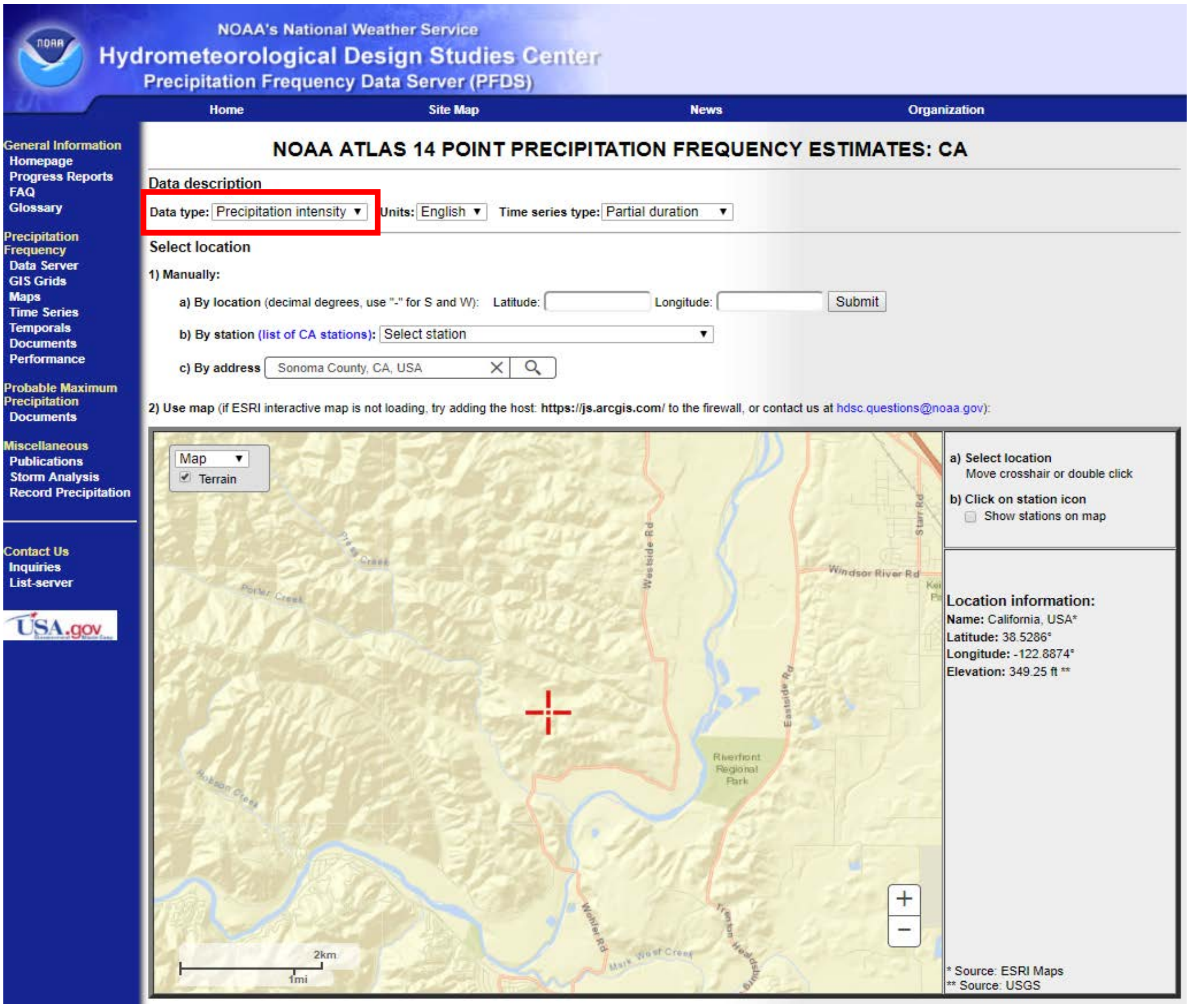
Col. 1 Steps 1-3. Sub-Basin defined from Att. E.1-1.
Col. 2 Steps 1-3. Computation Point defined from Att. E.1-1.
Col. 3 Steps 1-3. Area measured from Att. E.1-1.
Col. 4 Steps 1-3. Cumulative area measured from Att. E.1-1.
Col. 5 Steps 1-3. Flow path length measured from Att. E.1-1.
Col. 6 Steps 1-3. Upstream elevation measured from Att. E.1-1.
Col. 7 Steps 1-3. Downstream elevation measured from Att. E.1-1.
Col. 8 Step 3. Slope = (Col. 6 - Col. 7)/Col. 5
Col. 9 Steps 6 & 9. Runoff Type
Col. 10 Steps 6 & 9. Velocity for Sub-Basin B is calculated using Eqn. 3.6. Velocity for Sub-Basins C & D is calculated using Eqn. 3.8
Col. 11 Steps 6 & 9. Travel Time = Col. 5/(Col. 10 * 60)
Col. 12 Steps 4, 6 Time of Concentration = Col.11 + Col. 12 from upstream sub-basin
Col. 13 Steps 5, 6 & 9. Rainfall intensity determined from Col. 12 and Att. E.1-4
Col. 14 Steps 5, 7 & 9. Determined from Att. E.1-1 and Table C-1
Col. 15 Steps 5, 7 & 9. CA = Col. 14 * Col. 3
Col. 16 Steps 5, 7 & 9. Cumulative CA = Col. 15 + Col. 16 from upstream sub-basin
Col. 17 Steps 5, 8 & 9. Peak Flow = Col. 16 * Col. 13

Attachment E.1-3

100-Yr Peak Flow From Rock Creek Using Incremental Rational Method

Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6	Col. 7	Col. 8	Col. 9	Col. 10	Col. 11	Col. 12	Col. 13	Col. 14	Col. 15	Col. 16	Col. 17
Sub-Basin	**Comp. Pt.**	**Area**	**ΣArea**	**Flow Path Length**	**US Elev.**	**DS Elev.**	**Slope**	**Runoff Type**	**Velocity**	**Travel Time**	**Time of Conc.**	**100-Yr Rainfall Intensity**	**C**	**CA**	**ΣCA**	**Peak Flow**
		(acres)	(acres)	(ft)	(ft)	(ft)	(ft/ft)		(ft/sec)	(min)	(min)	(in/hr)				(cfs)
A	1	1.9	1.9	420	475	442	0.0786	Overland Flow		15.0	15.0	3.04	0.43	0.8	0.8	2.5
B	2	11.7	13.6	770	442	418	0.0312	Shallow Conc. Flow	2.8	4.5	19.5	2.68	0.45	5.3	6.1	16.3
C	3	14.1	27.7	560	418	398	0.0357	Open Channel Flow	3.9	2.4	21.9	2.54	0.37	5.2	11.3	28.7
D	4	13.9	41.6	760	398	388	0.0132	Open Channel Flow	3.3	3.9	25.8	2.34	0.36	5.0	16.3	38.2

ATTACHMENT E.1-4
NOAA Atlas 14



ATTACHMENT E.1-4 (continued)

NOAA Atlas 14 - Rainfall Intensity (inches/hour)

Data

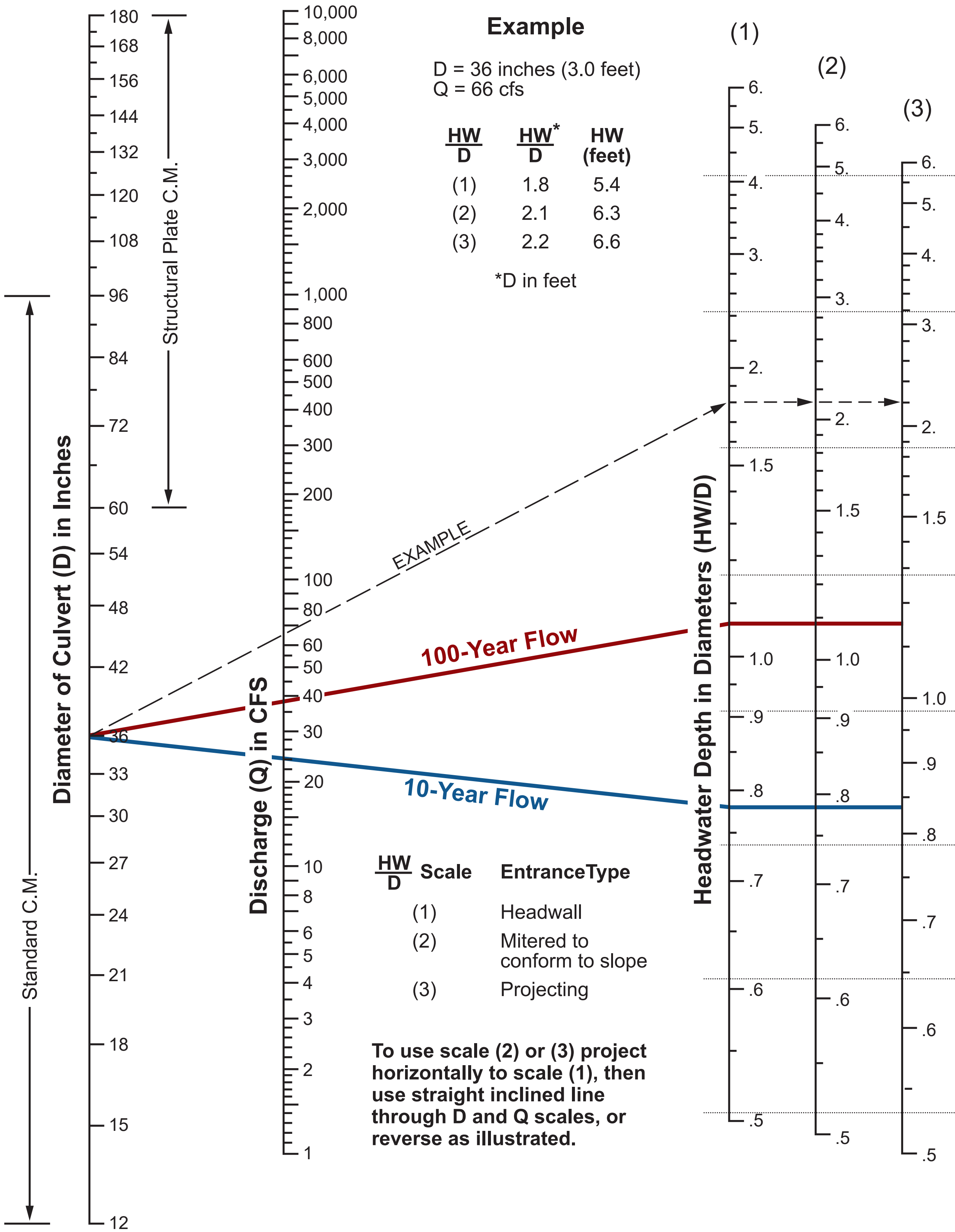
PDS-based precipitation frequency estimates with 90% confidence intervals (in inches/hour)							
Duration	Average recurrence interval (years)						
	1	2	5	10	25	50	100
5-min	**1.74** (1.55 1.98)	**2.20** (1.94 2.50)	**2.81** (2.48 3.20)	**3.32** (2.92 3.84)	**4.06** (3.42 4.87)	**4.64** (3.82 5.72)	**5.26** (4.20 6.66)
10-min	**1.25** (1.11 1.42)	**1.57** (1.40 1.79)	**2.02** (1.78 2.30)	**2.39** (2.09 2.75)	**2.91** (2.45 3.49)	**3.33** (2.74 4.10)	**3.77** (3.01 4.78)
15-min	**1.01** (0.896 1.14)	**1.27** (1.13 1.44)	**1.62** (1.44 1.86)	**1.92** (1.68 2.22)	**2.35** (1.98 2.82)	**2.68** (2.20 3.30)	**3.04** (2.42 3.85)
30-min	**0.71** (0.628 0.804)	**0.89** (0.790 1.01)	**1.14** (1.01 1.30)	**1.35** (1.18 1.56)	**1.65** (1.39 1.98)	**1.89** (1.55 2.32)	**2.13** (1.70 2.70)
60-min	**0.51** (0.448 0.574)	**0.64** (0.564 0.723)	**0.81** (0.719 0.928)	**0.96** (0.843 1.11)	**1.18** (0.989 1.41)	**1.34** (1.10 1.65)	**1.52** (1.21 1.93)
2-hr	**0.40** (0.356 0.454)	**0.50** (0.444 0.569)	**0.63** (0.558 0.720)	**0.74** (0.645 0.850)	**0.88** (0.742 1.06)	**0.99** (0.814 1.22)	**1.11** (0.881 1.40)
3-hr	**0.35** (0.312 0.399)	**0.44** (0.387 0.497)	**0.55** (0.484 0.624)	**0.64** (0.556 0.733)	**0.75** (0.634 0.905)	**0.84** (0.693 1.04)	**0.93** (0.745 1.18)
6-hr	**0.28** (0.245 0.314)	**0.34** (0.305 0.391)	**0.43** (0.379 0.489)	**0.50** (0.435 0.572)	**0.59** (0.493 0.703)	**0.65** (0.535 0.802)	**0.72** (0.572 0.909)
12-hr	**0.20** (0.179 0.229)	**0.26** (0.228 0.292)	**0.33** (0.290 0.374)	**0.38** (0.336 0.443)	**0.46** (0.386 0.550)	**0.51** (0.422 0.633)	**0.57** (0.454 0.722)
24-hr	**0.14** (0.129 0.163)	**0.19** (0.169 0.214)	**0.25** (0.220 0.280)	**0.29** (0.259 0.334)	**0.35** (0.303 0.416)	**0.40** (0.336 0.479)	**0.44** (0.367 0.546)
2-day	**0.10** (0.088 0.112)	**0.13** (0.115 0.146)	**0.17** (0.148 0.189)	**0.20** (0.174 0.224)	**0.23** (0.202 0.277)	**0.26** (0.222 0.317)	**0.29** (0.241 0.359)

ATTACHMENT E.1-5

Rock Creek Culvert Input and Results Summary

Total Basin Area (acres)	41.60				Reference:
Sub-Basin Area (acres)	Sub-Basin A	Sub-Basin B	Sub-Basin C	Sub-Basin D	
	1.90	11.70	14.10	13.90	Attachment E.1-1
Sub-Basin	Sub-Basin A	Sub-Basin B	Sub-Basin C	Sub-Basin D	
Predominant Soil Type	C	C	C	C	
Predominant Vegetation	Ag-Open Space	Ag-Open Space	Ag-Open Space	Ag-Open Space	
Composite C	0.43	0.45	0.37	0.36	Appendix C, Table C-1
Routing:	Sub-Basin C	Sub-Basin D			
Length (ft)	560	760			Attachment E.1-1
Slope (ft/ft)	0.0357	0.0132			Attachment E.1-1
Manning's n	0.05	0.05			Appendix D, Table D.2-1
Bottom width (ft)	5.0	5.0			Field Measurement
Side Slope xH:1V (ft)	2.0	2.0			Field Measurement
Hydraulic Radius 10-year	0.47	0.74			
Hydraulic Radius 100-year	0.58	0.93			
Rainfall:	Duration (min)	10-year Recurrence (inches/hr)	100-year Recurrence (inches/hr)		Attachment E.1-4
	5	3.32	5.26		
	10	2.39	3.77		
	15	1.92	3.04		
	30	1.35	2.13		
	60	0.96	1.52		
Results					
Q 25-year (cfs)	24.1				
Q 100-year (cfs)	38.2				

ATTACHMENT E.1-6
FHWA Hydraulic Design of Culverts Nomograph



Source: Hydraulic Design of Highway Culverts 3rd Ed. FHWA-HIF-12-026 HDS 5 April 2012 Chart 2B

Attachment E.1-7

2-Yr Peak Flow From Rock Creek Using Incremental Rational Method

Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6	Col. 7	Col. 8	Col. 9	Col. 10	Col. 11	Col. 12	Col. 13	Col. 14	Col. 15	Col. 16	Col. 17
Sub-Basin	**Comp. Pt.**	**Area**	**ΣArea**	**Flow Path Length**	**US Elev.**	**DS Elev.**	**Slope**	**Runoff Type**	**Velocity**	**Travel Time**	**Time of Conc.**	**10-Yr Rainfall Intensity**	**C**	**CA**	**ΣCA**	**Peak Flow**
		(acres)	(acres)	(ft)	(ft)	(ft)	(ft/ft)		(ft/sec)	(min)	(min)	(in/hr)				(cfs)
A	1	1.9	1.9	420	475	442	0.0786	Overland Flow		15.0	15.0	1.92	0.47	0.9	0.9	1.7
B	2	11.7	13.6	770	442	418	0.0312	Shallow Conc. Flow	2.8	4.5	19.5	1.14	0.45	5.3	6.2	7.0
C	3	14.1	27.7	560	418	398	0.0357	Open Channel Flow	3.1	3.0	22.5	1.07	0.37	5.2	11.4	12.1
D	4	13.9	41.6	760	398	388	0.0132	Open Channel Flow	2.5	5.0	27.5	0.97	0.36	5.0	16.4	16.0

Appendix E.2

Example Problem 2

Example Problem 2
Design a Culvert for Big Creek Using Synthetic Unit Hydrograph Method

For this example, the Synthetic Unit Hydrograph for Big Creek is calculated using the HEC-HMS computer model.

Step 1. Delineate the watershed and sub-basins.

Delineate the Big Creek watershed and water courses from topographic contours and divide the watershed into sub-basins, as shown in Attachment E.2-1, “Big Creek Sub-Basin Delineation.”

Set up the HEC-HMS model basin routing to reflect site hydrology as shown in Attachment E.2-2. HEC-HMS model input data for Sub-Basin 1A are shown below in Figure E.2-1, “HEC-HMS Sample Sub-Basin Input Screen.”

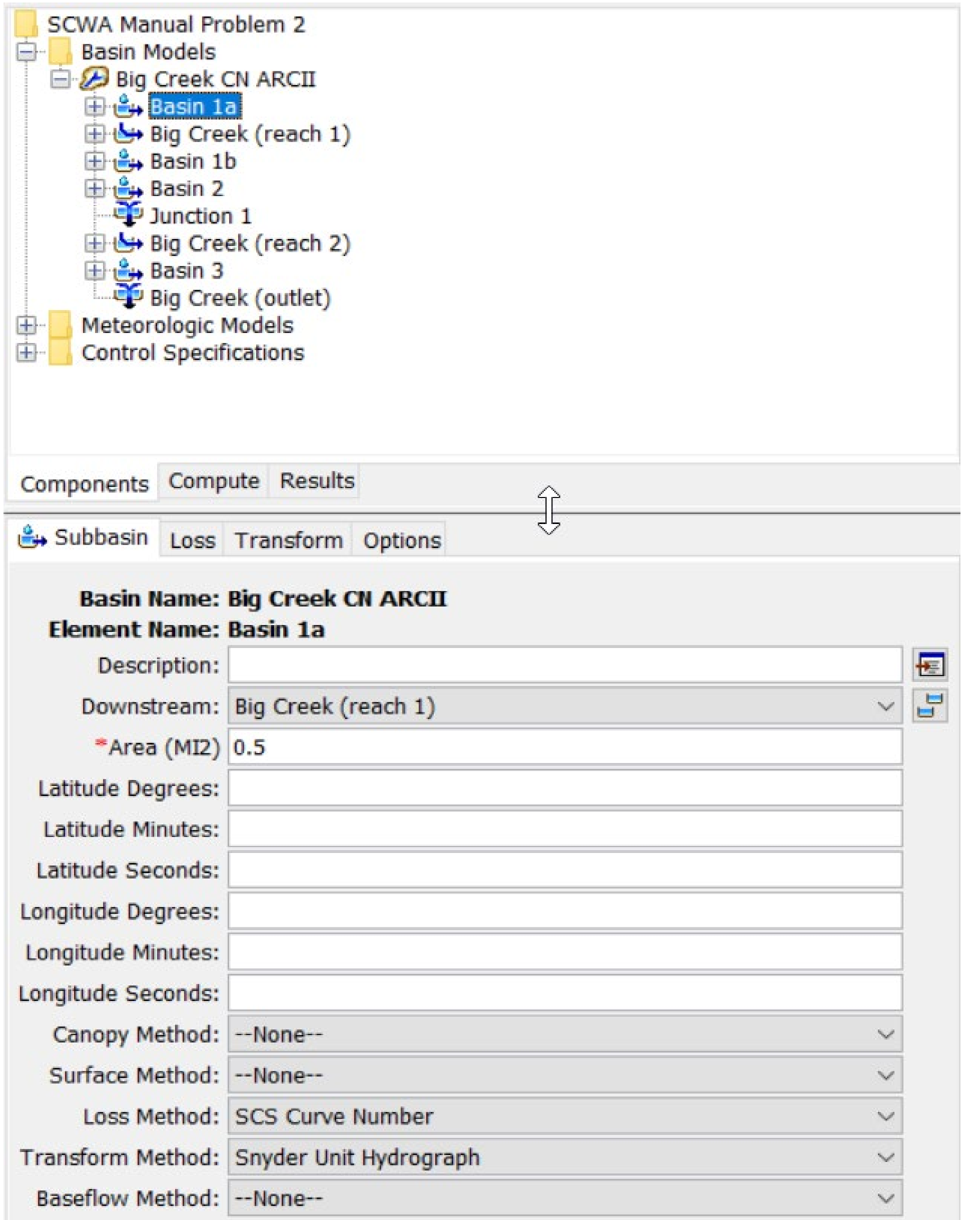


Figure E.2-1. HEC-HMS Sample Sub-Basin Input Screen

Step 2. Identify the design rain event.

The Big Creek watershed is 1.3 mi^2; therefore, the minimum design flow is the 25-year peak flow (Table 3-2 in Chapter 3).

Rainfall Intensity (inches) data are based on NOAA Atlas 14 precipitation data at the centroid for the watershed or the centroid of each sub-basin for larger watersheds. NOAA Atlas 14 precipitation data for the example watershed are found in Table E.2-1, "Summary of NOAA Atlas 14 Precipitation Frequency Estimates" and Attachment E.2-3.

Table E.2-1. Summary of NOAA Atlas 14 Precipitation Frequency Estimates

Duration	25-year Recurrence Interval (inches)	100-year Recurrence Interval (inches)
5 min	0.338	0.438
15 min	0.587	0.760
1 hr	1.18	1.52
2 hr	1.76	2.21
3 hr	2.26	2.81
6 hr	3.51	4.3
12 hr	5.52	6.87
24 hr	8.44	10.6

The Areal Reduction Factor (Figure 3-9 in Chapter 3) for the 1.3-mi^2 watershed is 100%. Therefore, the precipitation depths listed in Table E.2-1 can be input into HEC-HMS without modification (precipitation depths multiplied by 1).

Sonoma Water suggests using an intensity duration of 5 min and recommends using an intensity position of 50%. The storm duration will be 24 hours, as specified by Sonoma Water.

HEC-HMS model input for the design rainfall event are shown in Figure E.2-2, "HEC-HMS Frequency Storm for the 25-year Event," and Figure E.2-3, "HEC-HMS Frequency Storm Input Data for the 25-year Event," below.

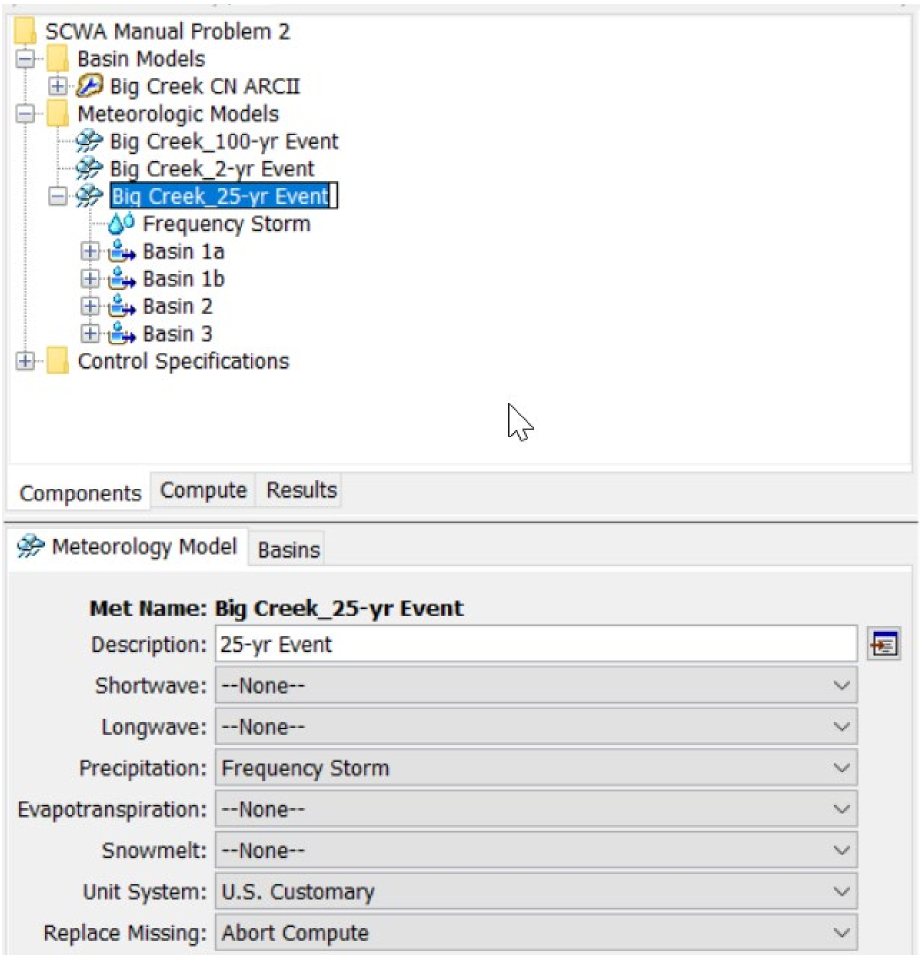


Figure E.2-2. HEC-HMS Frequency Storm for the 25-year Event

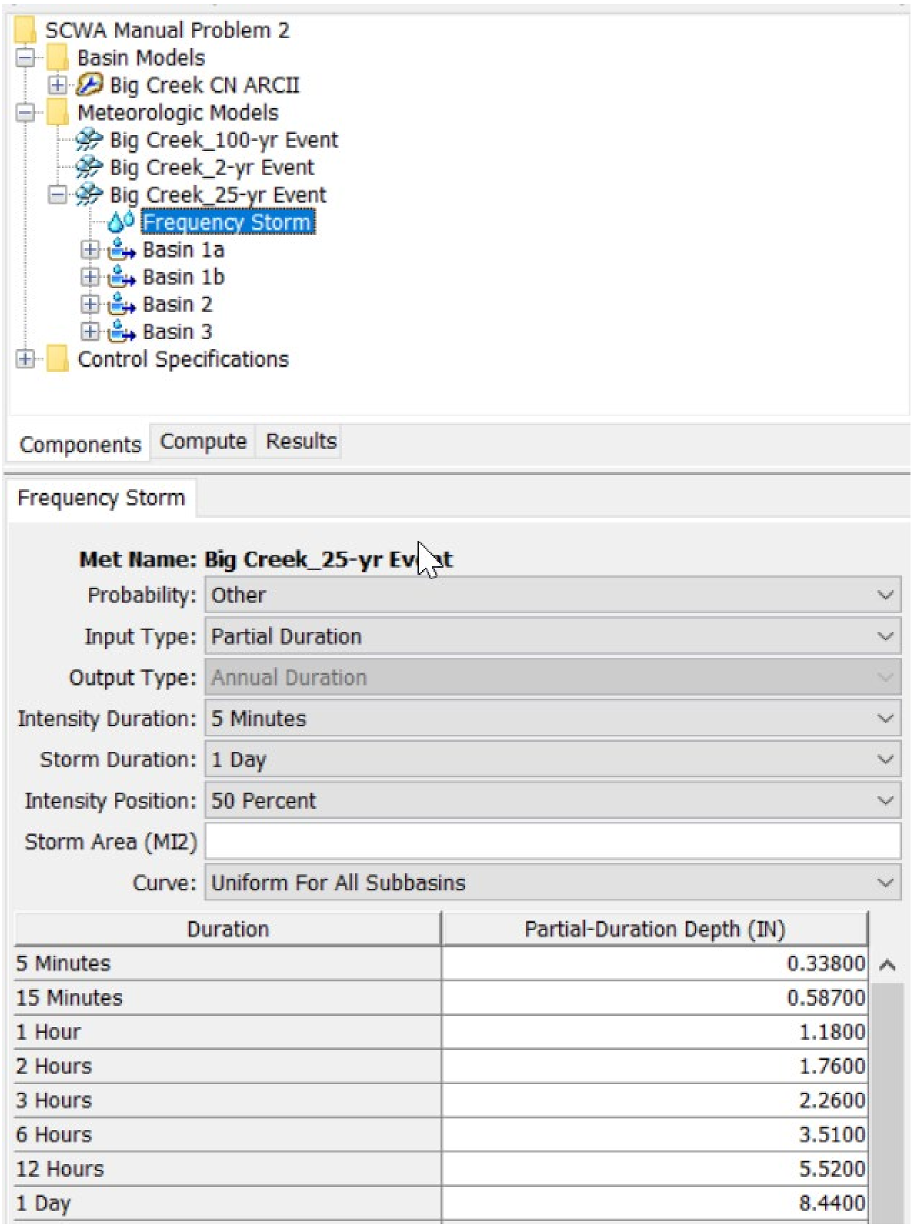


Figure E.2-3. HEC-HMS Frequency Storm Input Data for the 25-year Event

Step 3. Estimate the watershed losses.

Calculate the infiltration losses using the SCS Curve Number Loss Method. Calculate the Runoff Curve Number (CN) for each soil type and land cover combination using Runoff Curve Numbers for Synthetic Unit Hydrograph found in Tables C.2 through C.5, in Appendix C.

Soil type and land cover data are available from multiple sources, including those listed below:

- USDA Natural Resources Conservation Service. Web Soil Survey. Available at the following link: websoilsurvey.sc.egov.usda.gov.
- Sonoma County Vegetation Map. Available at: sonomavegmap.org.

The land cover and soil type or hypothetical Sub-Basins 1a, 2, and 3 are Oak Mountain Brush, and hydrologic soil group C in fair condition. Hypothetical Sub-Basin 1b has the same land cover, but is hydrologic soil type B in fair condition. Using Table C.5, “Runoff Curve Numbers for Arid and Semiarid Rangelands” in Appendix C, the category of Oak-aspen cover is identified and a curve number of 57 for Sub-Basins 1a, 2, and 3 and 48 for Sub-Basin 1b is selected.

Sample HEC-HMS model input data for the SCS Curve Number Loss Method are shown in Figure E.2-4, “HEC-HMS SCS Curve Number Loss Method Input,” below.

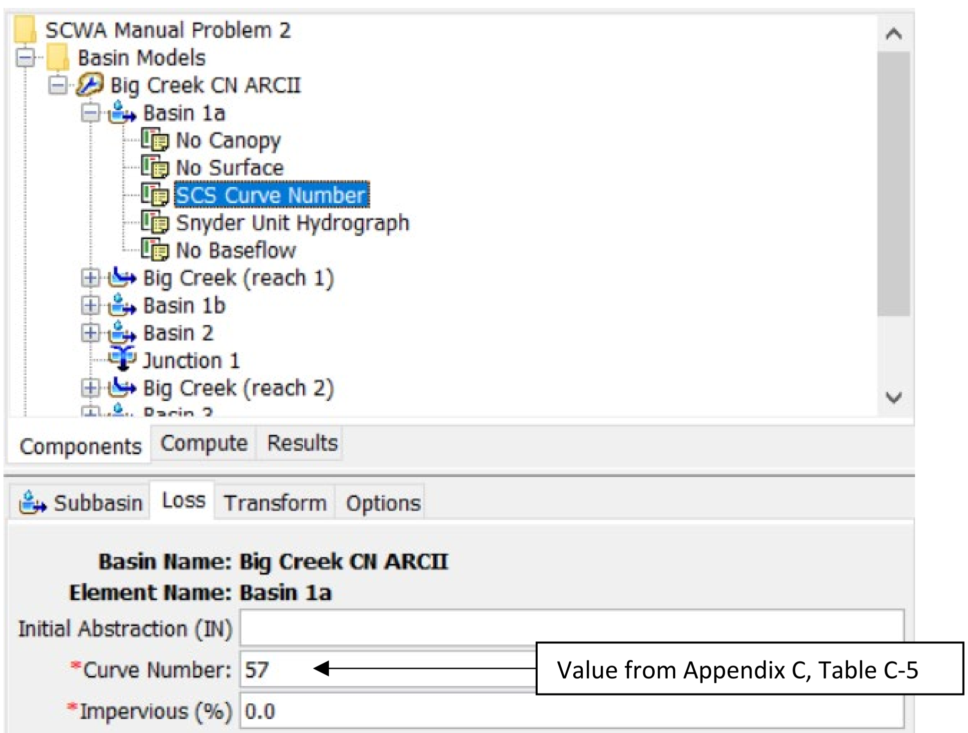


Figure E.2-4. HEC-HMS SCS Curve Number Loss Method Input

Step 4. Transform rainfall excess into direct runoff using the SUH method.

Surface runoff for each sub-basin is calculated by HEC-HMS using a Transform Method. Sonoma Water recommends the use of the Snyder Unit Hydrograph Transform Method, which is defined by a peaking coefficient and the standard lag or lag time.

The Peaking Coefficient describes that steepness of the hydrograph. Sonoma Water-required value is 0.75 (Chapter 3).

The standard lag, or lag time, is the difference between the timing of the hydrograph peak and the time of the rainfall centroid. The lag time is calculated from *Equation 3.13*, defined below.

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

Where,

t_L is the lag time (hr);
L is the length of the longest water course (miles [mi]);
L_C is the length of the longest water course measured from the watershed centroid to the watershed outlet (mi);
S_o is the average watershed slope (ft/mi).

Sub-basin watercourse lengths, watershed centroids, and the basis for the slope calculation are shown in Attachment E.2-1.

For Sub-Basin 1a, the uppermost watershed elevation along the watershed course is 1,730 ft, and the lowermost watershed elevation along the watershed course is 640 ft.

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

L = 0.91 mi
L_c = 0.38 mi
S_o = (1730 – 640)/0.70
S_o = 1,196 ft/mi

t_L = 1.56 $(0.91*0.38/1{,}196^{0.5})^{0.38}$
t_L = 0.27

Lag times for all Sub-Basins are tabulated in Table E.2-2, “Summary of Sub-Basin Lag Time Calculations,” below.

Table E.2-2. Summary of Sub-Basin Lag Time Calculations

	Sub-Basin 1a	**Sub-Basin 1b**	**Sub-Basin 2**	**Sub-Basin 3**
L (ft)	4,812	6,813	7,561	3,226
L_c (ft)	2,031	4,553	3,408	1,476
L (mi)	0.91	1.29	1.43	0.61
L_c (mi)	0.38	0.86	0.65	0.28
Uppermost Elevation (ft)	1,730	640	1,550	1,070
Lowermost Elevation (ft)	640	450	450	400
S_o (ft/mi)	1,196	147	768	1,097
t_L (hr)	0.27	0.63	0.43	0.21

HEC-HMS input data for the Snyder Unit Hydrograph Transform Method are shown in Figure E.2-5, “HEC-HMS Snyder Unit Hydrograph Transform Method Input,” below.

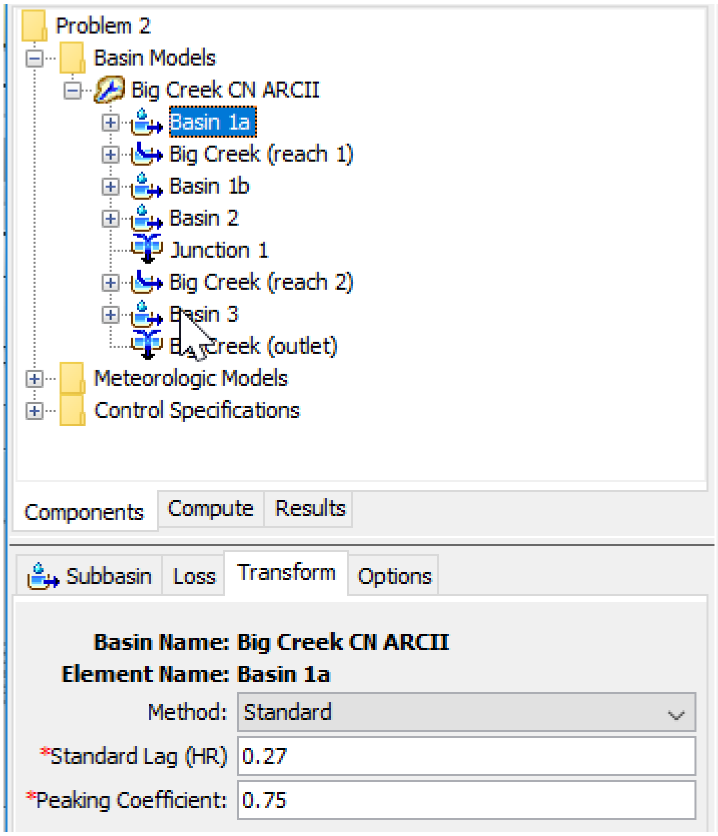


Figure E.2-5. HEC-HMS Snyder Unit Hydrograph Transform Method Input

Step 5. Route the direct runoff.

Sonoma Water recommends using the Muskingum-Cunge Routing Method to simulate flow in open channels. This method recalculates routing parameters based on flow depth at each calculation step and requires knowledge of channel geometry and Manning's number, based on a representative channel cross-section in each stream reach.

HEC-HMS input data for the Muskingum-Cunge Routing Method are shown in Figure E.2-6, "HEC-HMS Reach Input," and Figure E.2-7, "HEC-HMS Muskingum-Cunge Routing Method Input," below.

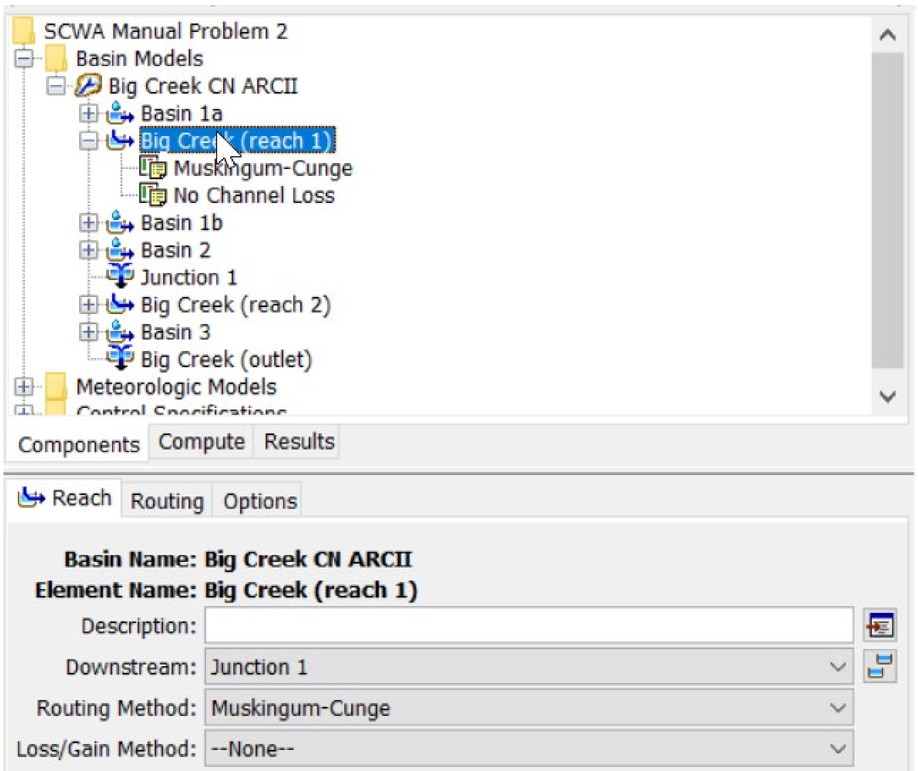


Figure E.2-6. HEC-HMS Reach Input

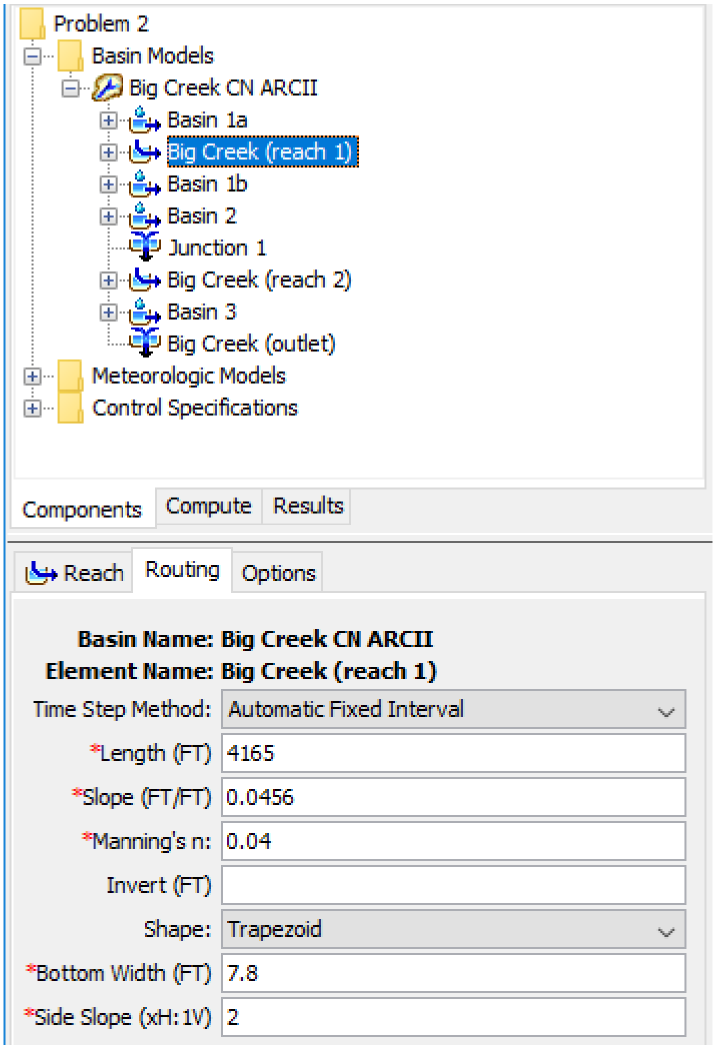


Figure E.2-7. HEC-HMS Muskingum-Cunge Routing Method Input

Step 6. Run the model.

To run the model, a control specifications file with the duration of the model simulation must be created. The model simulation duration should be longer than the storm duration.

Model simulation duration should be approximately twice as long as the storm duration. Therefore, the selected model simulation duration is 48 hr. The time interval selected for the computation timestep should be equal to the intensity duration selected and equal to the shortest duration of rainfall input in **Step 2**. In this example a time interval and intensity duration of 5 min was selected based on the shortest duration of rainfall data found in Table E.2-1. HEC-HMS model Control Specifications are shown in Figure E.2-8, “HEC-HMS Control Specifications Menu,” below.

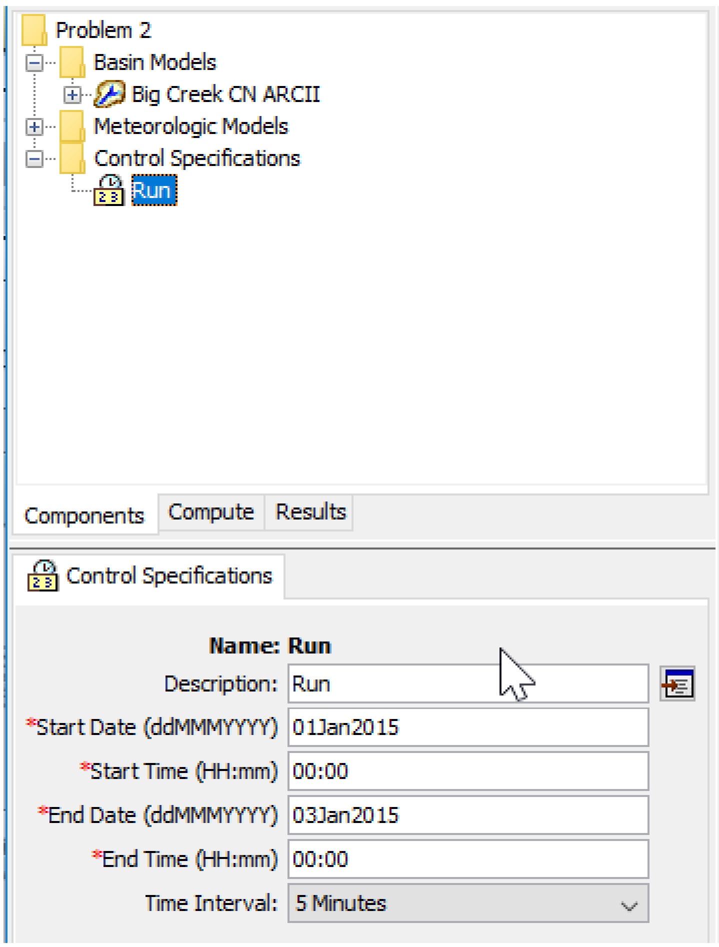


Figure E.2-8. HEC-HMS Control Specifications Menu

The model input parameters and results are summarized in Attachment E.2-4.

Step 7. Validation and calibration.

It is good engineering practice to validate the results created by any hydrologic model and it is required by Sonoma Water for watersheds greater than 1 sq mi in size. There are several approaches to validate the peak flows estimated by a hydraulic model, including:

- field indicators or historic accounts of high flow elevations,
- comparison with nearby gaged stream;
- calculation using regional regression equations, such as USGS Regional Regression equations found at https://streamstats.usgs.gov.

USGS Regional Regression equations can be utilized by navigating to the USGS StreamStats website, https://streamstats.usgs.gov. Search for the location of the study area or pan and zoom to locate the desired watershed outlet location. Select a point on the blue drainage network to delineate the watershed draining to that location (see Figure E.2-9, “USGS StreamStats Website Screenshot”). Once the watershed is delineated, the user may choose “Select Scenarios” and “Build A Report” to generate peak flow values for the watershed using regional regression equations, as shown in Table E.2-3, “USGS StreamStats Peak Flow Statistics for Big Creek.”

Figure E.2-9. USGS StreamStats Website Screenshot

Table E.2-3. USGS StreamStats Peak Flow Statistics for Big Creek
Adapted from Peak-Flow Statistics Flow Report [2012 5113 Region 1 North Coast]

Statistic	Value	Unit	PII	Plu	SEp
2 Year Peak Flood	103	ft^3/s	42	253	58.6
5 Year Peak Flood	202	ft^3/s	96.1	426	47.4
10 Year Peak Flood	275	ft^3/s	135	557	44.2
25 Year Peak Flood	370	ft^3/s	188	728	42.7
50 Year Peak Flood	444	ft^3/s	225	876	42.7
100 Year Peak Flood	522	ft^3/s	258	1050	44.3
200 Year Peak Flood	596	ft^3/s	294	1210	44.4
500 Year Peak Flood	694	ft^3/s	334	1440	46

Notes: ft^3/s = cubic feet per second; PII = Prediction Interval-Lower; Plu = Prediction Interval-Upper; Sep = Standard Error of Prediction; SE = Standard Error (other -- see report)

Sources: *Gotvald, A. J., Barth, N. A., Veilleux, A. G., and Parrett, Charles. 2012. Methods for determining magnitude and frequency of floods in California, based on data through water year 2006. U.S. Geological Survey Scientific Investigations Report 2012-5113, 38 p., 1 pl. (http://pubs.usgs.gov/sir/2012/5113/)*

Initial model results are compared with results of USGS Regional Regression equations in Table E.2-4, "HEC-HMS Model Results and Estimates from USGS Regional Regression Equations," below. The standard error (SE) percentages provided in Table E.2-3 above, or in Gotvald et al. (2012) as cited above, are used to define upper and lower limits of the approximately 68 percent confidence interval.

Table E.2-4. HEC-HMS Model Results and Estimates from USGS Regional Regression Equations

Flow	HEC-HMS (cfs)	USGS (cfs)	SE cfs (percent)	Lower Limit: USGS – SE (cfs)	Upper Limit: USGS + SE (cfs)
Q 2-year	95	103	60 (58.6)	43	163
Q 25-year	536	370	158 (42.7)	212	528
Q 100-year	859	522	231 (44.3)	291	753

The model outputs for both the Q25 and Q100 peak flows are outside of the 68 percent confidence interval based on the USGS regression equation results, as shown in Table E.2-4.[1] Therefore, the model results had to be reviewed with Sonoma Water to determine if adjustment of the hydrologic model is

[1] Note that the modeled 25- and 100-year peak flow rates do fall within the upper limit for peak flow (PIu in Table E.2-3 above), which represents the 90% confidence interval calculated by USGS StreamStats.

appropriate. The applicant's engineer consulted with Sonoma Water and it was determined that adjustment of the modeled flow rates was not appropriate. In most cases, results produced by a design storm event will be greater than analysis of historical gage data, such as were used to develop the regional regression relationships, would suggest. This difference is often the result of such factors as the extreme shape of a design hyetograph or the escape of, or constraint on, peak channel flows upstream of the gauged locations used to develop the regression equations.

Step 8. Culvert sizing.

For this example, assume the culvert that will be constructed is an 8-ft corrugated metal pipe (CMP) with a Manning's Roughness Coefficient of 0.024 (Appendix D.4), at a 3 percent slope with no tailwater.

Verify that an 8-ft culvert will be adequate to carry the design flow. Calculate the full capacity of the pipe, using a modification of Manning's equation (*Equation 3.8*) that incorporates *Equation 4.2*.

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

(*Equation 3.8*)

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

$n = 0.024$

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

(*Equation 3.9*)

$A = 50$ square ft

$P_w = 25$ ft

$R_h = 50/25$

$R_h = 2$

$S = 0.03$ ft/ft

$Q = 50 * (1.49/0.024) * (2)^{2/3} * (0.03)^{1/2}$

$Q = 854$ cfs

An 8-ft CMP culvert at 3 percent slope can carry 854 cfs when flowing full, which is greater than the design flow. Therefore, the 8-ft culvert has adequate pipe capacity.

The designer should evaluate the culvert for inlet control and estimate the headwater conditions. The headwater elevation associated with the design flow should be at least 1 ft below the roadway elevation according to Section 4.4.2.2, "Freeboard," in Chapter 4.

Headwater depth can be evaluated using the nomographs found in the U.S. Department of Transportation Federal Highway Administration (FHWA) Hydraulic Design of Highway Culverts (see Figure D.2-3 in Appendix D.2 for a reproduction).

For this example, the roadway surface is located 25 ft above the culvert inlet and the culvert is mitered to the roadway embankment slope. The relevant FHWA nomograph, for an inlet control CMP culvert, is reproduced in Attachment E.2-5.

To use the FHWA nomograph, project a straight line from the diameter of the culvert in inches, 96 inches, through the discharge for 25-year flow and 100-year flows. Where the projected line intersects the first Headwater Depth Diameters (HW/D) column, draw a horizontal line across all columns. The HW/D columns 1 through 3 reflect different culvert entrance types. The second HW/D column is representative of a mitered culvert.

Headwater (*HW*) is measured from the invert of the culvert and using the resultant nomograph ratio (*HW/D*).

$$HW = D * \frac{HW}{D}$$

Where,

D is culvert diameter in ft;
HW/D is Headwater Depth in Diameters from the FHWA nomograph.

$$HW = D * \frac{HW}{D}$$

D = 8 ft
HW/D = 1.31 for the 25-year design flow

HW = 8*1.31
HW = 10.5 ft

The 25-year design flow is associated with a headwater depth of 10.5 ft, measured from the invert of the culvert, which meets the required 1-ft clearance.

The potential for roadway overtopping in a 100-year event should be evaluated. Calculate the headwater depth associated with the 100-year flow.

$$HW = D * \frac{HW}{D}$$

D = 8 ft
HW/D = 2.38 for the 100-year flow

$HW = 8*2.38$
$HW = 19.0$ ft

The 100-year flow is associated with a headwater depth of approximately 19.0 ft, measured from the invert of the culvert, which does not overtop the roadway.

Chapter 4, Section 4.4.2, “Design Criteria for Culverts,” requires culverts to have a minimum velocity of 2.5 ft/sec during the 2-year flow event. The 2-year flow was calculated by HEC-HMS as 95 cfs, as shown in Table E.2-4.

To verify the adequacy of the 2-year velocity, use the Hydraulic Elements Chart for Circular Pipe (Attachment E.2-6).

Calculate proportional flow Q/Q_{full} for the 2-year design discharge.

$Q/Q_{full} = 95/854$
$Q/Q_{full} = 0.11$

Use the proportional discharge to find the proportional flow velocity V/V_{full} (shown on Attachment E.2-6, “Hydraulic Elements Chart for Circular Pipe,” in a red line).

$V/V_{full} = 0.67$

To find the flow velocity for the 2-year flow, multiply V/V_{full} by the full pipe velocity.

$$V = \frac{V}{V_{full}} * V_{full}$$

$$V = \frac{V}{V_{full}} * \frac{Q_{full}}{A_{full}}$$

$Q_{full} = 854$ cfs
$A_{full} = 50$ ft^2

$V = (0.67) * (854/50)$
$V = 11.44$ fps

The flow velocity in the culvert is greater than the minimum flow velocity.

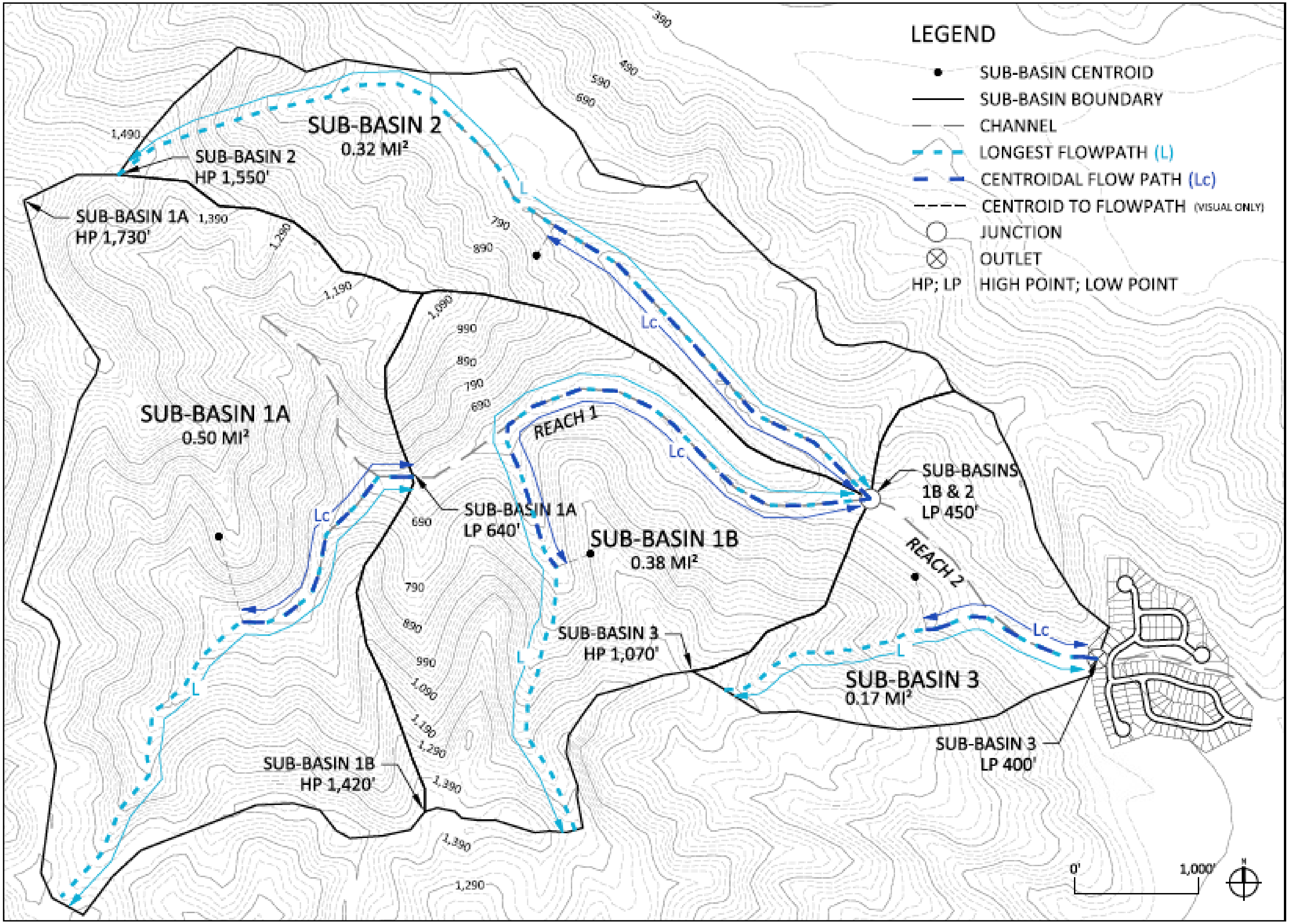
The figures and calculations included in a submittal to Sonoma Water for this example would include all of the information types listed in the Flood Management Design Review Plan Submittal Checklist (Appendix B.2), as indicated in parentheses below and shown in Appendix E.2:

Attachment E.2-1. Big Creek Sub-Basin Delineation (Hydrology Map)
Attachment E.2-2. HEC-HMS Routing Model Schematic (Hydrology Calculations, partial)
Attachment E.2-3. NOAA Atlas 14 map and rainfall data (Hydrology Coefficients and Parameters, partial)
Attachment E.2-4. Big Creek Culvert HEC-HMS Model Input and Results Summary (Hydrology Coefficients and Parameters, partial)
Attachment E.2-5. FHWA Hydraulic Design of Culverts Nomograph (Hydraulic Calculations, partial)
Attachment E.2-6. Hydraulic Elements Chart for Circular Pipe (Hydraulic Calculations, partial)

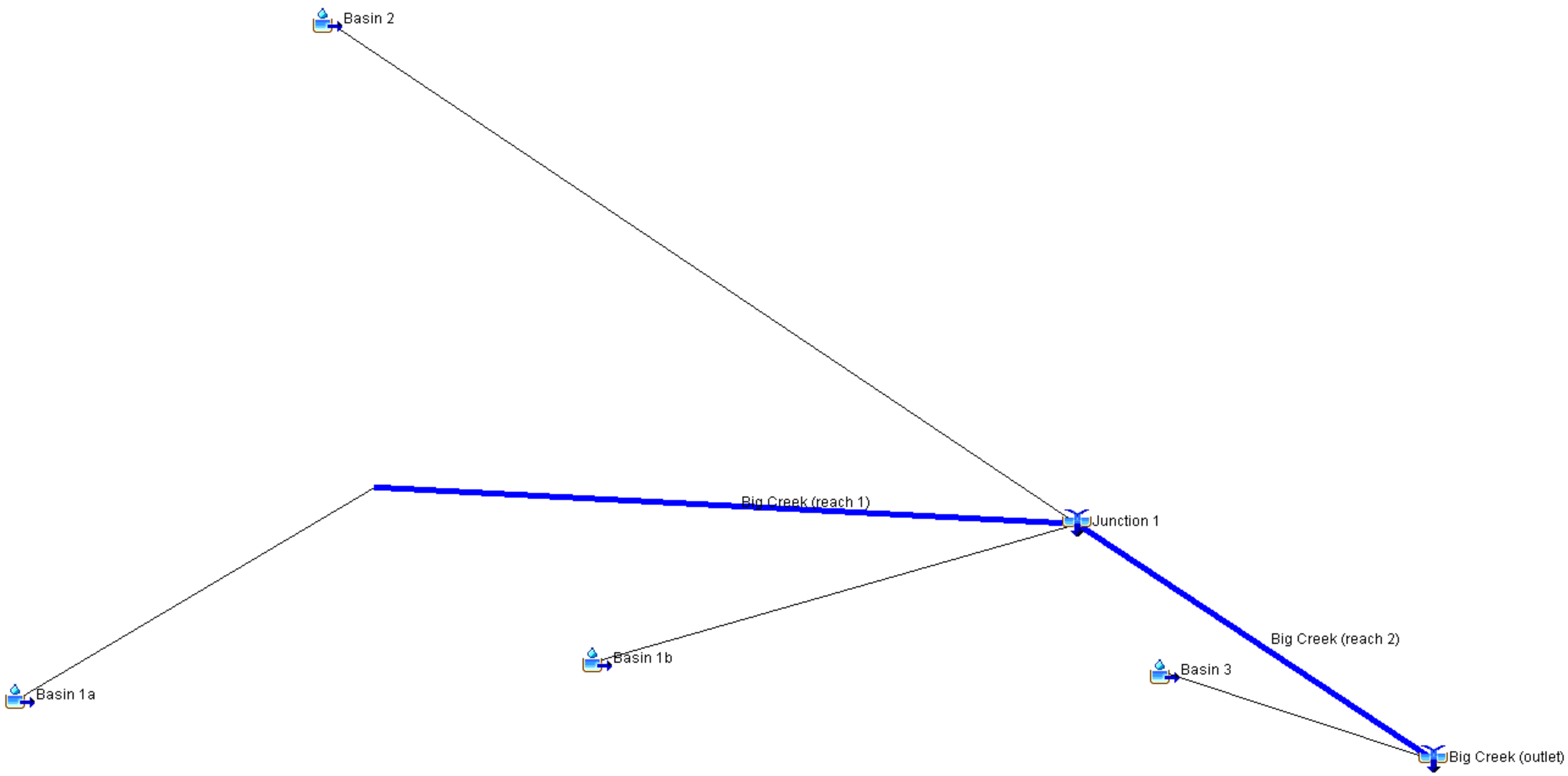
A clear and complete submittal will expedite the review process.

Attachment E.2

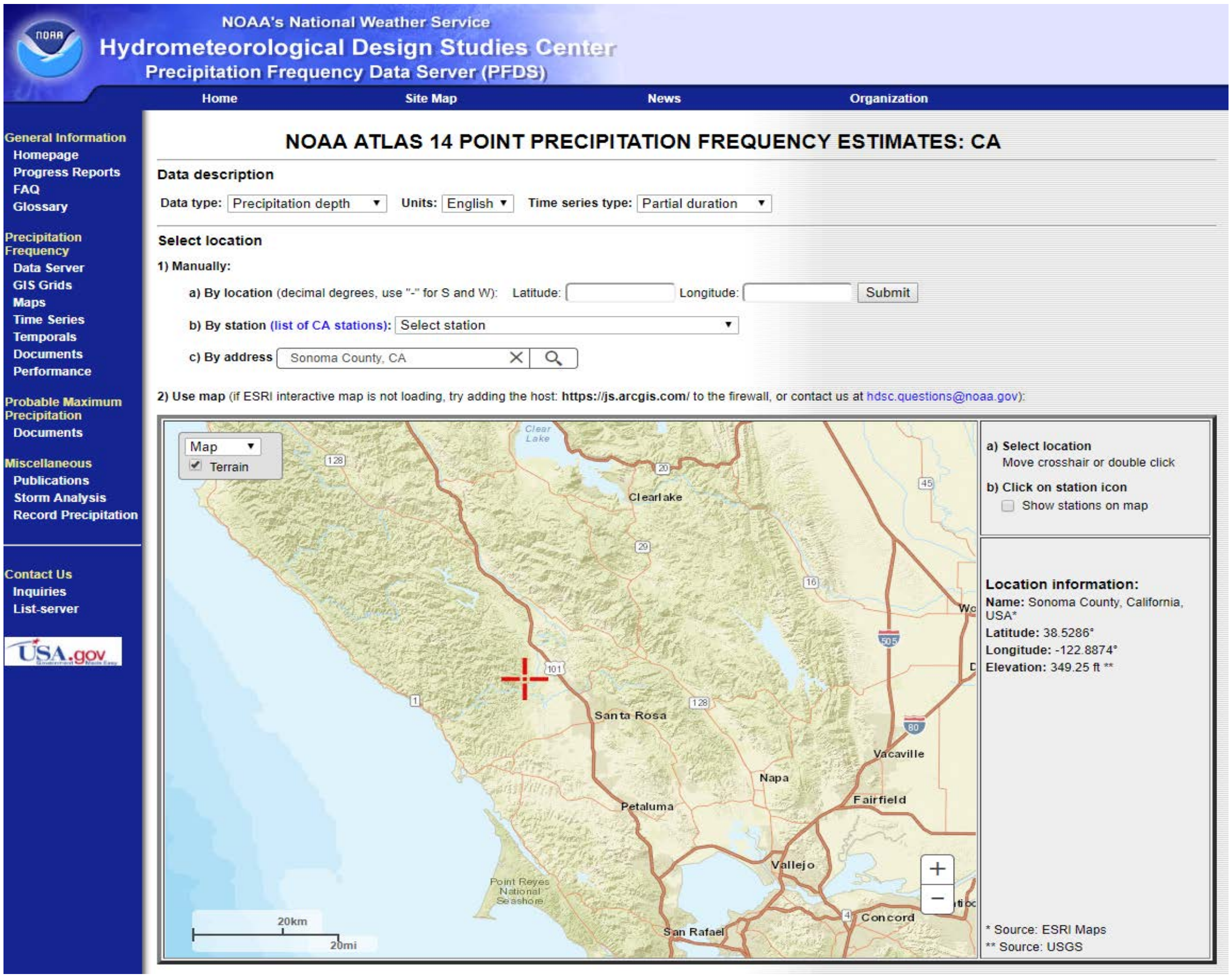
ATTACHMENT E.2-1.
Big Creek Watershed Basins



ATTACHMENT E.2-2.
HEC-HMS Routing Model Schematic



ATTACHMENT E.2-3.
NOAA Atlas 14



NOAA Atlas 14 Rainfall Depth (inches) Data

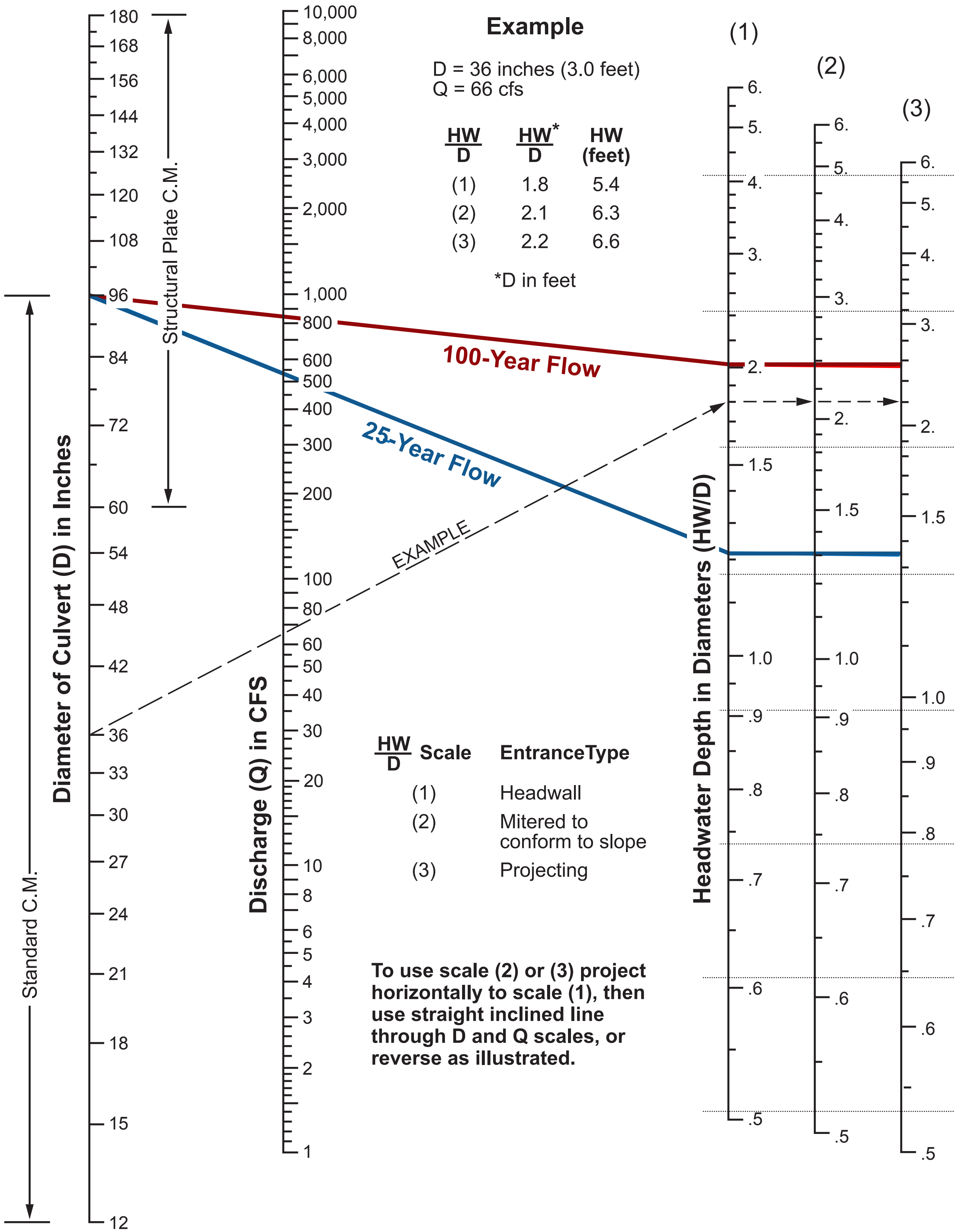
PDS-based precipitation frequency estimates with 90% confidence intervals (in inches)[1]

Duration	Average recurrence interval (years)						
	1	2	5	10	25	50	100
5-min	**0.145** (0.129 0.165)	**0.183** (0.162 0.208)	**0.234** (0.207 0.267)	**0.277** (0.243 0.320)	**0.338** (0.285 0.406)	**0.387** (0.318 0.477)	**0.438** (0.350 0.555)
10-min	**0.208** (0.185 0.237)	**0.262** (0.233 0.299)	**0.336** (0.297 0.383)	**0.398** (0.348 0.459)	**0.485** (0.408 0.582)	**0.555** (0.456 0.683)	**0.629** (0.501 0.796)
15-min	**0.252** (0.224 0.286)	**0.317** (0.282 0.361)	**0.406** (0.359 0.464)	**0.481** (0.421 0.555)	**0.587** (0.494 0.704)	**0.671** (0.551 0.826)	**0.760** (0.606 0.963)
30-min	**0.354** (0.314 0.402)	**0.446** (0.395 0.507)	**0.570** (0.504 0.651)	**0.675** (0.591 0.779)	**0.824** (0.693 0.989)	**0.943** (0.774 1.16)	**1.07** (0.851 1.35)
60-min	**0.505** (0.448 0.574)	**0.635** (0.564 0.723)	**0.813** (0.719 0.928)	**0.963** (0.843 1.11)	**1.18** (0.989 1.41)	**1.34** (1.10 1.65)	**1.52** (1.21 1.93)
2-hr	**0.800** (0.711 0.909)	**0.999** (0.887 1.14)	**1.26** (1.12 1.44)	**1.47** (1.29 1.70)	**1.76** (1.48 2.12)	**1.98** (1.63 2.44)	**2.21** (1.76 2.80)
3-hr	**1.05** (0.936 1.20)	**1.31** (1.16 1.49)	**1.64** (1.45 1.87)	**1.91** (1.67 2.20)	**2.26** (1.91 2.72)	**2.53** (2.08 3.12)	**2.81** (2.24 3.55)
6-hr	**1.65** (1.47 1.88)	**2.06** (1.82 2.34)	**2.57** (2.27 2.93)	**2.97** (2.60 3.43)	**3.51** (2.95 4.21)	**3.90** (3.20 4.80)	**4.30** (3.43 5.44)
12-hr	**2.43** (2.16 2.76)	**3.10** (2.75 3.52)	**3.95** (3.49 4.51)	**4.63** (4.05 5.33)	**5.52** (4.65 6.63)	**6.20** (5.09 7.62)	**6.87** (5.47 8.70)
24-hr	**3.45** (3.10 3.92)	**4.52** (4.06 5.14)	**5.89** (5.28 6.72)	**6.99** (6.21 8.02)	**8.44** (7.28 9.99)	**9.54** (8.07 11.5)	**10.6** (8.80 13.1)
2-day	**4.73** (4.25 5.37)	**6.16** (5.53 7.00)	**7.96** (7.13 9.08)	**9.38** (8.34 10.8)	**11.2** (9.69 13.3)	**12.6** (10.7 15.2)	**14.0** (11.6 17.2)

ATTACHMENT E.2-4.
Big Creek Culvert HEC-HMS Model Input and Results Summary

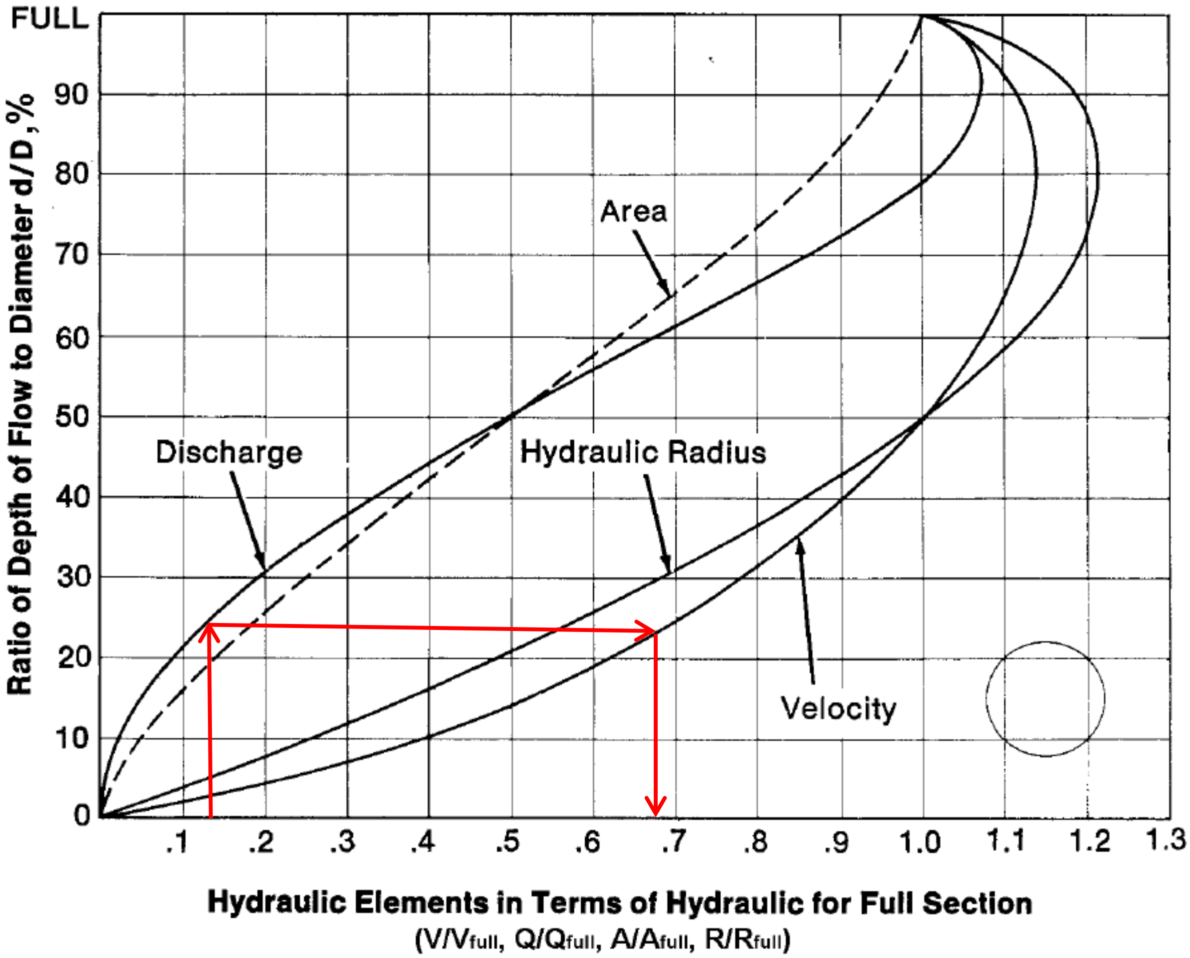
Total Basin Area (mi^2)	1.37				Reference:
Sub-Basin	Sub-Basin 1a	Sub-Basin 1b	Sub-Basin 2	Sub-Basin 3	
Area (mi^2)	0.50	0.38	0.32	0.17	Figure E.2-1
Loss (SCS Curve Number):	Sub-Basin 1a	Sub-Basin 1b	Sub-Basin 2	Sub-Basin 3	
Predominant Soil Type	C	B	C	C	Figure E.2-1
Predominant Vegetation	Brush	Brush	Brush	Brush	Figure E.2-1
Composite CN ARCII	57	48	57	57	Appendix C, Table C-5
Transform (Snyder Unit Hydrograph):	Sub-Basin 1a	Sub-Basin 1b	Sub-Basin 2	Sub-Basin 3	
t_L, Standard Lag (hours)	0.27	0.63	0.43	0.21	Eqn. 3.13
Peaking Coefficient	0.75	0.75	0.75	0.75	
Routing (Muskingum-Cunge):			Reach 1	Reach 2	
Length (ft)			4165	2250	Figure E.2-1
Slope (ft/ft)			0.0456	0.0222	Figure E.2-1
Manning's n			0.04	0.04	Appendix D.2, Table D.2-4
Bottom width (ft)			7.8	10.0	Field Measurements
Side Slope xH:1V (ft)			2.0	2.0	Field Measurements
Rainfall Depth-Duration-Frequency:	Duration	2-year Recurrence Depth (inches)	25-year Recurrence Depth (inches)	100-year Recurrence Depth (inches)	Attachment E.2-3
	5 min	0.183	0.338	0.438	
	15 min	0.317	0.587	0.760	
	1 hr	0.635	1.18	1.52	
	2 hrs	0.999	1.76	2.21	
	3 hrs	1.31	2.26	2.81	
	6 hrs	2.06	3.51	4.30	
	12 hrs	3.10	5.52	6.87	
	24 hrs	4.52	8.44	10.60	
Areal Reduction Factor	100%				Figure 3-9
Precipitation (Frequency Storm):					
Intensity duration (min)	5				
Storm duration (hr)	24				
Model Run duration (hours)	48				
Results					
Q 2-year (cfs)	95				
Q 25-year (cfs)	536				
Q 100-year (cfs)	859				

ATTACHMENT E.2-5.
FHWA Hydraulic Design of Culverts Nomograph



Source: Hydraulic Design of Highway Culverts 3rd Ed. FHWA-HIF-12-026 HDS 5 April 2012 Chart 2B

ATTACHMENT E.2-6.
Hydraulic Elements Chart for Circular Pipe



Source: Introduction to Highway Hydraulics FHWA-NHI-08-090, June 2008, Figure 7.1.

Usage: Calculate Q_2/Q_{full}, project line vertically from x axis to Discharge curve, project horizontally from Discharge curve to Velocity curve, project vertically down to determine ratio of V/V_{full}.

Appendix E.3

Example Problem 3

Example Problem 3
Design Stormwater Conveyance Pipe Network for Blue Heron South – Full-Flowing Pipe

The full-flowing pipe methodology described in Example Problem 3 is typical for designs that have flat pipe slopes and/or surcharged conditions at the outlet. If a conveyance network is anticipated to contain both partially full and full-flowing pipes, it is recommended to follow the design approach for full-flowing pipes.

Step 1. Layout conveyance pipe network and delineate drainage catchments.

The storm drainage system for Blue Heron South, a medium to high density residential subdivision is shown in Attachment E.3-1. The drainage system is composed of eight drainage areas that drain to catch basins.

Step 2. Evaluate pipe conveyance design and downstream conditions.

For this example, the layout of the site is generally flat (roadway and utility grades less than two percent). The drainage pipe network will drain to an existing 18-inch outfall that is submerged below the water surface elevation of Big Creek. The Hydraulic Grade Line (HGL) and Energy Grade Line (EGL) at the outfall is known to be 365.50 ft (previously modeled water surface elevation).

Step 3. Calculate runoff from each drainage catchment.

Use a computation form to calculate the hydrology of the system using the Incremental Rational Method as outlined in Example Problem 1. This form will facilitate calculation of the time of concentration and flow rate for each pipe segment in the system. The computation form used to analyze the drainage system hydrology is included as Attachment E.3-2. The development of the entries in this form for several sample pipe segments is described below.

Rainfall intensity (inches/hr) data are based on NOAA Atlas 14 data for the 10-year storm event, which were previously described in Example Problem 1, Table E.1-2. The trendline equation describing the relationship between time of concentration and rainfall intensity for Example Problem 1 presented in Figure E.1-1 has been reproduced below.

$$I = 6.887 * t_c^{-0.468}$$

Where,

I is rainfall intensity (inches/hr);
t_c is time of concentration (min).

The initial time of concentration for flow entering each catch basin and the pipe segments connecting catch basins to manhole is estimated to be 10 min (from Table 3-3).

t_c = 10.00 min

The rainfall intensity associated with a time of concentration of 10 min for the watershed is 2.39 inches/hr (from Table E.1-2).

I = 2.39 inches/hr

Find the Runoff Coefficient (C) for the subdivision using Table C-1 in Appendix C and the information listed below.

Soil: Type B (from Attachment E.1-1)
Land Use: Residential Medium-Density, average lot size 1/8 acre (from Attachment E.3-1)
Average Slope: 0-2% (from Attachment E.3-1)

Given the above:

Runoff Coefficient = 0.71 (from Table C-1 in Appendix C)

Rainfall in Sub-Basins 4A and 4B collect in catch basins CB-4A and CB-4B and ultimately flow into Pipe 4. Flow into each catch basin is calculated using the Incremental Rational Method, *Equation 3.1*.

$$Q = C * I * A \qquad \text{(Equation 3.1)}$$

$$Q_{Runoff} = C * I * A$$

C = 0.71 (Table C-1)
I_{4A} = 2.39 inches/hr (Table E.1-2)
I_{4B} = 2.39 inches/hr (Table E.1-2)
A_{4A} = 1.20 acres
A_{4B} = 1.20 acres

$Q_{CB\text{-}4A}$ = 0.71*2.39*1.20
$Q_{CB\text{-}4A}$ = 2.04 cfs

$Q_{CB\text{-}4B}$ = 0.71*2.39*1.20
$Q_{CB\text{-}4B}$ = 2.04 cfs

To calculate travel time and time of concentration in each pipe segment, estimate velocities in each pipe using the runoff calculated above.

$$V = \frac{Q_{Runoff}}{A} \qquad \textit{(Equation 4.2)}$$

Where,

Q_{Runoff} is the discharge (cfs)

A is the pipe cross-sectional area (ft^2)

Catch basin CB-4A is connected to manhole MH-4 with 15 linear feet of 12-inch reinforced concrete pipe (RCP) with a Manning's value of 0.014 (Appendix D.2, Table D.2-4) at a slope of 1.5%.

$$V = \frac{Q_{Runoff}}{A}$$

$Q_{Pipe\text{-}41}$ = 2.04 cfs (Attachment E.3-2)

$$A = \frac{\pi * D^2}{4}$$

D = 1.00 ft (Attachment E.3-2)

$A = (\pi * 1.00^2)/4$

$A = 0.79\ ft^2$

$V = (2.04/0.79)$

$V = 2.59$ ft/sec

Calculate the travel time in Pipe 41 using *Equation 3.10*.

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

$L_{Pipe\text{-}41}$ = 15.00 ft (Attachment E.3-2)

$V_{Pipe\text{-}41}$ = 2.59 fps

$t_{p\ Pipe\text{-}41} = 15 / (2.59 * 60)$

$t_{p\ Pipe\text{-}41} = 0.10$ min

Calculate the time of concentration in Pipe 4 by adding the time of concentration from the upstream contributing drainage area to the travel time in the upstream pipe, Pipe 41.

$t_{c\ Pipe\text{-}4} = 10.00 + 0.10$

$t_{c\ Pipe\text{-}4} = 10.10$ min

Use time of concentration to calculate rainfall intensity (using the trendline equation from Figure E.1-1).

$I_{Pipe\text{-}4} = 6.887*(10.10)^{-0.468}$

$I_{Pipe\text{-}4} = 2.33$ inches/hr

Calculate the flows in Pipe 4, Catch basin 3A, Catch basin 3B, and Pipe 3 using *Equation 3.1*.

$$Q_{Runoff} = C * I * A$$

$C = 0.71$ (from Table C-1 in Appendix C)

$$A_{Pipe\text{-}4} = A_{4A} + A_{4B}$$

$A_{Pipe\text{-}4} = 1.20 + 1.20 = 2.40$ acres (Attachment E.3-2)

$I_{Pipe\text{-}4} = 2.33$ inches/hr (Attachment E.3-2)

$Q_{Pipe\text{-}4} = 0.71*2.40*2.33 = 3.98$ cfs

$A_{CB3\text{-}A} = 1.06$ acres (Attachment E.3-2)

$I_{CB3\text{-}A} = 2.39$ inches/hr (Attachment E.3-2)

$Q_{CB3\text{-}A} = 0.71*1.06*2.39 = 1.80$ cfs

$A_{CB3\text{-}B} = 1.06$ acres (Attachment E.3-2)

$I_{CB3\text{-}B} = 2.39$ inches/hr (Attachment E.3-2)

$Q_{CB3\text{-}B} = 0.71*1.06*2.39 = 1.80$ cfs

$$t_{c\,Pipe\text{-}3} = t_{c\,Pipe\text{-}4} + t_{p\,Pipe\text{-}4}$$

$t_{c\ Pipe\text{-}4}$ = 10.10 min

$$t_{p\,Pipe\text{-}4} = \frac{L_{Pipe\text{-}4}}{V_{p\,Pipe\text{-}4} * 60}$$

$L_{Pipe\text{-}4}$ = 210.00 ft (Attachment E.3-2, Column 11)

$$V_{Pipe\text{-}4} = \frac{Q_{Pipe\text{-}4}}{A_{Pipe\text{-}4}}$$

$Q_{Pipe\text{-}4}$ = 3.98 cfs (Attachment E.3-2, Column 14)

$$A_{Pipe\text{-}4} = \frac{\pi * D^2}{4}$$

D = 1.50 ft (Attachment E.3-2)
$A = (\pi * 1.50^2)/4$
A = 1.77 ft^2

$V_{Pipe\text{-}4}$ = (3.98/1.77)
$V_{Pipe\text{-}4}$ = 2.25 ft/sec (Attachment E.3-2, Column 13)

$t_{pPipe\text{-}4}$ = 210.00/(60*2.25) = 1.56 min (Attachment E.3-2, Column 3)

$t_{c\ Pipe\text{-}3}$ = 10.10 + 1.56 = 11.65 min (Attachment E.3-2, Column 4)

$I_{Pipe\text{-}3} = 6.887*(11.65)^{-0.468}$
$I_{Pipe\text{-}3}$ = 2.18 inches/hr

$A_{Pipe\text{-}3} = A_{Pipe\text{-}4} + A_{3A} + A_{3B}$
$A_{Pipe\text{-}3}$ = 2.40 + 1.06 + 1.06
$A_{Pipe\text{-}3}$ = 4.52 acres (from Attachment E.3-2, Column 2)

$$Q_{Pipe\text{-}3} = C * I_{Pipe\text{-}3} * A_{Pipe\text{-}3}$$

$Q_{Pipe\text{-}3}$ = 0.71*2.18*4.52 = 7.00 cfs

If there is more than one upstream segment, add the travel time to the time of concentration of the upstream pipe segments and use the upstream flow path which has the highest cumulative value of contributing drainage area multiplied by the runoff coefficient (Attachment E.3-2, Column 8). Keep in mind when calculating time of concentration that the upstream flow path along the main storm line should have priority over a lateral storm line.

For example, Pipe 3 and Pipe 5 connect at MH-2.

$\Sigma CA_{\ Pipe\text{-}3} = 3.21$

or

$\Sigma CA_{\ Pipe\text{-}5} = 1.63$

Since $\Sigma CA_{\ Pipe\text{-}3} > \Sigma CA_{\ Pipe\text{-}5}$, the flow path upstream of MH-2 in the direction of Pipe 3 is used to calculate the time of concentration for Pipe 2.

$$t_{cPipe\text{-}2} = t_{cPipe\text{-}3} + t_{pPipe\text{-}3}$$

$t_{c\ Pipe\text{-}3}$ = 11.65 min (from Attachment E.3-2, Column 4)

$$t_{pPipe\text{-}3} = \frac{L_{Pipe\text{-}3}}{V_{Pipe\text{-}3} * 60}$$

$L_{\ Pipe\text{-}3}$ = 55.00 ft (from Attachment E.3-2, Column 11)

$$V_{Pipe\text{-}3} = \frac{Q_{Pipe\text{-}3}}{A_{Pipe\text{-}3}}$$

$Q_{\ Pipe\text{-}3}$ = 7.00 cfs (from Attachment E.3-2, Column 14)

$$A_{Pipe\text{-}3} = \frac{\pi * D^2}{4}$$

D = 1.50 ft (Attachment E.3-2, Column 9)
$A = (\pi*1.50^2)/4$
A = 1.77 ft^2

$V_{\ Pipe\text{-}3}$ = (7.00/1.77)
$V_{\ Pipe\text{-}3}$ = 3.96 ft/sec (Attachment E.3-2, Column 13)

$t_{p\ Pipe\text{-}3}$ = 55.00/(60*3.96) = 0.23 min (Attachment E.3-2, Column 3)

$t_{c\ Pipe\text{-}2}$ = 11.65 + 0.23 = 11.88 min (Attachment E.3-2, Column 4)

Continue to calculate flow in each pipe segment utilizing the following steps:

1. Calculate velocity using $Q_{\ Runoff}$ from the upstream pipe segment or catch basin.
2. Utilize velocity to calculate travel time (*Equation 3.10*)

3. Calculate time of concentration using the travel time and the greatest upstream $\Sigma C*A$ value.
4. Utilize time of concentration to find rainfall intensity.
5. Solve for runoff using area, runoff coefficient, and rainfall intensity (*Equation 3.1*)

Using the Incremental Rational Method to determine peak flows does not take into account the size of the pipe conveying the flow. Check that each pipe has the capacity to convey the calculated stormwater runoff using *Equations 3.8, 4.2, and 4.3* (as shown in Example Problem 1): $Q_{Full} > Q_{Runoff}$

The calculated full pipe capacity, Q_{Full}, is used only as a preliminary check when designing a storm drainage network as Manning's equation (*Equation 3.8*) assumes the pipe is free flowing and not surcharging. Whereas, in this example problem the system is surcharging under pressure conditions due to the relatively flat topography of the site and the submerged outfall where stormwater discharges to Big Creek. The process of designing a storm drainage network may take multiple iterations in order to determine the correct pipe sizes and slopes to meet design standards and provide positive drainage without flooding.

Step 4. Develop hydraulics calculation form for full-flowing pipe network.

A sample hydraulics calculation form is provided in Attachment E.3-3. Organize the calculation points for the drainage network in sequential rows moving from downstream to upstream with the discharge point serving as the first row. The calculation points should be defined immediately upstream of each structure and pipe segment, as shown in Attachment E.3-1. This form should enable the calculation of head losses and EGL and HGL elevations at each calculation point.

Step 5. Utilize hydraulics calculation form.

Pipe Data

List the conveyance pipe length, diameter, cross-sectional area, and the appropriate Manning's coefficient based on the pipe material. Typical Manning's Coefficients for different types of pipe material can be found in Appendix D.2, Table D.2-4. The pipe network is constructed with reinforced concrete pipe (RCP) with a Manning's value of 0.014 (Appendix D.2, Table D.2-4).

Calculate the velocity head for each computation point using the following equation:

$$V_h = \frac{v^2}{2g}$$

Where,

v is the velocity (ft/sec)
g is the acceleration due to gravity (32.2 ft/sec^2)

Pipe 1 Example Calculation (Attachment E.3-3, Row 3, Column 10):

$$V_h = \frac{v^2}{2g}$$

$$Q = A * V \qquad (\textit{Equation 4.2})$$

$$v = \frac{Q}{A}$$

Q = 12.95 cfs (Attachment E.3-2, Column 14)
A = 1.77 sf (1.5-ft-diameter or 18-inch-diameter pipe)

v = 12.95/1.77
v = 7.33 ft/sec

V_h = (7.33)^2 / (2 * 32.2)
V_h = 0.83 ft

Calculate the frictional slope (S_f) for each pipe segment using *Equation 4.6*:

$$S_f - 4.66 * \frac{n^2 * Q^2}{D^{5.33}} \qquad (\textit{Equation 4.6})$$

Pipe 1 Example Calculation (Attachment E.3-3, Row 3, Column 11):

$$S_f - 4.66 * \frac{n^2 * Q^2}{D^{5.33}}$$

n = 0.014 (Attachment E.3-3)
D = 1.5 ft (Attachment E.3-3)
Q = 12.95 cfs (Attachment E.3-3)

S_f = 4.66 * [(0.014)^2 * (12.95)^2] / (1.5)$^{5.33}$
S_f = 0.0176 ft/ft

Head Losses

Calculate the appropriate head losses for structure and pipe segment, including pipe friction loss, contractions and expansion losses, and bend/other losses. This example includes head losses along the length of the pipe due to friction, flow through manholes (expansion, contraction, and bend losses), and lateral losses (contraction, pipe length, and expansion). Other head loss equations may be warranted depending on the design and outlet conditions. Refer to Section 4.3.3.2 for additional information regarding head loss equations.

Friction Losses

Calculate the friction loss (h_f) through each pipe segment using *Equation 4.8*.

$$h_f = S_f * L \qquad \text{(Equation 4.8)}$$

Pipe 1 Example Calculation (Attachment E.3-3, Row 3, Column 12):

$$h_f = S_f * L$$

S_f = 0.0176 ft/ft
L = 116 ft

h_f = 0.0176 * 116 = 2.05 ft

Manhole Losses

Manhole losses include losses where flow enters the manhole, manhole losses due to a change in flow direction, and contraction losses where flow exits the manhole.

Manhole Losses due to Expansion:

A sudden Expansion Loss occurs where the pipe enters the manhole or structure, equivalent to a Sudden Pipe Enlargement where the ratio of pipe diameters approaches infinity.

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

Where,

h_e is the head loss due to expansion (ft);
k_e is the expansion loss coefficient (dimensionless);
V_1 is the flow velocity upstream of the expansion (ft/sec); and
V_2 is the flow velocity downstream of the expansion (ft/sec).

The flow velocity downstream of the expansion (V_2) is the flow within the manhole or structure and is assumed to be negligible, therefore the *Equation 4.12* becomes:

$$h_e = k_e * \frac{V_1^2}{2g}$$

Pipe 2 entering MH-1 Example Calculation (Attachment E.3-3, Row 6, Columns 6, 13):

The ratio of pipe diameters, D_2/D_1, is calculated using the upstream pipe diameter and the diameter of the downstream manhole

D_2 = 48.00 inches
D_1 = 18.00 inches
D_2 / D_1 = 2.67
V_1 is 5.93 ft/sec, therefore, the expansion loss coefficient is 0.71 (Table D.2-5)

Note: 0.71 is interpolated from values in Table D.2-5.

$$h_e = k_e * \frac{V_1^2}{2g}$$

k_e = 0.71 (Table D.2-5, interpolated)
V_1 = 5.93 ft/sec

$h_e = 0.71 * (5.93^2 / [2*32.2 \text{ cfs}])$

h_e = 0.39 ft

Manhole Losses due to a Change in Flow Direction:

Include this term where the direction of flow changes across a manhole or structure.

$$h_{fd} = k_{fd} * \frac{V^2}{2g}$$

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; and
V is the flow velocity in the conduit (ft/sec).

Pipe 4 enters manhole MH-3 at an angle of 19 degrees. Table 4-5, "Values of k_{fd} for Determining Loss of Head due to Change in Flow Direction," lists loss coefficients due to flow direction changes in manholes. The value closest to 19 degrees (15 degrees), was used to select the associated loss coefficient for this calculation.

MH-3 Example Calculation (Attachment E.3-3, Row 7, Column 14):

$$h_{fd} = k_{fd} * V_h$$

k_{fd} = 0.19 (Table 4-5)
$V_{h\ Pipe\ -4}$ = 0.08 ft

h_{fd} = 0.19*0.08
h_{fd} = 0.01 ft

Manhole Losses due to Contraction Losses

Contraction losses occur where flow exits a manhole or structure (*Equation 4.13*):

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

(*Equation 4.13*)

Where,

h_c is the head loss due to contraction (ft);
k_c is the contraction loss coefficient (dimensionless);
V_1 is the velocity upstream of the contraction (ft/sec); and
V_2 is the velocity downstream of the contraction (ft/sec).

The flow velocity upstream of the contraction (V_1) is the flow within the manhole or structure and is assumed to be negligible; therefore, *Equation 4.13* becomes:

$$h_c = k_c * \frac{V_2^2}{2g}$$

MH-1 exit to Pipe 1 Example Calculation (Attachment E.3-3, Row 4, Column 15):

The ratio of pipe diameters, D_2/D_1, is calculated using the upstream manhole diameter and downstream pipe diameter.

D_2 = 48.00 in
D_1 = 18.00 in
D_2/D_1 = 2.67

V_1 is 7.33 ft/sec, therefore, the expansion loss coefficient is 0.41 (Table D.2-7, interpolated).

$$h_c = k_c * \frac{V_2^2}{2g}$$

$k_c = 0.41$
$V_2 = 7.33$ ft/sec

$h_c = 0.41*(4.33^2/[2*32.2 \text{ ft/sec}^2])$

$h_c = 0.34$ ft

Outlet Losses

Calculate outlet losses, such as where the system discharges to OF-X, using *Equation 4.16*.

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

OF-X Outlet Example Calculation (Attachment E.3-3, Row 1, Column 16):

$$h_O = 1.0 * \frac{7.33^2}{2*32.2 ft / \sec^2}$$

$h_O = 0.83$ ft

Starting at the downstream end of the system, calculate the energy grade line (*EGL*) at the upstream end of each pipe or manhole using the following formula:

$$EGL_{upstream} = EGL_{downstream} + \sum h$$

Where,

Σh is the summation of all head loss, including friction loss, expansion loss, loss due to change in direction, and contraction loss.

$$\sum h = h_f + h_e + h_{fd} + h_c + h_o$$

MH-1 Example Calculation (Attachment E.3-3, Row 5, Column 18):

$$EGL_{HM\text{-}1} = EGL_{OF\text{-}X} + \sum h$$

$EGL_{Outlet} = 365.50$ ft
$\Sigma h = h_o + h_f + h_c$
$\Sigma h = 0.83+2.05+0.34 = 3.22$ ft

$EGL_{MH\text{-}1} = 365.50 + 3.22$
$EGL_{MH\text{-}1} = 368.72$ ft

Calculate the hydraulic grade line (HGL) by subtracting the velocity head from the EGL. Since all pipes are flowing under pressure conditions (water is surcharging in the manhole like a reservoir), the velocity in each manhole is assumed to be zero, therefore: $HGL_{mh} = EGL_{mh}$

$HGL = EGL - V_h$, where V_h = 0 in the manhole (however $HGL \neq EGL$ outside the manhole)

To plot the EGL and HGL (Attachment E.3-4), determine the EGL within the pipe at the upstream and downstream ends, as shown below.

MH-1 EGL/HGL Example Calculation (Attachment E.3-3, Rows 1-5, Columns 18-19):

$$EGL_{Pipe\text{-}1,outlet} = EGL_{OF\text{-}X} + h_o$$

$EGL_{Pipe\ 1,\ outlet}$ = 365.50 + 0.83 = 366.33 ft

$$EGL_{Pipe\text{-}1,inlet} = EGL_{Pipe\text{-}1,outlet} + h_f$$

$EGL_{Pipe\ 1,\ inlet}$ = 366.33 + 2.05 = 368.38 ft

(upstream end of Pipe 1, just downstream of MH-1)

$$HGL_{Pipe\text{-}1,inlet} = EGL_{Pipe\text{-}1,inlet} - V_{hPipe\text{-}1}$$

$HGL_{Pipe\ 1,\ Inlet}$ = 368.38 – 0.83 = 367.55 ft

$$EGL_{MH\text{-}1} = EGL_{Pipe\text{-}1,inlet} + h_c$$

$EGL_{MH\text{-}1}$ = 368.38 + 0.34 = 368.72 ft

$$HGL_{MH\text{-}1} = EGL_{MH\text{-}1} + V_{hMH\text{-}1}$$

$HGL_{MH\text{-}1}$ = 368.72 – 0 = 368.72 ft

The EGL elevation at a manhole or catch basin should be designed to be at least 1 ft below the Rim elevation of the same structure. If the EGL is less than 1 ft below the manhole or catch basin Rim elevation, the downstream pipe segment should be modified, usually by increasing the pipe diameter. In this case, the EGL at MH-1 is 368.72, while the Rim elevation is 372.00, meeting the 1-ft requirement. The same check is used for catch basins and their lateral connections to manholes. After the head loss is determined in the catch basin and lateral, the EGL at the catch basin can be calculated. In this example, the EGL at catch basin CB-1A is calculated to be 368.77, while the Rim elevation is 371.67, also meeting the 1-ft requirement. A sample profile is provided in Attachment E.3-4.

Designing a storm drainage network where the EGL is greater than 1 ft below the rim of a respective catch basin or manhole is an iterative process. Storm pipes are preliminarily sized based upon hydrologic conditions and the full pipe capacity under free-flowing conditions. When determining actual hydraulic conditions and establishing the EGL it will be necessary to iterate pipe sizes, slopes and even pipe material type in order to size the storm drainage network under surcharged conditions to meet design standards. As shown in this problem a good starting point is to make sure that the free flowing full pipe capacity at the selected slope is greater than the stormwater runoff entering the system. Then use the approach outlined here to model the surcharged outlet condition and iterate pipe diameters and other factors to arrive at a design that meets jurisdictional standards and provides reliable conveyance without flooding at the design storm event.

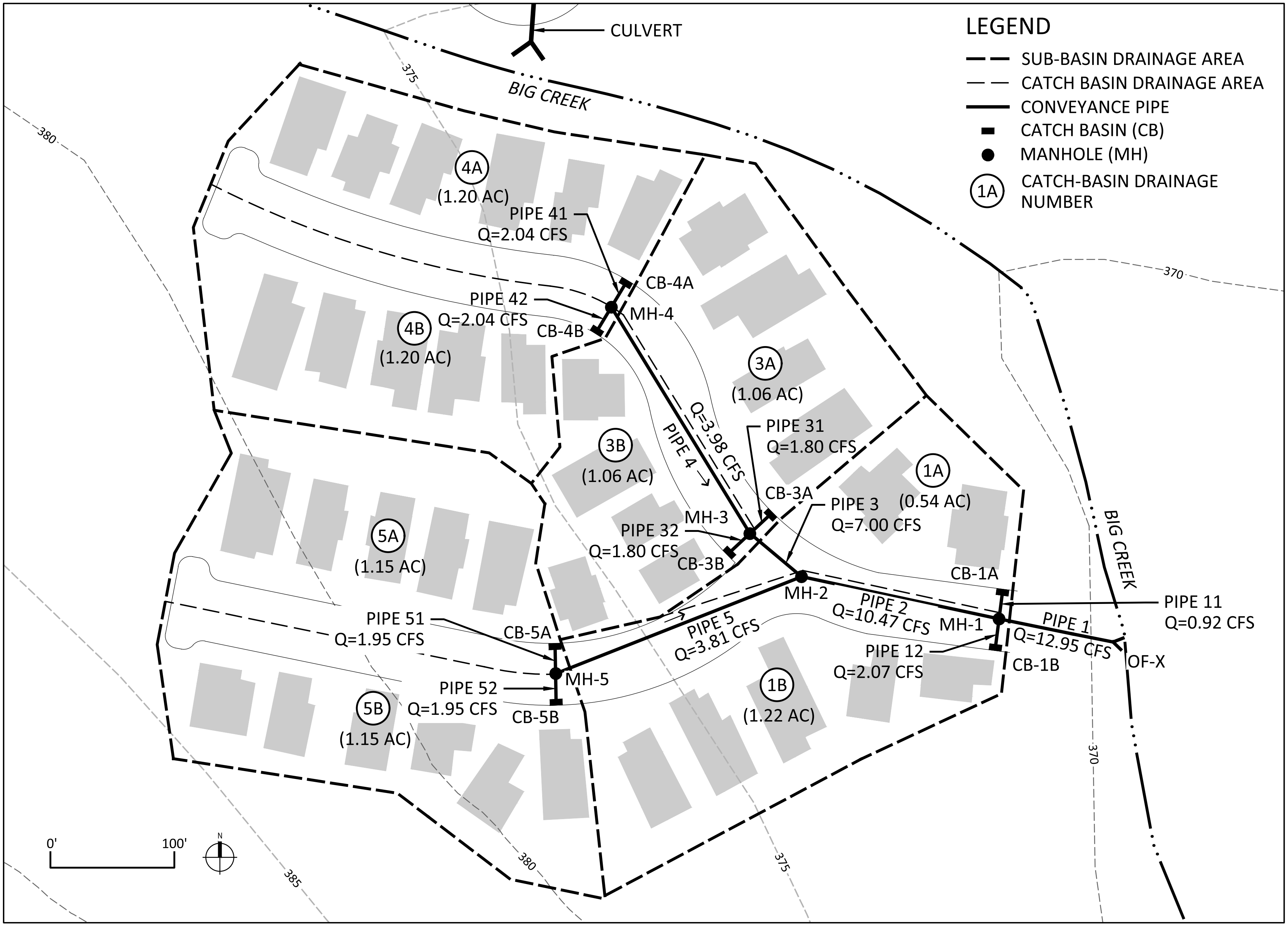
The figures and calculations included in a submittal to Sonoma Water for this example would include all of the information types listed in the Flood Management Design Review Plan Submittal Checklist (Appendix B.2), as indicated in parentheses below and shown in Appendix E.3.

Attachment E.3-1. Blue Heron South Drainage (Hydrology Map)
Attachment E.3-2. Blue Heron South Hydrology Calculations for Underground Pipe Drainage (Hydrology Calculations; Coefficients and Parameters)
Attachment E.3-3. Blue Heron South Hydraulics Calculations for Underground Pipe Drainage (Hydraulic Calculations; Coefficients and Parameters)
Attachment E.3-4. Sample Storm Drainage Profile MH-4 to OF-X (EGL and HGL plot)

A clear and complete submittal will expedite the review process.

Attachment E.3

ATTACHMENT E.3-1
Blue Heron South Drainage



ATTACHMENT E.3-2

Blue Heron South Hydrology Calculations for Underground Pipe Drainage

10-Year Storm Event			Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6	Col. 7	Col. 8	Col. 9	Col. 10	Col. 11	Col. 12	Col. 13	Col. 14
ID	**Pipe Segment**	**Drainage**	**A**	**ΣA**	**Tp**	**Tc**	**I**	**C**	**C*A**	**ΣCA**	**D**	**Q_{full}**	**L**	**S**	**V**	**Q_{Runoff}**
			(acres)	(acres)	(min)	(min)	(in/hr)				(in)	(cfs)	(ft)	(ft/ft)	(fps)	(cfs)
CB-4A		4A	1.20	1.20	-	10.00	2.39	0.71	0.85	0.85	-	-	-	-	-	2.04
Pipe 41	CB-4A to MH-4	4A	-	1.20	0.10	10.00	2.39	0.71	-	0.85	12.00	4.06	15.00	0.0150	2.59	2.04
CB-4B		4B	1.20	1.20	-	10.00	2.39	0.71	0.85	0.85	-	-	-	-	-	2.04
Pipe 42	CB-4B to MH-4	4B	-	1.20	0.10	10.00	2.39	0.71	-	0.85	12.00	4.69	15.00	0.0200	2.59	2.04
Pipe 4	MH-4 to MH-3	4A&B	2.40	2.40	1.56	10.10	2.33	0.71	1.70	1.70	18.00	6.92	210.00	0.0050	2.25	3.98
CB-3A		3A	1.06	1.06	-	10.00	2.39	0.71	0.75	0.75	-	-	-	-	-	1.80
Pipe 31	CB-3A to MH-3	3A	-	1.06	0.11	10.00	2.39	0.71	-	0.75	12.00	4.06	15.00	0.0150	2.29	1.80
CB-3B		3B	1.06	1.06	-	10.00	2.39	0.71	0.75	0.75	-	-	-	-	-	1.80
Pipe 32	CB-3B to MH-3	3B	-	1.06	0.11	10.00	2.39	0.71	-	0.75	12.00	4.06	15.00	0.0150	2.29	1.80
Pipe 3	MH-3 to MH-2	3A&B, 4A&B	2.12	4.52	0.23	11.65	2.18	0.71	1.51	3.21	18.00	11.78	55.00	0.0145	3.96	7.00
CB-5A		5A	1.15	1.15	-	10.00	2.39	0.71	0.82	0.82	-	-	-	-	-	1.95
Pipe 51	CB-5A to MH-5	5A	-	1.15	0.10	10.00	2.39	0.71	-	0.82	12.00	4.69	15.00	0.0200	2.48	1.95
CB-5B		5B	1.15	1.15	-	10.00	2.39	0.71	0.82	0.82	-	-	-	-	-	1.95
Pipe 52	CB-5B to MH-5	5B	-	1.15	0.10	10.00	2.39	0.71	-	0.82	12.00	4.69	15.00	0.0200	2.48	1.95
Pipe 5	MH-5 to MH-2	5A&B	2.30	2.30	1.62	10.10	2.33	0.71	1.63	1.63	18.00	6.41	210.00	0.0043	2.16	3.81
Pipe 2	MH-2 to MH-1	3A&B, 4A&B, 5A&B	-	6.82	0.45	11.88	2.16	0.71	-	4.84	18.00	11.61	160.00	0.0141	5.93	10.47
CB-1A		1A	0.54	0.54	-	10.00	2.39	0.71	0.38	0.38	-	-	-	-	-	0.92
Pipe 11	CB-1A to MH-1	1A	-	0.54	0.21	10.00	2.39	0.71	-	0.38	12.00	4.06	15.00	0.0150	1.17	0.92
CB-1B		1B	1.22	1.22	-	10.00	2.39	0.71	0.87	0.87	-	-	-	-	-	2.07
Pipe 12	CB-1B to MH-1	1B	-	1.22	0.09	10.00	2.39	0.71	-	0.87	12.00	4.92	15.00	0.0220	2.64	2.07
Pipe 1	MH-1 to OF-X	1A&B, 3A&B, 4A&B, 5A&B	1.76	8.58	0.26	12.33	2.13	0.71	1.25	6.09	18.00	14.51	116.00	0.0220	7.33	12.95

Col. 1: Contributing Area, Att. E.3-1
Col. 2: Total Contributing Area
Col. 3: Pipe travel time, Tp = velocity/pipe length (Eqn 3.10)
Col. 4: Time of concentration, Tc = Tp + Tc upstream
Col. 5: Rainfall Intensity, Att. E.1-4
Col. 6: Table C-1
Col. 7 C*A

Col. 8: Summation of C*A
Col. 9: Pipe Diameter
Col. 10: Qfull = (1.49/n)*[(((D/12)/4))^(2/3)]*[S^(1/2)]*(∏/4)*(D/12)^2 (Eqn 3.8. Eqn 4.2)
Col. 11: Pipe Length
Col. 12: Pipe Slope
Col. 13: Velocity, V = Q/((∏*(D/12)^2)/4) (Eqn 4.2)
Col. 14 Qrunoff = C*I*A

Note: Check that Q full > Q Runoff. If Q Full < Q Runoff, you may need to increase pipe diameter or increase slope or change to a smoother pipe type.

ATTACHMENT E.3-3

Blue Heron South Hydraulics Calculations for Underground Pipe Drainage

			Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6	Col. 7	Col. 8	Col. 9	Col. 10	Col. 11	Col. 12	Col. 13	Col. 14	Col. 15	Col. 16	Col. 17	Col. 18	Col. 19	Col. 20
	Comp. Pt.	Pipe ID	Pipe Invert Elev.	Pipe Length	Pipe Slope	n	Pipe, MH, CB Dia	D_2/D_1	XS Area	Q	V	Vel. Head (v_h)	Friction Slope (S_f)	Friction Loss (h_f)	Expansion Loss (k_e) h_e	Flow Direction Losses (k_{fd}) h_{fd}	Contraction Loss (k_c) h_c	Outlet Loss (h_O)	Total Head Loss	EGL	HGL	Rim Elev.
			(ft)	(ft)	(ft/ft)		(inches)	(inch/inch)	(sf)	(cfs)	(fps)	(ft)	(ft/ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)			
Row 1	OF-X																	0.83	0.83	365.50	365.50	-
Row 2		Outlet	362.85																0.00	366.33	365.50	-
Row 3		Pipe 1		116.00	0.0220	0.014	18.00		1.77	12.95	7.33	0.83	0.0176	2.05					2.05	-	-	-
Row 4		Inlet	365.40					2.67									(0.41) 0.34		0.34	368.38	367.55	-
Row 5	MH-1						48.00													368.72	368.72	372.00
		Outlet	365.60					4.00							(0.92) 0.02	(0.70) 0.01			0.03	368.75	368.73	-
		Pipe 11		15.00	0.0150	0.014	12.00		0.79	0.92	1.17	0.02	0.0008	0.01					0.01	-	-	-
		Inlet	365.83					2.00									(0.38) 0.01		0.01	368.76	368.74	-
	CB-1A						24.00												-	368.77	368.77	371.67
		Outlet	365.70					4.00							(0.90) 0.10	(0.70) 0.08			0.17	368.89	368.78	-
		Pipe 12		15.00	0.0220	0.014	12.00		0.79	2.07	2.64	0.11	0.0039	0.06					0.06	-	-	-
		Inlet	366.03					2.00									(0.38) 0.04		0.04	368.95	368.84	-
	CB-1B						24.00												-	368.99	368.99	371.67
Row 6		Outlet	365.50					2.67							(0.71) 0.39				0.39	369.11	368.56	-
		Pipe 2		160.00	0.0141	0.014	18.00		1.77	10.47	5.93	0.55	0.0115	1.85					1.85	-	-	-
		Inlet	367.75					2.67									(0.42) 0.23		0.23	370.96	370.41	-
	MH-2						48.00													371.19	371.19	373.20
		Outlet	367.85					2.67							(0.73) 0.18	(0.35) 0.09			0.26	371.45	371.21	-
		Pipe 3		55.00	0.0145	0.014	18.00		1.77	7.00	3.96	0.24	0.0052	0.28					0.28	-	-	-
		Inlet	368.65					2.67									(0.43) 0.10		0.10	371.73	371.49	-
		Outlet	367.95					2.67							(0.78) 0.06	(0.35) 0.03			0.08	371.27	371.20	-
		Pipe 5		210.00	0.0043	0.014	18.00		1.77	3.81	2.16	0.07	0.0015	0.32					0.32	-	-	-
		Inlet	368.85					2.67									(0.43) 0.03		0.03	371.59	371.52	-

Note: values shown in this table are the result of spreadsheet calculations, and therefore do not reflect the same degree of rounding that would occur in manual calculations.

Col. 1: Pipe Outlet and Inlet Invert Elevation

Col. 2: Att. E.3-1

Col. 3: Slope, S = (US IE - DS IE)/Pipe Length

Col. 4: Manning's n, Table D.2-4 Polyethene (smooth) pipe

Col. 5: Diameter of Pipe, Manhole, or Catch Basin

Col. 6: Ratio of pipe/structure diameters, used with Col. 9 to determine k_c and k_e (Eqns. 4.12 and 4.11)

Col. 7: Cross-Sectional Area, $A = 3.14 * (D/2/12)^2$

Col. 8: Runoff, Q, see Att. E.3-2, Column 14

Col. 9: Velocity, V = Q/A

Col. 10: Velocity Head, $V_h = V^2 / (2 * 32.2)$

Col. 11: $S_f = 4.66 * [n^2 * Q^2] / (D/12)^{5.33}$

Col. 12: Friction Loss, $h_f = L * S_f$

Col. 13: Expansion Loss where pipe enters manhole.

Col. 14: Flow Direction Loss, Losses due to Change in Flow Direction through the manhole (See Section 4.3.3.2)

Col. 15: Contraction Loss where pipe exits manhole or catch basin

Col. 16: Outless Loss, $h_o = 1.0 * V_h$

Col. 17: Total Head Loss = $h_f + h_e + h_{fd} + h_c + h_O$

Col. 18: Energy Grade Line EGL = $EGL_{downstream}$ + total headloss

Col. 19: Hydraulic Grade Line, HGL = EGL - v_h

Col. 20: Rim Elevation of Structure

Attachment E.3-3 (continued)

			Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6	Col. 7	Col. 8	Col. 9	Col. 10	Col. 11	Col. 12	Col. 13	Col. 14	Col. 15	Col. 16	Col. 17	Col. 18	Col. 19	Col. 20
	Comp. Pt.	Pipe ID	Pipe Invert Elev.	Pipe Length	Pipe Slope	n	Pipe, MH, CB Dia	D_2/D_1	XS Area	Q	V	Vel. Head (v_h)	Friction Slope (S_f)	Friction Loss (h_f)	Expansion Loss (k_e) h_e	Flow Direction Losses (k_{fd}) h_{fd}	Contraction Loss (k_c) h_c	Outlet Loss (h_O)	Total Head Loss	EGL	HGL	Rim Elev.
			(ft)	(ft)	(ft/ft)		(inches)	(inch/inch)	(sf)	(cfs)	(fps)	(ft)	(ft/ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)			
	MH-3						48.00													371.83	371.83	374.75
		Outlet	368.85					4.00							(0.91) 0.07	(0.70) 0.06			0.13	371.96	371.88	-
		Pipe 31		15.00	0.0150	0.014	12.00		0.79	1.80	2.29	0.08	0.0030	0.04					0.04	-	-	-
		Inlet	369.07					2.00									(0.38) 0.03		0.03	372.00	371.92	-
	CB-3A						24.00												-	372.03	372.03	374.70
		Outlet	368.95					4.00							(0.91) 0.07	(0.70) 0.06			0.13	371.96	371.88	-
		Pipe 32		15.00	0.0150	0.014	12.00		0.79	1.80	2.29	0.08	0.0030	0.04					0.04	-	-	-
		Inlet	369.17					2.00									(0.38) 0.03		0.03	372.00	371.92	-
	CB-3B						24.00												-	372.03	372.03	374.42
Row 7		Outlet	368.75					2.67							(0.78) 0.06	(0.19) 0.01			0.08	371.91	371.83	-
		Pipe 4		210.00	0.0050	0.014	18.00		1.77	3.98	2.25	0.08	0.0017	0.35					0.35	-	-	-
		Inlet	369.80					2.67									(0.43) 0.03		0.03	372.26	372.18	-
	MH-4						48.00													372.29	372.29	375.50
		Outlet	369.90					4.00							(0.90) 0.09	(0.70) 0.07			0.17	372.46	372.36	-
		Pipe 41		15.00	0.0150	0.014	12.00		0.79	2.04	2.60	0.10	0.0038	0.06					0.06	-	-	-
		Inlet	370.12					2.00									(0.38) 0.04		0.04	372.52	372.42	-
	CB-4A						24.00												-	372.56	372.56	375.30
		Outlet	370.00					4.00							(0.90) 0.09	(0.56) 0.06			0.15	372.44	372.34	-
		Pipe 42		15.00	0.0200	0.014	12.00		0.79	2.04	2.60	0.10	0.0038	0.06					0.06	-	-	-
		Inlet	370.30					2.00									(0.38) 0.04		0.04	372.50	372.40	-
	CB-4B						24.00												-	372.54	372.54	375.40
	MH-5						48.00													371.62	371.62	375.00
		Outlet	368.95					4.00							(0.91) 0.09	(0.64) 0.06			0.15	371.77	371.67	-
		Pipe 51		15.00	0.0200	0.014	12.00		0.79	1.95	2.48	0.10	0.0035	0.05					0.05	-	-	-
		Inlet	369.25					2.00									(0.38) 0.04		0.04	371.82	371.72	-
	CB-5A						24.00												-	371.86	371.86	374.67
		Outlet	369.05					4.00							(0.90) 0.09	(0.70) 0.07			0.15	371.77	371.67	-
		Pipe 52		15.00	0.0200	0.014	12.00		0.79	1.95	2.48	0.10	0.0035	0.05					0.05	-	-	-
		Inlet	369.35					2.00									(0.38) 0.04		0.04	371.82	371.72	-
	CB-5B						24.00												-	371.86	371.86	374.67

Col. 1: Pipe Outlet and Inlet Invert Elevation

Col. 2: Att. E.3-1

Col. 3: Slope, S = (US IE - DS IE)/Pipe Length

Col. 4: Manning's n, Table D.2-4 Polyethene (smooth) pipe

Col. 5: Diameter of Pipe, Manhole, or Catch Basin

Col. 6: Ratio of pipe/structure diameters, used with Col. 9 to determine k_c and k_e (Eqns. 4.12 and 4.13)

Col. 7: Cross-Sectional Area, $A = 3.14 * (D/2/12)^2$

Col. 8: Runoff, Q, see Att. E.3-2, Column 14

Col. 9: Velocity, V = Q/A

Col. 10: Velocity Head, $V_h = V^2 / (2 * 32.2)$

Col. 11: $S_f = 4.66 * [n^2 * Q^2] / (D/12)^{5.33}$

Col. 12: Friction Loss, $h_f = L * S_f$

Col. 13: Expansion Loss where pipe enters manhole.

Col. 14: Flow Direction Loss, Losses due to Change in Flow Direction through the manhole (See Section 4.3.3.2)

Col. 15: Contraction Loss where pipe exits manhole or catch basin

Col. 16: Outless Loss, $h_o = 1.0 * V_h$

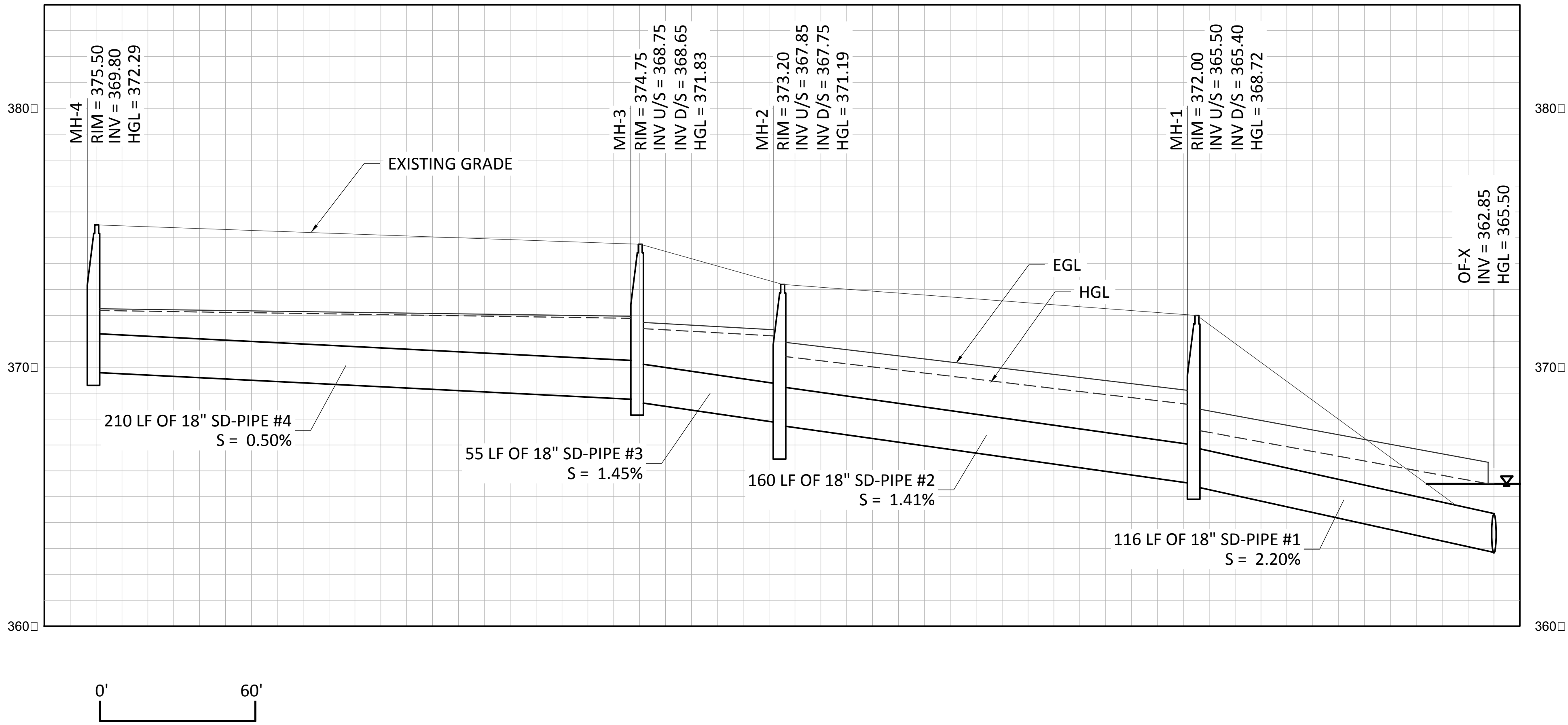
Col. 17: Total Head Loss = $h_f + h_e + h_{fd} + h_c + h_O$

Col. 18: Energy Grade Line EGL = $EGL_{downstream}$ + total headloss

Col. 19: Hydraulic Grade Line, HGL = EGL - v_h

Col. 20: Rim Elevation of Structure

ATTACHMENT E.3-4
Sample Storm Drainage Profile MH-4 to OF-X



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