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Designing Watercourse Crossings for Passage of 100-Year Flood Flows, Wood, and Sediment (Updated 2017)

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Executive Summary

Watercourse crossings associated with timber harvesting operations can produce substantial amounts of sediment which can be directly delivered into streams. To reduce the potential for crossing failures and resulting impacts, the California Forest Practice Rules applicable to non-federal timberlands have specified since July 2000 that all constructed or reconstructed permanent watercourse crossings must accommodate the estimated 100-year flow, including debris and sediment loads.

Four suggested methods for making office-based estimates of one-hundred year recurrence interval peak discharges (i.e., 100-year flood flows) in ungaged basins are presented: (1) an analytical relationship between storm precipitation, watershed characteristics, and runoff, (2) updated regional regression equations based on long-term flow records, (3) flow transference methods that adjust nearby measured discharges for differences in drainage basin size, and (4) computer models currently available for more complicated situations. Watershed area limitations for the first three methods are also identified. In general, flow transference methods are preferred for determining 100-year flood flows in drainage basins where nearby long-term stream gaging station data are available, because local streamflow data are more likely to represent drainage-basin characteristics that determine peak flows than analytical relationships or regional regression equations. The estimated 100-year flood flow value is then used to determine crossing dimensions (e.g., culvert diameter) large enough to handle this estimated peak flow, as well as accommodate flood-associated wood and sediment loads.

Research conducted in northwestern California and the Pacific Northwest has shown that culverts fail less often from flood flows alone than from accumulations of wood and sediment that commonly accompany flood flows. Foresters designing watercourse crossings are therefore required to design crossings to handle flood-associated sediment and debris in addition to the estimated peak flows. Several techniques are suggested to decrease the risk of crossing failure from wood and sediment plugging. Other issues related to fish passage through culverts are covered elsewhere in the literature and also need to be considered in crossing design for fish-bearing streams.

Crossing capacity determined from estimated peak flows needs to be checked in the field by making direct channel cross-section measurements. The 3 times (3 X) bankfull area method is suggested as one approach for field verification, but has only been validated for the rain-dominated North Coast region of California and is not appropriate for the interior part of the state subject to rain-on-snow events. Annual high flow line or active channel width measurements are alternatives for small, more entrenched channels, channels dominated by spring flow, or channels with resistant boundaries where bankfull characteristics may be poorly developed. A field-based approach relying on channel morphology might be more appropriate than office-based techniques for spring-dominated watercourses.

In this revised 2017 edition of the 2004 Report, we include (1) considerations for crossings in post-fire environments; (2) approaches to address high risk crossings with large fills; (3) methodologies for designing rock-armored crossings, including how to size rock riprap to withstand overtopping 100-year flood flows; and (4) considerations for permanent bridge design in forested watersheds.

Examples are displayed in appendices to illustrate how to use the methods presented. Watershed data from two small tributary basins located in California are included in Appendix A to show how to apply the watercourse crossing sizing techniques presented for culverts. For a Caspar Creek watershed tributary located near Fort Bragg, California, 100-year flood flows are estimated using several office methods and field checked using the 3 X bankfull area method. Additionally, the various discharge-estimating office techniques for ungaged basins are used to estimate a 10-year peak flow, and these results are compared to gaging station data. For a Teakettle Creek tributary located in the southern Sierra Nevada east of Fresno, California, the 100-year flood flow is estimated with multiple office methods and compared to a flood frequency analysis estimate for a 100-year runoff event.

Appendix B provides watercourse crossing definitions and diagrams and Appendix C includes tables of crossing fill heights and fill volumes. Appendix D provides rock-armored crossing design information, including (1) examples for sizing rock riprap for overtopping 100-year flood flows, and (2) detailed information on the U.S. Bureau of Reclamation/Colorado State University (USBR/CSU) methodology for sizing rock riprap to withstand overtopping flows. Appendix E presents permanent bridge design information, including an example of a bridge design project.

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List of Abbreviations

AASHTO	American Association of State Highway and Transportation Officials
BCMOF	British Columbia Ministry of Forests
BLM	Bureau of Land Management
Board	California State Board of Forestry and Fire Protection
CAL FIRE	California Department of Forestry and Fire Protection
Caltrans	California Department of Transportation
CBC	California Building Code
CCR	California Code of Regulations
CEG	Certified Engineering Geologist
CFS	Cubic feet per second
CGS	California Geological Survey
CHG	Certified Hydrogeologist
CSU	Colorado State University
CVRWQCB	Central Valley Regional Water Quality Control Board
DEM	Digital Elevation Model
DFW	California Department of Fish and Wildlife
DWR	California Department of Water Resources
FAA	Federal Aviation Administration
FPRs	California Forest Practice Rules
GE	Geotechnical Engineer
GIS	Geographic Information System
GRS	Geosynthetic reinforced soil
HDPE	High Density Polyethylene Pipe
HEC-HMS	USACE Hydrologic Engineering Center- Hydrologic Modeling System
HEC-SSP	USACE Hydrologic Engineering Center- Statistical Software Package
HW/D	Headwater Depth/Pipe Diameter
H:V	Horizontal: Vertical
IDF	Intensity Duration Frequency
LWD	Large Woody Debris
MSE	Mechanically stabilized earth
NCASI	National Council for Air and Stream Improvement
NCRWQCB	North Coast Regional Water Quality Control Board
NOAA	National Oceanic and Atmospheric Administration
NRCS	Natural Resources Conservation Service
NSS	National Streamflow Statistics
OSU	Oregon State University
PE	Professional Engineer
PG	Professional Geologist
PH	Professional Hydrologist (American Institute of Hydrology)
PRC	Public Resources Code
PRISM	Parameter-Elevation Regressions on Independent Slopes Model
RI	Recurrence Interval (or return period)
RNSP	Redwood National and State Parks
RPF	Registered Professional Forester
RWQCBs	Regional Water Quality Control Boards
SCS	Soil Conservation Service (now NRCS)

SE	Standard Error
SEE	Standard Error Estimate
THP	Timber Harvesting Plan
TOC	Time of Concentration
TR-55	NRCS Technical Release 55
USACE	U.S. Army Corps of Engineers
USBR	U.S. Bureau of Reclamation
USEPA	U.S. Environmental Protection Agency
USFS PSW	U.S. Forest Service Pacific Southwest Research Station
USGS	U.S. Geological Survey

I. Introduction

Timberland owners and foresters have been required by the California Forest Practice Rules to design all new and reconstructed permanent watercourse crossings to accommodate an estimated 100-year flood flow, including wood and sediment loads, since July 2000. As a result of this change, in 2004 the first version of this document was produced (Cafferata et al. 2004). The 100-year flood flow and associated requirements were made a permanent regulation of the Forest Practice Rules beginning in January 2015 with the passage of the Road Rules, 2013 rule package.

Hillslope monitoring work conducted throughout California's forestlands has shown that water quality-related problems frequently occur at watercourse crossings (Cafferata and Munn 2002, Staab 2004, Brandow et al. 2006, USFS 2009, USFS 2013, Brandow and Cafferata 2014). Inadequate design was cited as one of the primary reasons for these results. While culverts have been commonly sized to accommodate some level of flood flow, studies in northwestern California have shown that flood discharge alone is usually not the primary cause of crossing failures (Furniss et al. 1998; Flanagan 2004; Flanagan, unpublished data, Figure 1). To date, similar studies have not been completed outside of northwestern California to determine if these results apply elsewhere in the state. Furniss et al. (1998) conclude that "because stream crossing failure in Pacific Northwest forested watersheds is caused predominantly by accumulations of sediment and debris at the inlet, hydraulic models are not reliable predictors of crossing failure."

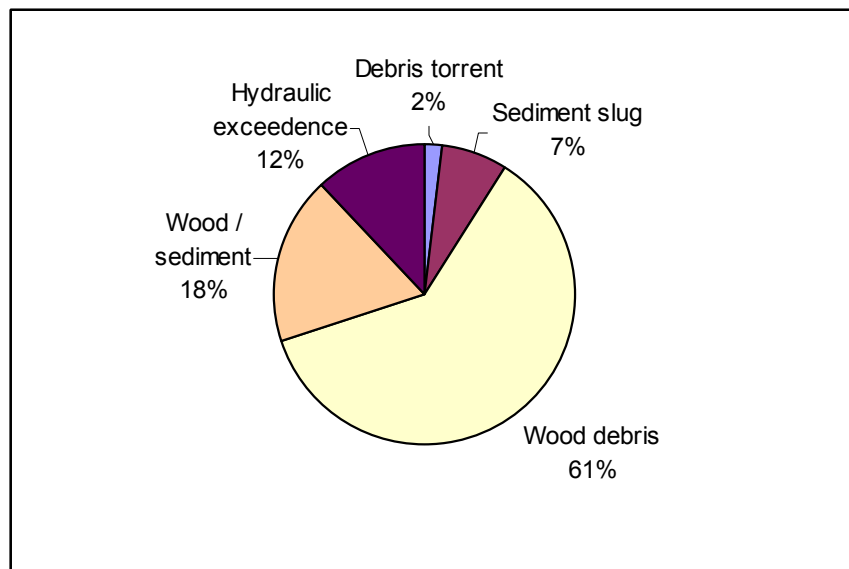


Figure 1. Failure mechanisms for culverts occurring along forest roads in northwestern California associated with storm events with recurrence intervals less than approximately 12 years (S. Flanagan, BLM, Arcata, CA, unpublished information; n = 57). Note that the specific distribution of failure mechanisms will vary depending on numerous factors, including storm intensity and watershed characteristics. See Furniss et al. (1998) for additional information on failure mechanisms following very large floods in the Pacific Northwest and northern California.

Chapter II of this report presents four office techniques for estimating the 100-year flood flow:² (1) the **Rational Method** (Chow 1964, Dunne and Leopold 1978, CDF 1983, Weaver and Hagans 1994, Weaver et al. 2015), (2) updated **USGS regional regression equations** for estimating the magnitude and frequency of floods in California (Gotvald et al. 2012), (3) **flow transference methods** (Waananen and Crippen 1977, Skaugset and Pyles 1991), and (4) **computer models** (e.g., NRCS TR-55, USACE HEC-HMS, WinXSPRO for the slope-area method using Manning's Equation) for more complex sites and watershed conditions.

The 2017 California Forest Practice Rules specify that flood flows are to be estimated by flood flow measurement records and by empirical relationships between precipitation, watershed characteristics, and runoff, and may be modified by direct channel cross-section measurements informed by local experience (CAL FIRE 2017). The Rational Method, USGS updated regional regression equations, flow transference methods, and computer modeling can be used to meet the first part of this requirement, while the second part can be addressed with field methods such as 3 X bankfull area in the California Coast Ranges, or where bankfull area characteristics are difficult to determine, using the annual high flow line or active channel width. A discussion of these field techniques for evaluating proposed crossing design is presented in Chapter III.

Chapter IV describes how to design watercourse crossings for adequate passage of wood and sediment, reducing the likelihood of catastrophic crossing failure. Additional design considerations, and approaches for evaluating the failure risk of crossings, including those located in watersheds burned by wildfires and those with large fills, are included in Chapters V, VI, VII, and VIII.

While monitoring work shows that culverts still account for nearly 70 percent of the permanent crossings installed on non-federal timberlands in California (Brandow and Cafferata 2014), other types of crossings have been installed at higher rates in the past 15 years. In particular, large numbers of rock fords and rock-armored crossings have been installed in headwater streams, although there has been limited guidance available on sizing rock riprap to withstand overtopping 100-year flood flows. Additionally, numerous legacy culverts have been replaced with bridges to accommodate fish passage. Guidance is provided for proper design of these types of crossings in Chapters IX and X of this revised edition.

Although proper watercourse crossing design is critical to ensure adequate passage of water, sediment, and wood, the most important method for reducing environmental impacts is to locate roads to avoid or minimize crossings (Keller and Sherar 2003, Weaver et al. 2015).³ Proper location of roads, and hence

² The 100-year flood flow has a one percent chance of being equaled or exceeded in any given year, and a 26% chance of being equaled or exceeded during a 30-year period.

³ Weaver et al. (2015) provide detailed information on the various types watercourse crossings available for forested watersheds, including information on round culverts and pipe-arch culverts

crossings, reduces both chronic sediment impacts and the potential for catastrophic failure. Higher, flatter, and drier locations require fewer and smaller watercourse crossings than sites low on hillslopes. Where there are many connections between roads and streams, impacts are inevitable, but where roads are distant from streams, their impacts are greatly reduced (Furniss et al. 2000, Jones et al. 2000).

Finally, some of the concepts included in this guidance document are complex. Resource professionals using this reference are reminded not to design watercourse crossings beyond their level of expertise and capabilities, protecting against professional negligence and possible life-safety threats.

II. Office Techniques for Determining the 100-Year Flood Flow

The first step in designing an appropriate watercourse crossing is estimating the 100-year flood flow at a given site with one or more appropriate office techniques. Each of the four methods discussed below has unique advantages and disadvantages for particular situations.

II.1 Rational Method

The Rational Method is an analytical approach for predicting peak runoff rates that has been used for engineering calculations for more than 150 years (Chow 1964, Portland Cement Association 1964, Dunne and Leopold 1978, Rossmiller 1980, Rosbjerg et al. 2013). This method was developed before long-term flow records became widely available. It remains one of the most widely used approaches around the world to estimate design floods in small ungaged watersheds (Rosbjerg et al. 2013). The Rational Method is frequently used for flood prediction in urban watersheds, where most of the storm flow travels as overland flow on impermeable surfaces, and for small (less than 200 acres) undeveloped watersheds (Dunne and Leopold 1978).

The Rational Method equation for the 100-year flood flow is stated as follows:

$$Q_{100} = CIA$$

where:

Q_{100}	=	predicted peak runoff from a 100-year storm in cfs
C	=	runoff coefficient ⁴
I	=	rainfall intensity for the 100-year storm in inches per hour
A	=	basin drainage area in acres

(e.g., durability, alignment, inlet and outlet treatments), open bottom arches, bridges, fords, vented fords, and armored fill crossings. Appendix A in Weaver et al. (2015) provides a summary of the methods available to estimate 100-year flood flows.

⁴The runoff coefficient is dimensionless because it represents the estimated proportion of rainfall that becomes runoff. Note that no proportionality constant is needed when the Rational Method equation is computed using English units because one acre-inch/hour of precipitation is equal to 1.008 cfs.

To determine the rainfall intensity variable in the Rational Method equation, one must (1) determine the time of concentration for the drainage basin upstream of the watercourse crossing, and (2) use intensity-duration-frequency (IDF) rainfall data to identify the 100-year return period rainfall for a storm duration equivalent to the time of concentration.

For a forested watershed, the greatest difficulty is determining the time of concentration so that the rainfall intensity can be estimated. Two methods are commonly used: the California culvert practice equation (Cal Div of Highways 1944--modified Kirpich (1940) equation, Rossmiller 1980, Subramanya 2008), and the FAA Airport Drainage formula (FAA 1970).⁵ Both formulas and examples using the equations are presented in Appendix A. A minimum value of 10 minutes is recommended for the time of concentration for small forested basins with both equations; smaller values tend to overestimate predicted runoff and rainfall-depth-duration data for five minutes are rarely available (Yee 1995).⁶ With the California culvert practice equation, the time of concentration is calculated from the channel length and elevation change from the top of the basin to the watercourse crossing, both of which can be obtained from topographic maps. The Airport Drainage formula incorporates the runoff coefficient (C), in addition to upstream watershed gradient and runoff distance, and generally produces longer estimates for the time of concentration.⁷

Cafferata and Reid (2013) reported that the time of concentration estimates generated by both the California culvert practice and Airport Drainage equations were shorter than those expected for a small headwater forested basin located in the northern part of the California Coast Ranges. They state that “because hillslopes in the watershed contribute runoff through subsurface flow and saturation overland flow, both of which respond more slowly than Horton overland flow, methods that assume that runoff is generated primarily by Horton overland flow are likely to underestimate flow times and so overestimate peak discharges.”⁸ A method of estimating time of concentration using flow path lengths and likely flow velocities for various segments of the flow path (Dunne

⁵ Improved methods for determining the time of concentration have been developed by Papadakis and Kazan (1987) and Loukas and Quick (1996). These approaches use empirically calibrated kinematic wave equations specifically designed to determine the time of concentration for small rural watersheds and have been adopted by several hydrology manuals. While they are improved methods, the equations must be iteratively solved. There remains a need for a simple method.

⁶ San Diego County (2003) recommends assuming an initial time of concentration of 7 to 13 minutes based on slope and cover and adding that time to the travel time calculated using the Kirpich equation. Rossmiller (1980) suggests multiplying the time of concentration obtained using the Kirpich equation by a factor of two. Typically, for watersheds of less than about 100 acres where the Kirpich equation is appropriate, the $T_c + 10$ min. (San Diego County) and the $T_c * 2$ render roughly the same rainfall intensity, making Rational Method flow results more consistent with those from other methods.

⁷ Yee (1995) recommended the use of the Airport Drainage equation to calculate the time of concentration over the California culvert practice method.

⁸ Loukas and Quick (1996) compared several time of concentration formulas, including the Kirpich method, and found all of them to significantly underestimate measured values for two forested mountainous watersheds in British Columbia.

and Leopold 1978) is provided in Appendix A, as well as Appendix D in Cafferata and Reid (2013).

The British Columbia Ministry of Environment, Lands and Parks (BC Government 1991) recommended the use of the Kirpich equation for urban watersheds and an empirical chart for forested areas. Use of the chart appears to provide more reasonable time of concentration values for small forested watersheds than either the Kirpich, California culvert practice, or Airport Drainage methods (see examples in Appendix A). This method results in 100-year peak flow estimates that more closely match those obtained by the USGS regression equations for watersheds; however, details on the input parameters used to develop the chart are not well known and it remains uncertain how well the chart can be extrapolated and applied in California's forested landscapes.

Once the time of concentration is known, the rainfall intensity for a storm with a 100-year recurrence interval (RI) and a duration equal to the time of concentration can be obtained from available intensity-duration-frequency (IDF) data. To obtain rainfall intensity data for a 100-year recurrence interval event for short time periods, we suggest using the NOAA Atlas 14 Point Precipitation Frequency Estimates for California, available on the NOAA webpage at: http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=ca

A major benefit to using NOAA's Atlas 14 program is that it has internal routines that provide interstation interpolation that accounts not only for distance from weather stations, but for other factors such as elevation. The user simply locates the area of interest by its coordinates (latitude and longitude), or by using the available interactive map, and Atlas 14 performs the necessary interpolation and generates the corresponding IDF plots and tables; no additional effort is required of the user. Rainfall intensity can also be obtained using rainfall depth-duration-frequency data available on the California Department of Water Resources "Climate Data" webpage at: http://www.water.ca.gov/floodmgmt/hafoo/csc/climate_data/#.

Determining the appropriate runoff coefficient (C) for the crossing site is critical when using the Rational Method. For 100-year flood flows in California's North Coast region, Buxton et al. (1996) suggest that the runoff coefficient should be 0.40.⁹ Experience in the Redwood Creek watershed has led to the use of runoff coefficients ranging from 0.35 to 0.45 for 100-year flood flows, depending on terrain type (G. Bundros, RNSP (retired), unpublished data). Dunne and Leopold (1978) state that C values for small forested mountainous watersheds with sandy-loam soils can be 0.40 or higher for long duration storms with a recurrence interval of 100 years. Table 1 provides a general guide for Rational Method runoff coefficients that has often been cited. **In general, we recommend a C factor**

⁹ This report provides the results of field tests on the Rational Method and other techniques made in southern Humboldt County during a large runoff event.

ranging from 0.25 to 0.45, depending on the specific location of the crossing.¹⁰

Utilizing a relatively large runoff coefficient is conservative in that it reduces the risk of failure from flood flow. We suggest a conservative estimate to reliably avoid under design at sites where runoff data are not available. This is particularly important in watersheds with disturbed hillslopes and/or denuded vegetation. Estimates can be refined using local-specific information if it exists for large storms at the appropriate watershed size.

Assumptions with the Rational Method include (1) the design storm covers the entire basin with constant rainfall intensity until design discharge at the crossing site is reached (time of concentration), (2) Horton overland flow occurs, (3) the runoff coefficient is uniform across the watershed, and (4) the 100-year rainfall event produces the 100-year flood flow. In actuality, there are problems with each of these assumptions, particularly the overland flow assumption, as discussed above. These issues are minimized, however, when the basin size above the crossing site is small. Chow (1964) recommends that the Rational Method be limited to watersheds less than 100 acres and never used for basins larger than 200 acres. Dunne and Leopold (1978) reported that this method should only be used for catchments of less than 200 acres, but state that it has frequently been used for basins up to 640 acres (one square mile). **Similar to Chow (1964), we recommend that the Rational Method be generally limited to watersheds less than 100 acres, and never used for basins greater than 200 acres.**

This method is easy to use, generally understood, and can account for local conditions, or change in conditions such as land conversion, timber harvest, and fire effects. Disadvantages include the assumptions listed above that are usually not met, difficulty in determining an appropriate time of concentration, and a lack of documented field validation to determine appropriate runoff coefficients for different parts of the state. Detailed examples for use of the Rational Method (and other methods) are provided in CDF (1983), Wopat (2003), Weaver et al. (2015), and Appendix A of this document. Cafferata and Reid (2013) illustrate the use of local data to calibrate the method for application in a particular area.

¹⁰ Rossmiller (1980) lists the variables that have been used by previous investigators to estimate the runoff coefficient (C). Table 1 only takes into account one factor—soil type. Caltrans' (2015) Highway Design Manual provides a table for estimating C values that takes into account four variables: (1) differing topographic relief, (2) infiltration rates based on soil type, (3) proportion and kind of vegetal cover, and (4) degree of surface storage. Several authors have suggested that C factors should recognize that longer recurrence interval (RI) storm events (e.g., 100-year RI) tend to have a higher proportion of runoff than shorter RI storms. **Caltrans considers the C values obtained from the Caltrans table to be applicable for storms up to 5 to 10 years and suggests that such C values be multiplied by 1.25 to obtain an appropriate C value for 100-year RI storms.** Where seasonal snowpack may be present during periods of rain, the runoff coefficient may be increased by 0.10 to account for rain-induced snowmelt (BC Government 1991). Cutter and McCuen (2007) developed a model that shows the relationship between the runoff coefficient and watershed slope for steeply sloped watersheds.

Table 1. Rational Method runoff coefficients (Dunne and Leopold 1978).

Woodland Soils in Rural Areas	Runoff Coefficient (C)
Sandy and gravelly soils	0.10
Loams and similar soils without impeding horizons	0.30
Heavy clay soils or those with a shallow impeding horizon; shallow soils over bedrock	0.40

II. 2 Updated USGS Magnitude and Frequency Method

The updated USGS Magnitude and Frequency Method is based on a set of empirical equations derived from precipitation and runoff data collected through water year 2006 at stream gaging stations located throughout California (Gotvald et al. 2012). These regional regression equations replace those generated by Waananen and Crippen (1977) that have been widely used in the past.¹¹ Gotvald et al. (2012) report that Waananen and Crippen’s (1977) equations were based on data only through 1975, and thus may be unreliable given the 30 years of additional data currently available. In addition, they state that improved regionalization techniques have been developed since 1977. For example, Parrett et al. (2011) developed a method for estimating regional skew, an important component in the statistical analysis of gaging station data.¹²

Gotvald et al. (2012) analyzed streamflow records from 771 stream gaging stations; data from 630 stations were used to derive updated equations which were developed for 2, 5, 10, 25, 50, 100, 200, and 500-year flow recurrence intervals. Note that there are substantial differences in some of the regional boundaries used by Gotvald et al. (2012) when compared to those in Waananen and Crippen (1977), especially for the Sierra Nevada and Lahontan Regions.

The equations for 100-year flood flows for the newly defined six regions of California are as follows (see Figure 2 for the regional boundaries):¹³

¹¹ Mann et al. (2004) reported that the Waananen and Crippen (1977) 100-year regression equation produced generally accurate predictions of peak flows in the North Coast Region of California.

¹² Skew can be defined as the shape of the annual peak discharge distribution, which is often significantly affected by the presence of very small or very large discharges in the flow record (i.e., outliers).

¹³ A web tool for using these updated equations is available at:

<http://onlinecalc.sdsu.edu/onlineusgsfloodscalifornia.php>

North Coast	$Q_{100} = 48.5 A^{0.866} P^{0.556}$
Sierra Nevada	$Q_{100} = 20.6 A^{0.874} P^{1.24} H^{-0.250}$
Lahontan	$Q_{100} = 0.713 A^{0.0.731} P^{1.56}$
Central Coast	$Q_{100} = 11.0 A^{0.840} P^{0.994}$
South Coast	$Q_{100} = 3.28 A^{0.891} P^{1.59}$
Desert	$Q_{100} = 1350 A^{0.506}$

where: Q_{100} = predicted 100-year flow in cfs
A = drainage area above the crossing in square miles
P = mean annual precipitation in inches
H = mean basin elevation in feet

Watershed drainage area may be estimated by using a dot grid overlay, planimeter, topographic map program (e.g., Maptech® Terrain Navigator), Google Earth Pro software program, or with a Geographic Information System (GIS). Mean basin elevation can be determined with GIS (e.g., 30-meter DEM) or manually with USGS topographic maps.

Mean annual precipitation data are available from isohyetal maps (e.g., Rantz 1972) and several internet sites. For example, tabular precipitation data may be obtained for numerous California stations from the Western Regional Climate Center at: <http://www.wrcc.dri.edu/summary/climsmnca.html>. To obtain an estimate of annual precipitation for any location in California, we recommend using the Oregon State University PRISM website, found at: <http://www.prism.oregonstate.edu/explorer/>.¹⁴

This method is easy to use, rainfall data are readily available, and flow estimates are based on measured discharge data from numerous, widely distributed locations, including rain-on-snow flow events. For these reasons, the updated USGS Magnitude and Frequency Method is generally preferred over the Rational Method for drainage areas larger than 25-50 acres in California, except for the Central Coast Region, where it is preferred for drainage areas greater than 70 acres, and the Lahontan Region, where it is preferred for drainage areas greater than 100 acres.¹⁵ The primary disadvantage is that it generalizes vast regions of the state, resulting in overestimation in some areas and underestimation in other areas. It is unvalidated for use with very small watersheds, because very small basins are outside of the range of the drainage areas used to generate the regression equations. Table 2 summarizes minimum drainage areas and other information used to generate the updated 100-year flood flow regression equations.

¹⁴ The OSU PRISM site provides an estimate of mean annual precipitation over a 30 year period (1981-2010) for any site in California using a Google map interface.

¹⁵ Note that the minimum drainage area used for the Lahontan Region was 288 acres, but it is expected that the regression equation for this region will generate superior estimates of the 100-year flood flow for watersheds greater than 100 acres compared to the Rational Method.

The updated USGS Magnitude and Frequency Method regression equations are used for the National Streamflow Statistics (NSS) program in California. NSS is a widely utilized and accepted Windows-based software program, developed by the U.S. Geological Survey, that is used to estimate approximate peak discharges for ungaged basins throughout the United States (Ries 2007; see the following website for more information on NSS and the associated software available online: <http://water.usgs.gov/osw/programs/nss/index.html>). Use of the program allows 100-year flood flow estimates to be generated along with standard error estimates. The standard errors of estimate for all the hydrologic regions except the Desert Region are lower for the updated regression equations than for the equations developed by Waananen and Crippen (1977).

A second program of interest is the USGS program “StreamStats” available at: <https://water.usgs.gov/osw/streamstats/>. StreamStats is a web-based GIS application that will delineate drainage basin boundaries and provide streamflow statistics for user-selected ungaged crossing sites.

Table 2. Updated USGS Magnitude and Frequency Method 100-year regression equation information (Gotvald et al. 2012).

Region	Minimum Drainage Area (ac)	Maximum Drainage Area (ac)	# of Stations used in the Analysis	Std Error of Estimate (log ₁₀ units) ¹⁶
North Coast	26	2,048,000	207	0.18
Sierra Nevada	45	1,280,000	231	0.23
Lahontan	288	960,000	63	0.27
Central Coast	70	2,944,000	114	0.24
South Coast	26	544,000	121	0.17
Desert	26	110,720	33	NA

¹⁶ To use the standard error estimate (SEE), obtain the Q₁₀₀ discharge estimate, convert it to log₁₀ units, add and subtract the SEE to get the value of 1-SEE above and below the predicted Q, then obtain the antilogs of the 1-SEE limits to find the 1-SEE range of the estimate.



Figure 2. Regions used to generate the updated USGS Magnitude and Frequency Method regression equations (from Gotvald et al. 2012). Blue = North Coast Region; violet = Lahontan Region, green = Sierra Nevada Region, buff = Central Coast Region, orange = South Coast Region, and purple = Desert Region. The map in full resolution may be downloaded at: <http://pubs.usgs.gov/sir/2012/5113/>

II. 3 Flow Transference Methods

If a stream gaging station is located on the same stream as the proposed crossing site or is on a nearby stream that is hydrologically similar, it is possible to adjust the 100-year discharge estimate to account for the difference in drainage area between the ungaged basin and the gaged basin by using the following flow transference equation (Waananen and Crippen 1977):

$$Q_{100u} = Q_{100g} (A_u/A_g)^b$$

where:

- Q_{100u} = 100-year flow at the ungaged site in cfs
- Q_{100g} = 100-year flow at the gaged site in cfs
- A_u = drainage area of ungaged site in mi²
- A_g = drainage area of gaged site in mi²
- b = exponent for drainage area from the appropriate Waananen and Crippen 1977 USGS Magnitude and Frequency equation (e.g., 0.77 for the 100-year equation for the Sierra Region, 0.87 for the North Coast Region, 0.59 for the Northeast Region, and 0.88 for the Central Coast Region)

The downstream or nearby gaging station used with the flow transference method should have a long-term station record (suggested to be more than 20 years). Additionally, the 100-year flood flow estimate for the gaged station must be known. This can be determined relatively easily for USGS gaging stations through the use of PeakFQ, a USGS Windows 7/8 software program that performs a flood-frequency analysis based on Bulletin 17B (log-Pearson Type III distribution), which is the accepted methodology published by the Interagency Advisory Committee on Water Data (IACWD 1982).¹⁷ It is available from the USGS website at: <http://water.usgs.gov/software/PeakFQ/>. Alternatively, Q_{100} can be determined using the US Army Corps of Engineers (USACE) HEC-SSP computer program, available at: <http://www.hec.usace.army.mil/software/hec-ssp/>,¹⁸ or through a manual flood frequency analysis (i.e., plotting discharges and recurrence intervals; see Dunne and Leopold (1978).

Waananen and Crippen (1977) state that the flow transference method is superior to the more general USGS Magnitude and Frequency Method regional regression equations if the stream gaging station is nearby and the available stream gaging annual peak discharge records are adequate. **Under these conditions, the flow transference method is preferable to the updated USGS regional regression equations because local data are likely to better represent the drainage-basin characteristics in terms of slope, geology,**

¹⁷ The USGS is expected to release Bulletin 17C in the second half of 2017, with improvements for addressing historical flood data and low flow outliers. A flood frequency analysis example using instantaneous peak flow data (log-Pearson Type III distribution) is provided at: <http://streamflow.engr.oregonstate.edu/analysis/floodfreq/example.htm>

¹⁸ The HEC-SSP and the PeakFQ programs generate slightly different estimates of 100-year flood flows, since PeakFQ uses a weighted skew value, while HEC-SSP uses station skew.

soils, and climate, when compared to the more general regional equations. The highest level of confidence in this method occurs when the drainage area of the ungaged site is between 50 and 150 percent of the drainage area of the gaged site (Sumioka et al. 1998).

An alternate approach to the Waananen and Crippen (1977) flow transference approach can be used if the gaged watershed is relatively small (e.g., <2,500 acres), the gaged and ungaged basins are in close proximity, are hydrologically similar, **and are approximately the same size (within one order of magnitude)**. Skaugset and Pyles (1991) term this approach “**direct flow transference**” and state that the simplest method of direct transfer is by adjusting streamflow records by the ratio of the watershed areas:

$$Q_{100u} = Q_{100g} (A_u/A_g)$$

A considerably more detailed flow transference method is provided in Gotvald et al. (2012). They state that flow estimates at ungaged sites on the same stream as the gaged sites can be improved by weighting the estimates obtained from the updated regression equations with estimates that are determined on the basis of flow at an upstream or downstream stream gage. See Gotvald et al. (2012) for the specific methodology.

II. 4 Computer Programs

Several computer programs are available to estimate 100-year flood flows for watercourse crossing sites in more complicated situations. The USACE HEC-HMS and NRCS TR-55 programs utilize the unit hydrograph approach and can be used where streamflow is regulated by upstream ponds or reservoirs (Thomas et al. 2001). While not commonly used for flood flow estimates at typical forest road crossings, these programs are often used when assessing flood flows associated with vineyard timberland conversion projects.

NRCS (1986) provides detailed information on the TR-55 program, which is based on the SCS curve number methodology. While the unit hydrograph analysis using SCS curve numbers results in reasonable estimates for predicting a **percent change** (i.e., relative change) in flood flows due to land use modification, the calculated peak discharges may overestimate absolute values of stream discharge in forested watersheds. Methods based on SCS curve numbers have not proven to accurately project peak discharge rates in forest environments in past analyses (Skaugset and Pyles 1991, Fedora 1987, G. Ice, NCASI (retired), personal communication).

The HEC-HMS unit hydrograph program (USACE 2013) simulates precipitation-runoff and routing processes, both natural and controlled, and replaced HEC-1 (<http://www.hec.usace.army.mil/software/hec-hms/features.aspx>). The program is designed for surface water hydrology simulation. It includes components for

most facets of the hydrologic cycle, including precipitation, evaporation, snowmelt, infiltration, surface runoff, saturation overland flow, and stream flow. Flow hydrographs can be developed for multiple sub-basins within a watershed, and these can then be routed through the system to predict 100-year flood flows at multiple potential crossing locations within the larger basin. Detailed information and examples for using both the HEC-HMS and TR-55 programs are provided in Kinoshita et al. (2013).

Thomas et al. (2001) reported that there has not been conclusive evidence of greater accuracy using these types of simulation models for extreme floods. Additionally, they state these programs are not commonly used because of their significant data requirements and the time and effort involved in their calibration. However, they note that recent improvements in development of databases and GIS technology have allowed users to apply these models with less effort than in the past. Their analyses of two regions in the U.S. (including Lake County in California) indicated that regional regression equations provided more accurate and reproducible estimates of flood discharges than rainfall-runoff models such as HEC-1 and TR-20 (note that TR-55 is a simplified version of TR-20). **Therefore, unless special situations, such as the need to route flow through a watershed, require the use of these programs, the other methods described in this section are likely to be sufficient for watercourse crossing design work.**

In addition to programs that use the unit hydrograph approach, there are software packages that use the slope-area method for calculating peak discharges. For example, WinXSPRO is a Windows™ software package designed to analyze stream channel cross-section data for geometric, hydraulic, and sediment transport parameters for high-gradient streams (>1%). WinXSPRO utilizes the slope-area method of estimating flood flows using Manning's Equation. This method is a direct approach to calculating the volume of flow that has occurred in the channel where the crossing will be installed and it utilizes physical laws of fluid movement in an open channel.¹⁹ WinXSPRO can be used for estimating river stage and discharge at an individual cross-section (Hardy et al. 2005). Additionally, water surface profiles and average channel velocities for the design flow can be determined with this method. WinXSPRO is available online at no cost; see the following website:
<https://www.fs.fed.us/biology/nsaec/products-tools.html>.

The slope-area method can provide a good field check on maximum flows if high water marks can be determined and they can be tied to an event of some known recurrence interval. It is useful in regions where good hydrologic data do not exist.

¹⁹ A description of the slope-area method is provided in CDF (1983) and under the "Permanent Bridge Design" in Chapter X of this document.

III. Field Techniques for Evaluating Proposed Culvert Diameters

III.1 Culvert Sizing Nomograph

Following the office calculation of 100-year flood flows for crossings utilizing a culvert, it is necessary to determine the required pipe diameter required to convey the flow. This evaluation must take culvert hydraulics into account and is often made through the use of a culvert sizing nomograph (see Figure A-5 or Normann et al. 2005 and Schall et al. 2012).²⁰ For most culverts in upland or mountainous forested watersheds where channel gradients are generally greater than 2%, it is possible to assume that there is inlet control when using these nomographs.²¹ Inlet control means that the culvert has a slope great enough that discharge is only controlled by hydraulic factors at the pipe's inlet (i.e., inlet geometry and headwater depth). The limiting hydraulic condition is at the pipe inlet, not the barrel, therefore inlet conditions control pipe size. Culverts operating under inlet control will always flow partially full.

It is critical to specify an appropriate headwater depth to pipe diameter ratio (HW/D) for culverts when making this calculation. Crossing design guidance documents produced prior to 2000 usually specified a maximum HW/D ratio of 1.0 (CDF 1983; M. Furniss, USFS-PNW (retired), personal communication).²² A HW/D ratio of 0.67 (Figure 3) allows additional open head space (unsubmerged area) in the culvert to accommodate sediment and debris passage and is one approach available to lower the potential for plugging (additional methods for improving sediment and debris passage are provided in Chapter IV). The proposed pipe diameter can then be field checked using methods based on either (1) channel cross-sectional area at bankfull stage, (2) the annual high flow line, or (3) the width of the active stream channel in the vicinity of the crossing. Using the size of existing upstream or downstream crossing structures to justify culvert size should not be done without additional hydrologic analysis confirming that the upstream or downstream crossing was adequately designed to accommodate the 100-year flood flow and associated sediment and debris.

III. 2 3 X Bankfull Area Method

The **3 X bankfull area method** (BC MOF 1995, 2002) is a potential field check that appears to be valid for the coastal portions of northwestern California, but very likely underestimates Q_{100} crossing sizes for inland areas away from the

²⁰ Normann et al. (2005) and Schall et al. (2012) provide nomographs for determining flow capacity for both round pipe culverts and other types of stream crossing structures (e.g., pipe-arch culverts).

²¹ A culvert that has a slope greater than 1.5% to 2% will normally exhibit inlet control (Beschta 1984, Piehl et al. 1988).

²² Beckstead et al. (2000) and Keller and Ketcheson (2015) suggest a HW/D of approximately 0.8 in areas where considerable floatable debris would not be expected during flood events. WADNR (2006, 2013) states that in areas without mobile wood, a HW/D up to 0.9 may be acceptable. Based on these references, a HW/D up to 0.8 to 0.9 may be acceptable in areas without substantial mobile wood or high volumes of sediment. Also, note that the pipe inlet type (i.e., projecting, mitered, headwall) must be considered when using the nomograph.

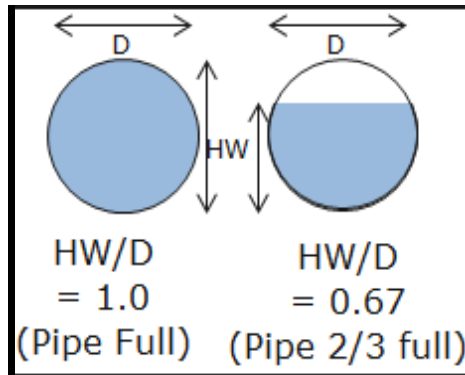


Figure 3. Diagram illustrating headwater to pipe diameter ratios of 1.0 (left) and 0.67 (right). Figure created by Stacy Stanish, CAL FIRE.

rain-dominated portion of the Coast Ranges. This procedure assumes that (1) the bankfull stage and corresponding wetted cross-sectional area of any stream represents the mean annual flood cross-sectional flow area for the stream (Q_2);²³ (2) that the ratio of Q_{100} culvert cross-sectional flow area to Q_2 is 3.0 or less; and (3) that the discharge cross-sectional flow areas are not sensitive to influences from pipe slope and roughness or other factors (since pipes are almost always inlet controlled). These assumptions are not truly representative of all situations, but within the accuracy expected for establishing design discharge, this method was considered to be acceptable for verifying proposed stream-culvert diameters smaller than 78 inches on forest roads in rain-dominated portions of British Columbia (BC MOF 1995, 2002), and the similarity between conditions at those sites and those counties along the coast of northern California suggest that it should also be useful at these sites.

To utilize the 3 X bankfull area technique, first identify a representative stream reach that is within alluvium and is free of disturbances due to natural or anthropogenic causes, such as log jams, tractor logging impacts, and roads. Next, take measurements at a minimum of three cross-sections along the stream reach. Ideally the measurements should be made near the location of the proposed crossing, approximately 100 feet upstream from the crossing, and approximately 100 feet downstream from the crossing (WDNR 2005). Specifically, measure the width of the stream at the top of the bank (W_1 = bankfull width) and at the stream bottom (W_2 = active channel width) in feet (see Figure 4). Measure the depth of the stream at several spots across the opening to obtain the average depth (D) in feet. In unconfined stream channels, bankfull stage, or depth (D), is associated with the flow that just fills the channel to the top of its banks and where water begins to overflow onto a floodplain (Rosgen 1996). The individual measurements at each of the three locations should be averaged to derive the final W_1 , W_2 , and D values.

²³ Q_2 is actually the median annual flood; mean annual flood is more often approximately a $Q_{2.5}$ recurrence interval event (R. Beschta, Professor Emeritus, Oregon State Univ., Corvallis, personal communication). Bankfull stage or flow is typically assumed to be a $Q_{1.5}$ to Q_2 event.

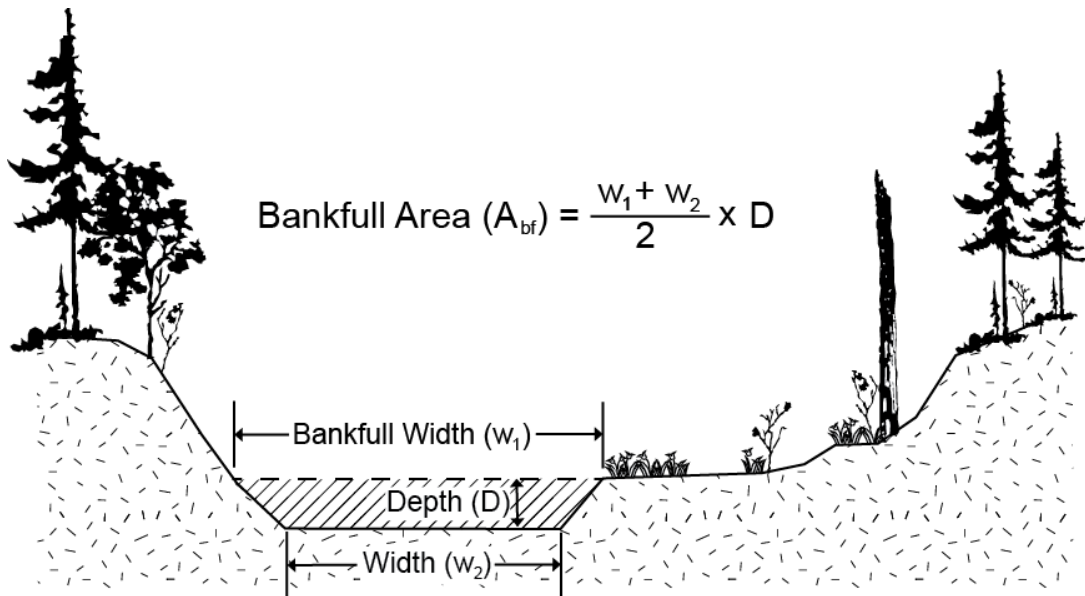


Figure 4. Diagram illustrating how to determine bankfull channel area. Figure created by Jacob Lee, CGS.

Calculate the bankfull cross-sectional area of the stream, $A_{bf} = (W_1 + W_2)/2 \times D$. Calculate the area of the required culvert opening (A_c) as follows:

$$A_c = 3 A_{bf}$$

Using an alternative notation where $A_c = \pi r^2$ (r = radius of the culvert opening), the diameter ($d = 2r$) of the culvert opening can easily be calculated as follows:

$$\begin{aligned} \pi r^2 &= 3 A_{bf} \\ r^2 &\approx A_{bf} \text{ (note that this is approximate)} \\ r &\approx (A_{bf})^{1/2} \\ \mathbf{d} &\approx \mathbf{2[(A_{bf})^{1/2}] \text{ or } 2\sqrt{A_{bf}}} \end{aligned}$$

Therefore, the culvert diameter can be approximated by the equation: $d \approx 2[(A_{bf})^{1/2}]$. For example, a stream with a bankfull cross-sectional area of three (3) square feet would need a culvert diameter of approximately 3.5 feet (i.e., 42 inches):

$$\begin{aligned} d &= 2[(3 \text{ ft}^2)^{1/2}] \\ d &= 2(1.73 \text{ ft}) \\ d &= 3.5 \text{ ft} = 42 \text{ inches} \end{aligned}$$

Table 3 provides bankfull cross-sectional areas and corresponding culvert diameters for situations commonly encountered in forested watersheds.

Table 3. Bankfull cross-sectional area vs. culvert diameter.

Bankfull Cross-Sectional Area (ft ²)	3 X Bankfull Cross-Sectional Area (ft ²)	Culvert Diameter (inches)	Culvert Area (ft ²)
0.5	1.5	18	1.8
1.0	3.0	24	3.1
1.5	4.5	30	4.9
2.0	6.0	36	7.1
3.0	9.0	42	9.6
4.0	12.0	48	12.6
5.0	15.0	54	15.9
6.0	18.0	60	19.6
8.0	24.0	66	23.8
9.0	27.0	72	28.3
10.0	30.0	78	33.2

Any evidence from major storms also needs to be taken into account when this method is applied. If there is a debris line along the stream channel that indicates the flood flow had a cross-sectional area greater than 3 times A_{bf} , then the culvert diameter should be increased to match or exceed the flood cross-sectional area.²⁴ In addition to the need to accommodate large storm streamflow, wood and sediment passage must also be considered (see the discussion on sediment and debris passage in Chapter IV). **The 3 X bankfull area method works best for pipe sizes up to 48 inches (G. Bundros, RNSP (retired), unpublished information), and it is not applicable to culverts greater than 78 inches in diameter (BC MOF 1995, 2002).**

The 3 X bankfull area method uses on-site field conditions, is easy to use, approximates the culvert diameter directly, and offers an easy field check of office calculations for rain-dominated northwestern California watersheds. There are, however, several limitations to the use of this method. **The most significant limitation is that it requires a clear indicator of bankfull stage, which can be very difficult to discern for small watersheds.²⁵ Also, the assumptions regarding bankfull channel geometry and flow recurrence interval may not apply for channels with more resistant boundaries, channels subject to frequent mass wasting events (Wohl 2004), or those subjected to episodic high-magnitude flows (such as rain-on-snow events).**

²⁴ Major storm events have a recurrence interval of greater than 20 years. If an area of a watershed had just experienced a major storm, this would likely cause an increase in culvert size relative to what the design would have been without the major storm.

²⁵ The term "bankfull stage" is difficult to apply to small, entrenched stream channels. Bankfull stage can be determined by stage indicators situated along the boundary of the bankfull channel (Rosgen 1996). Bankfull discharge is associated with a flow which, on the average, has a recurrence interval of 1.5 years (Dunne and Leopold 1978). Field personal must be trained in identifying bankfull stage.

For intermittent or ephemeral watersheds where it is hard to determine bankfull stage and/or where longer-recurrence interval flooding has obscured bankfull indicators, it can be acceptable to approximate bankfull stage with the annual high flow line (M. Furniss, USFS-PNW (retired), personal communication). Another approach for these types of small channels is to simply make the culvert diameter at least equal to the active channel width (W_2) at the crossing location.²⁶

When using the 3 X bankfull area method, cross-sections measured should be representative of the channel in the general crossing area and not be affected by roads. Field review should include an evaluation of bankfull indicators upstream of and away from the influence of existing or previous crossings, especially if they are or were undersized and aggraded material is present in the channel. The identification of bankfull stage in severely impacted channels is difficult, especially when accumulations of large wood and sediment are present in the channel. **Another limitation is that while some field verification of this method has occurred in northwestern California²⁷ (Figure 5), virtually none has taken place in interior areas of California. Evidence suggests that use of the factor of 3 would overestimate appropriate culvert diameters in snow-melt dominated inland areas and underestimate diameters in areas susceptible to rain-on-snow flooding.**

To illustrate this point, Beckers et al. (2002) reviewed the 3 X bankfull area method proposed by the BC MOF (1995, 2002) and found that the ratios of 100-year flood flow to 2-year discharge (Q_{100}/Q_2) vary substantially with basin area and climate.²⁸ For flood peaks generated by rainfall and rain-on-snow events in coastal British Columbia, the range was 3.1 to 2.6, but for snowmelt-dominated peak flows in the Canadian Rocky Mountains, the Q_{100}/Q_2 ratio decreases with increasing drainage area from 2.3 to 1.9. Similarly, Pitlick (1994) reported that for regions where flooding is caused by large-scale frontal storms in the western U.S., 100-year floods may be 3 to 6 times the mean annual flood, but in regions dominated by snowmelt the Q_{100} is less than two times the mean annual flood.

²⁶ Because the diameters of culverts sized to handle northwestern California 100-year flood flows alone (not considering flood-associated sediment and floating debris) average approximately two-thirds the width of the active channel (W_2), a culvert sized large enough that its diameter **equals** the active channel width (W_2) should accommodate the expected 100-year flood flow and have enough additional headroom to accommodate flood-associated sediment and debris as well.

²⁷ The method was only field tested in coastal regions underlain by schist and mélange units of the Franciscan Complex, Central Belt terrain. The method may be more difficult to apply in more resistant geologic units where bankfull stage may be hard to discern. More testing of this approach is needed.

²⁸ There is abundant data for the two-year recurrence interval discharge (Q_2) at gaging stations, and it is the recurrence interval most similar to the 1.5 year flow commonly associated with bankfull flows.

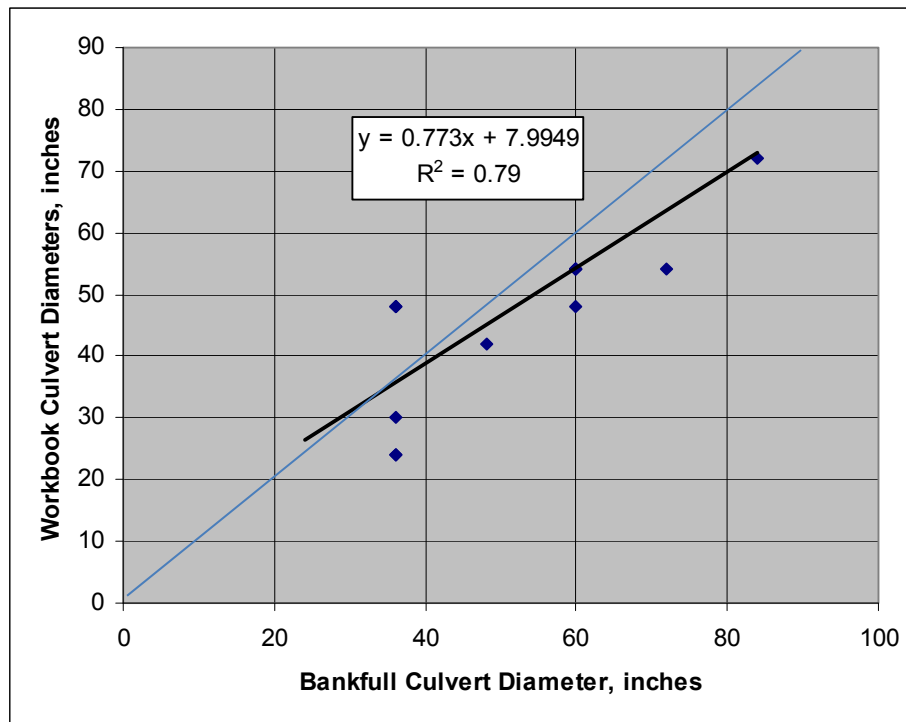


Figure 5. Plot of 3 X bankfull area determined culvert diameters for drainage areas less than 200 acres (x axis) vs. culvert diameters determined by a workbook spreadsheet (y axis) using either the Rational Method (for drainage areas less than 80 acres and a runoff coefficient of 0.40) or the 1977 USGS Magnitude and Frequency Method equation (for drainage areas greater than 80 acres and less than 200 acres) for the Redwood Creek watershed in northwestern California. Pipe diameters were determined from 100-year recurrence interval flood flows estimated using a culvert sizing nomograph (for example, see Figure A-5), and assuming a projecting pipe entrance and HW/D = 1.0 (unpublished data collected by Greg Bundros, RNSP (retired)). The blue 1:1 fit line illustrates that on average the 3 X bankfull area field method slightly over predicts pipe diameter in this northwestern California watershed.

Rain-on-snow events greatly elevate discharge above snowmelt alone and have resulted in some of the largest floods on record in California. For example, Kattelmann (1990) found that during a 60-year period in the Sierra Nevada, six large floods with recurrence intervals of only 10 to 20 years produced discharges that were 4 to 10 times the magnitudes of the mean annual flood. Rain-on-snow was an important mechanism in all but one of these events.

A brief review of Q_{100}/Q_2 ratios for California, using flood-flow values from 12 stations along an east-west transect approximately parallel to latitude 40°N (data from Waananen and Crippen 1977), shows average Q_{100}/Q_2 flood-flow ratios to increase eastward from the coast. Average Q_{100}/Q_2 flood-flow ratios increased eastward from the North Coast flood-frequency region (FFR) (avg. $Q_{100}/Q_2 = 3.7$, $n = 6$), through the Sierra FFR (avg. $Q_{100}/Q_2 = 5.4$, $n = 6$).²⁹ The increase in the

²⁹ Although the average Q_{100}/Q_2 ratio for the North Coast FFR stations (= 3.65) exceeds 3.0, such flood flows can be handled by culverts with cross-sectional areas only 3 times bigger than the Q_2 bankfull watercourse cross-sectional area for two reasons: (1) the roughness of natural

Q_{100}/Q_2 ratio with distance inland from the northern California coast suggests that 100-year flood flows increase relative to 2-year bankfull flows with distance from the coast. Consequently, using the 3 X bankfull area method to size culverts inland from the coast should result in increasingly undersized culverts as distance from the coast (and the Q_{100}/Q_2 ratio) increases. Because of the change in the Q_{100}/Q_2 ratio with distance from the coast, **we recommend that the 3 X bankfull area method be used inland from the coast only as a field check of minimum culvert diameter.** In other words, the diameter of a culvert designed to handle the 100-year flood flow and associated sediment and debris inland from the coast should be no smaller than the diameter obtained using the 3 X bankfull area stage method and will likely need to be larger. Another field approach for small incised channels located in interior California is to install culverts that have a diameter at least equal to the active channel width at the crossing location.

III. 3 Spring-Dominated Watercourses

For spring-dominated watercourses, flood flows may be poorly correlated with drainage area (Figure 6). The lack of correlation between drainage area and flow may violate the assumptions associated with the Rational Method and flow transference methods, which both assume that flow is proportional to drainage area. Additionally, drainage area is the most significant variable in the USGS Magnitude and Frequency Method, and Figure 6 indicates a poor relationship between drainage area and flow for spring-dominated watercourses draining young volcanic rocks. For these types of watercourses, it is generally more appropriate to use field-based rather than office-based methods to determine appropriate crossing size.

Recognition of spring-dominated watercourses can allow Registered Professional Foresters (RPFs)³⁰ and other resource professionals to adjust office-based calculations of flow in response to observed field indicators. Some indicators of spring-fed streams include (1) sustained year-round flow, (2) stable wood accumulations, (3) lack of developed floodplains, and (4) poorly-organized channel bedforms (Grant et al. 2012).

Due to their unique geologic and hydrologic characteristics (as described in Whiting and Moog 2001), Quaternary-age volcanic deposits are frequently associated with high volume spring-dominated watercourses and are common in the northeastern part of California. Springs may also be concentrated in Tertiary-aged volcanic rocks (Jefferson et al. 2010), but they are not as common, nor

streambeds is greater than that of culverts, resulting in slower flow velocities and, for a given discharge, larger cross-sectional areas in a natural stream bed relative to a culvert of similar capacity, and (2) transport efficiency (Q/ft^2) increases with culvert size. For example, increasing culvert cross-sectional area 3 times increases flow capacity approximately 3.9 times.

³⁰ In California, a person known as a RPF must hold a valid license to practice as a professional forester pursuant to Article 3, Chapter 2, Division 1 of the California Public Resources Code (PRC).

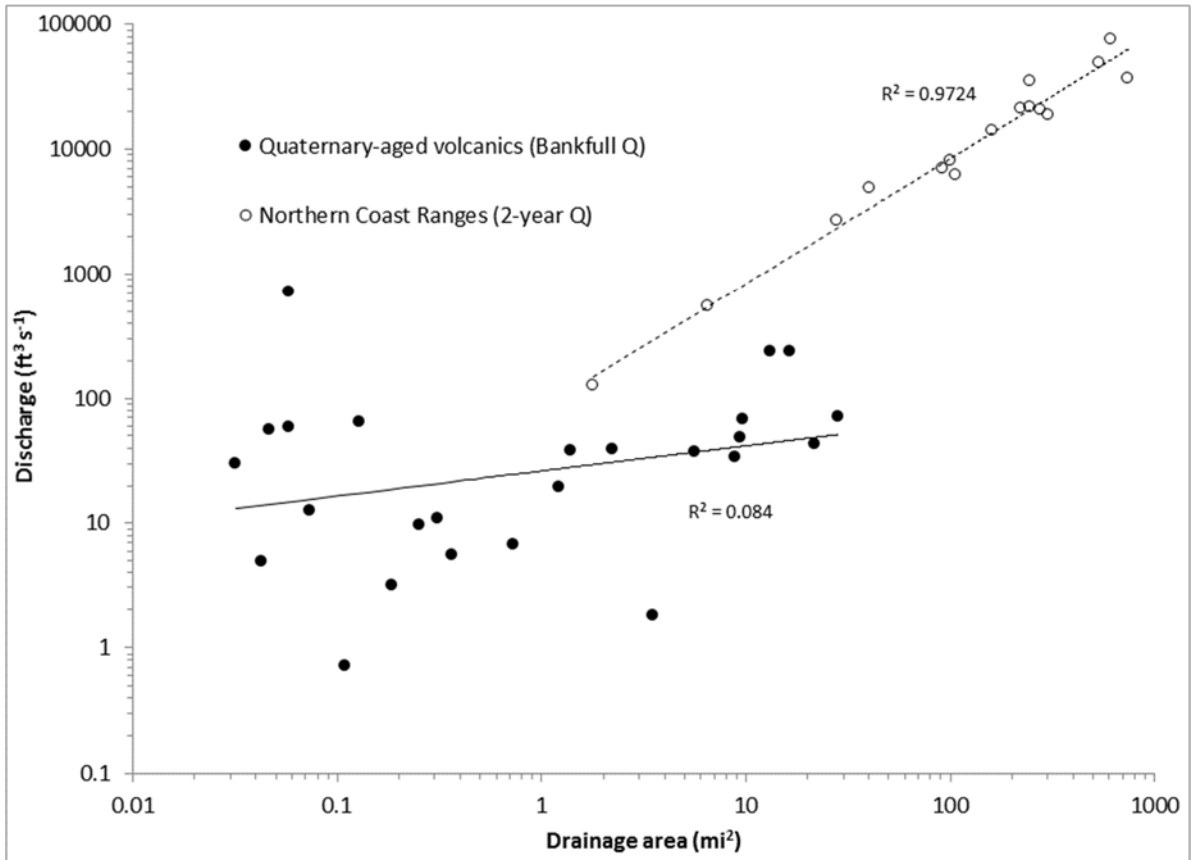


Figure 6. The relationship between discharge and drainage area for spring-dominated watercourses versus stormflow-dominated watercourses. Data for spring-dominated watercourses in watersheds draining Quaternary-aged volcanics were taken from Whiting and Moog (2009). Data for stormflow-dominated watercourses in the northern Coast Ranges were obtained from the USGS. The figure displays the bankfull discharge for the watersheds draining Quaternary-aged volcanics and the 2-year recurrence interval flows for watersheds in the northern Coast Ranges.

typically as large, as those in the Quaternary volcanic rocks (Figure 7). Other areas of the state may also have spring-dominated watercourses, and they may be associated with transitions and/or discontinuities in surface/subsurface geology (e.g., Quaternary deposits, geologic contacts and/or faulting) (Costigan et al. 2016).



Figure 7. A map of Quaternary and Tertiary-age volcanic rocks in California. These areas are more likely to contain spring-dominated watercourses.

IV. Wood and Sediment Passage at Culvert Crossings

While determination of culvert diameter based on streamflow is often the easiest aspect of crossing design, it is not the only design issue to be considered. Consideration for wood and sediment passage is often of equal or greater concern than hydraulic capacity for preventing culvert failure (Flanagan 2004, see Figure 1). Furniss et al. (1998) provide advice on crossing design to accommodate wood and sediment passage. Unfortunately, no rigorous design techniques are available to size pipes for wood and sediment passage, and prediction of the loading of sediment and wood at a given crossing remains difficult. There are, however, strategies available to determine if wood is likely to be hazardous to infrastructure (e.g., Wohl et al. 2016). The goal is to design crossings to better accommodate wood and sediment, thereby reducing the risk of crossing failure.

How wood and sediment are conveyed through a culvert is mostly controlled at the pipe inlet, which in turn determines plugging potential and the actual culvert capacity. Research using flumes and larger rivers suggests that wood will generally not deposit in straight, narrow reaches (Braudrick and Grant 2001). Thus, maintaining channel orientation and dimensions through the crossing will help facilitate debris passage through the culvert (Figures 8 and 9). In other words, it is appropriate to utilize culverts that are as wide, or nearly as wide, as the active channel (i.e., the zone of active, annual streambed scour and deposition), particularly in small streams.



Figure 8. Example of woody debris blocking a culvert inlet following a January 2012 storm event in northern California. The culvert, which is not visible under the debris, is narrower than the channel is wide immediately upstream of the crossing.



Figure 9. Woody debris blocking a culvert inlet in the upper Sacramento River watershed (photo provided by Matt Boone, CAL FIRE).

Furniss et al. (1998) describe several additional techniques for reducing the risk of culvert failure related to wood and sediment passage. **These include (1) specifying a headwater depth to pipe diameter ratio (HW/D) significantly smaller than 1.0, such as 0.50 or 0.67 (i.e., at maximum flow, the pipe would be flowing one-half full to two-thirds full, respectively),³¹ (2) avoiding wide areas above the pipe inlet, (3) installing culverts at the same gradient as the natural stream channel, and (4) avoiding angular deviation by installing culverts so that they are aligned parallel to the natural channel (Figure 10).** Additionally, a single large culvert at a crossing minimizes plugging potential in most channels and is always better for wood passage than several small ones (Keller and Sherar 2003, Furniss et al.1991, Weaver et al. 2015).³²

Passing debris through a culvert is not always possible. Where this is an issue, alternate design strategies should be considered, including converting the culverted crossing into a free-spanning crossing using an arch-culvert or bridge, building an armored crossing, or installing debris control structures, such as

³¹ Note that most guidelines issued before 2000 (e.g., CDF 1983) specified a maximum HW/D ratio of 1.0.

³² Wargo and Weisman (2006) and Keller and Sherar (2003) report that there are benefits to using multiple pipes for channels that are not incised and do not carry large debris loads. They state that multiple pipes provide high depths of flow at low flow conditions within a culvert, which may enhance fish passage (particularly if one of the pipes is set deeper than the others to maintain sufficient flow depth).



Figure 11. Example of a metal flared end section (photo provided by Mike Derrig, USFS).

Installation of metal flared end sections yields large gains in capacity for all watershed products (Figure 11). For example, flared end sections (e.g., side-tapered inlets) decrease turbulence at the inlet creating a smooth transition that can increase culvert flow capacity when the HW/D ratio is 1 by about 20 percent when compared to a pipe with a projecting inlet (Harrison et al. 1972).³³ Additionally, they can prevent the lodging of rocks and woody debris at the inlet lip (M. Furniss, USFS, Pacific Northwest Research Station (retired), Corvallis, OR, personal communication; AISI 1971; Weaver et al. 2015). Including a flared inlet as part of a new culvert installation will also yield these same benefits. Another approach to consider includes using temporary crossings,³⁴ rock fords, rock-armored crossings, oval or pipe-arch culverts, open bottom arches, or bridges in channels with large amounts of mobile wood, instead of a round pipe.

Mitering the culvert inlet is an inexpensive approach commonly used to reduce the potential for blockage by woody material (WTIC 2004, Weaver et al. 2015). A fully mitered culvert is formed when the culvert is cut to conform to the face of the fillslope embankment (Figure 12). Mitering the culvert improves flow efficiency at the inlet and can increase culvert discharge capacity from 5 to 20 percent compared to a projecting inlet culvert, with the higher flow increases occurring at

³³ The hydraulic capacity improvements for inlet controlled pipes is a function of the HW/D ratio and increases as the ratio goes up. For example, to get above approximately a 20% increase in flow capacity would require HW/D ratios greater than one (which are not recommended).

³⁴ Temporary watercourse crossings must be removed and stabilized before the start of the winter period, or as specified in the plan (14 CCR § 923.9 [943.9, 963.9] (r)). The winter period is defined by the California FPRs as beginning on November 15th, except in selected coastal counties, when it is either October 1st or October 15th. Weaver et al. (2015) provide a detailed discussion on temporary crossing alternatives.

high headwater depths (e.g., HW/D ratios greater than one) (Harrison et al. 1972). Although mitering does slightly improve flow capacity with lower recommended HW/D ratios, there are two more important advantages to mitering culvert inlets. The first is that it reduces plugging by providing an enlarged face area that transitions into the culvert barrel. The large face area increases viable pathways for water to enter the culvert barrel when the inlet is partially blocked by debris. The second benefit is that this method allows debris to float or be pushed up above the angled culvert inlet (Weaver et al. 2015). This is unlike what happens with a projecting culvert, where debris is more likely to get lodged against the square inlet face or against the side of the projecting culvert and the fill face. It is critical, however, to have the mitered inlet project no more than six inches from the fill face to receive these benefits. Furthermore, mitering large-diameter culverts can reduce their structural integrity and their ability to resist lateral earth pressures. For this reason, large-diameter culverts typically have concrete-reinforced headwalls or additional bracing along the mitered inlet.



Figure 12. Example of a mitered culvert inlet. This culvert is located in the northern part of the California Coast Ranges.

Beschta (1984), and more recently Keller and Ketcheson (2015), have reported that (1) various types of structures, commonly denoted as trash racks, trash screens, and debris barriers, can be constructed upstream of the inlet to help prevent plugging by wood, but **winter maintenance of these structures is critical for success** (particularly after large storm events and prior to the start of the winter season); and (2) organic debris can create continual maintenance problems when the culvert diameter is too small to freely pass floatable wood. Properly designed and maintained trash racks can be successfully employed to act as a first line of defense upstream of the pipe inlet if large floating debris is

common.³⁵ On fish-bearing streams or where other aquatic organism passage is a concern, trash racks may not be appropriate.

Studies conducted in the Pacific Northwest and northwestern California reveal that the impacts of culvert failures caused by very large, infrequent storms (e.g., greater than 20-year recurrence interval) that initiate landslides and debris flows can be reduced by minimizing the interference that the crossing presents in the path of the mass wasting feature (Furniss et al. 1998). Crossing failures associated with such mass wasting processes, rather than by fluvial processes, are not the result of inadequate culvert sizing. More frequent large storms (e.g., less than 12-year recurrence interval) have been found to often cause failures by fluvial mechanisms—wood transport and fluvial sediment—and failure probability for these events can be reduced through careful culvert sizing and configuration (Flanagan 2004, Flanagan et al. 1998). In areas of elevated debris flow risk, (1) crossing fills should be lowered in order to present minimal interference, and (2) an evaluation is needed to determine if a rock fill, rock ford, or rock-armored crossing would increase the durability of the watercourse crossing during a mass wasting event.

For more frequent smaller magnitude storms, the dominant failure mechanism is wood and sediment accumulation at the culvert inlet, and typically the wood causing these failures is small (i.e., twigs, sticks, and branches), not large logs. Studies have indicated that fluvially transported wood is strongly controlled by the width of the channel (e.g., Flanagan 2004, Braudrick and Grant 2001, Nakamura and Swanson 1994). Pieces of wood initiating culvert plugging are usually not much longer than the culvert diameter and seldom exceed the width of the channel (Figure 13). As stated above, culvert sizing should be driven by consideration of channel dimensions, including active channel width and channel slope. Sizing for a 100-year flood flow alone does not ensure adequate capacity for wood and sediment. When a sample of culverts in northwestern California were sized for the 100-year flood flow, the resulting pipe diameters were, on average, only about two-thirds the active channel width (i.e., culvert diameter/channel width $\approx 2/3$). However, if the culvert is sized to allow for wood passage (i.e., the pipe is approximately equal to the active channel width), it typically ensures adequate hydraulic capacity for 100-year flood flows or greater.

Additionally, for wood passage it is critical to avoid culvert sizing and installation that creates ponded conditions (e.g., settling basins) at the inlet (Figure 10). Consequently, it is important to not widen the channel at the culvert inlet, as this will cause ponding during higher flow, resulting in wood rotating and accumulating at the inlet and not passing through the culvert in an optimal orientation. Deepening the channel above the culvert inlet will cause similar problems and increases the potential for crossing failure.

³⁵ Weaver et al. (2015) suggest that trash rack be placed across the channel slightly upstream of the culvert inlet (4 pipe diameters), with the spacing of the vertical posts approximately equal to the span or diameter of the culvert.

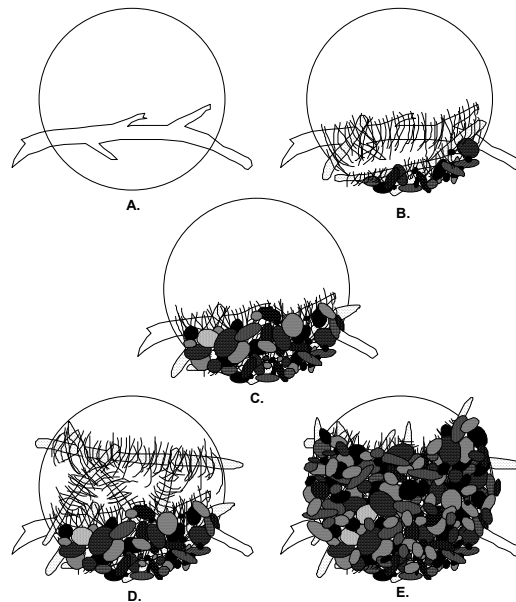


Figure 13. Plugging of culverts by wood is usually initiated by a single piece lodging across the inlet, especially with a projecting pipe inlet configuration (a). This piece becomes a locus for the accumulation of small wood and sediment (b). As the plug grows, sediment and debris seal off a portion of the inlet (c). The initiation process may be repeated with a second piece, allowing the plug to grow upwards (d and e) (Flanagan 2004).

V. Additional Design Considerations for Culvert Crossings

V.1. Vented Crossings

Where a crossing site has perennial or late season flow (e.g., a Class II watercourse as defined by the California Forest Practice Rules) and a high debris load potential that could plug a culvert, or where the calculated pipe diameter is too large to fit in the channel, it is often preferred to install a “vented” crossing.³⁶ A vented crossing is either a ford (Figure B-4), a porous rock-filled crossing (Figure 14), or rock-armored crossing, all of which have a culvert (i.e., “vent”) installed to accommodate low flows during hauling activities. During periods of high flow, it is assumed that the capacity of the culvert will be exceeded and the crossing will be overtopped.

A vented ford implies very little fill is placed in the crossing, thus the approaches need to be lowered and the fill used to bed the pipe(s) needs to be composed of select material free of oversized rock (e.g., >6 inches) and fines to prevent damage to the pipe, provide a suitable travel surface, and limit downstream

³⁶ Vented fords can be constructed to pass large flows and large amounts of debris while still providing fish passage. Detailed information is provided in Clarkin et al. 2006 and Weaver et al. 2015.

impacts in the event it is washed away during a flood event. If the plugging potential at the crossing site is low, then it may be appropriate to install a vented rock-armored crossing. Under this crossing design, the largest pipe that would fit in the channel is installed that would still allow the crossing to be dipped over the pipe, while maintaining the minimal cover per the pipe manufacturer's requirements. In some situations, a pipe-arch culvert may be a better option than a round culvert.

As with other types of crossings, the hydraulic capacity of vented crossings must accommodate the 100-year flood flow plus associated sediment and debris. This typically requires the crossing to have a pronounced dip to provide an adequate wetted perimeter to accommodate overtopping flows without risk of diversion.³⁷ If the plugging potential at the crossing site is low, it may be appropriate to assume the culvert will remain partially open and able to pass a percentage of the flood flow depending on the pipe hydraulics, anticipated debris loading, etc., with the remainder of the flood flow overtopping the crossing. If the plugging potential at the crossing site is high, it is best to assume that the culvert will become plugged during a large flood event and that 100 percent of the anticipated flow will overtop



Figure 14. A vented rock-filled crossing that is subject to debris flows in northeastern California.

³⁷ Schall et al. (2012) describe a method to determine the overtopping flow capacity of dipped road surfaces using the broad-crested weir formula.

the crossing. Where rock is used to armor the outfall of the crossing, such as in a vented rock-armored crossing, the rock armor should be designed based on the anticipated flow per unit width following the procedures outlined in Chapter IX. In some cases, the road surface is capped with concrete or other non-erodible material, and the armor on the fill face is grouted in place to prevent erosion (Weaver et al. 2015).

Due to site-specific conditions, such as limited rock availability, high diversion potential, or high overtopping flows, standard vented crossings may not be appropriate and alternative crossing designs may need to be considered. Sound engineering and construction principles must be employed when alternative types of crossings are used to ensure long term stability of the crossing.

For example, in some rare cases in the California Coast Ranges, site conditions may indicate that incomplete removal of pre-existing “Humboldt” crossings (crossings consisting of logs placed parallel to the stream channel and covered with fill) built with sound redwood logs could potentially limit the overall impacts on the environment by minimizing the area of disturbance. When this condition occurs, a vented crossing consisting of the remaining logs, a culvert, and large rock as described above, may be a feasible reconstruction alternative. This type of construction needs to be designed to allow low flows to pass through the remaining logs, culvert, and large rock, while allowing seasonal high flows to pass over the armored roadbed and down an armored fill slope to the natural channel below.

In the rare case where such an alternative design is proposed, special consideration should be given regarding the long-term strength and soundness of the buried redwood logs and their ability to provide adequate support for the crossing above, and the potential for interstitial flows to either scour below the logs, jeopardizing the foundation of the crossing, or create voids in the fill above through soil piping. This practice should only be used where it is expected to minimize sediment delivery compared to complete removal of the preexisting logs (particularly in watersheds with erodible soils where past crossing upgrade or removal work has resulted in considerable post-construction channel erosion). Supporting evidence justifying the proposed crossing design should be provided in written plans submitted for state review. This may include empirically-derived evidence from past buried log structures.

V.2 Construction Practices for Culvert Crossings

Additional elements can be incorporated into stream crossing design that can reduce the risk of crossing failure and potential impacts to watercourses if crossings fail. This may include conducting a channel stability assessment through visual inspection prior to crossing design work. Proposed crossing designs should be adjusted to fit all of the field conditions present (field data can be recorded on the form included in Appendix E, Part B).

The height of fill that will exist above a culvert should be accounted for when determining the appropriate pipe diameter. In general, the higher the fill, the larger the pipe diameter that should be installed. For example, a rule of thumb that has been used in the past increases the pipe diameter by 6 inches for every 5 feet of fill above the pipe on the discharge side of the crossing. The increased pipe diameter provides an added margin of safety to reduce (1) the need for replacement of a failed crossing that would be relatively expensive compared to the cost of a slightly larger diameter pipe, and (2) the higher environmental consequences of failure due to the larger volume of fill. Additional information on large fills and high-risk crossings is provided in Chapter VIII.

Minimizing the amount of crossing fill and constructing a crossing without diversion potential can significantly reduce sediment impacts if a crossing fails (Furniss et al. 1997, Weaver et al. 2015, Keller and Ketcheson 2015). Constructing a crossing that prevents the potential for diversion is more cost- and time-effective than having to continually maintain a crossing and road in order to accommodate flow and prevent a stream diversion, or repair/rebuild a large section of road damaged by a stream diversion that results from a plugged culvert (Figure 15).

Waterbars rarely prevent stream diversions when culverts plug during a large storm and they also require long-term maintenance. In contrast, a broad critical dip (diversion dip) at a watercourse crossing, when properly constructed, is a low-maintenance permanent structure that allows for the passage of standard log trucks at reduced speeds. The critical dip must be designed and constructed to reduce the potential for overtopping flows to erode the crossing fill (Weaver et al. 2015, Keller and Ketcheson 2015). It is normally constructed to discharge at the intersection of the crossing fill with the valley wall (i.e., hinge point), but it can be built to flow over the fill face in an armored spillway (BOF Technical Rule Addendum No. 5). The California Forest Practice Rules [14 CCR § 923.9 [943.9, 963.9] (k) and (j)] require that (1) all permanent watercourse crossings be constructed or maintained to prevent diversion of streamflow down the road should the drainage structure become plugged, and to minimize fill erosion should the drainage structure become obstructed, and (2) critical dips must be installed for crossings utilizing culverts, except where diversion is addressed by other methods. Alternative practices (i.e., exceptions to the rules), such as installing a significantly oversized culvert, may be more appropriate in some situations, and can be proposed by the RPF [14 CCR § 923 [943, 963 (c)].

Armoring the crossing fill face around the culvert inlet will reduce the potential for flow to erode the crossing fill or undercut the pipe, as well as improve pipe efficiency by having the pipe inlet flush with the rock armor (Clarkin et al. 2006). When armoring the crossing fill face with rock riprap, it is best to first place graded small rock, gravel, or a geotextile fabric under coarse riprap to prevent fines from being scoured and migrating through the rock protection. Other methods that are cost effective for reducing crossing failure risk include installing an emergency overflow pipe that is 50-60% of the design diameter (usually not less than 36 inches in diameter), and where necessary, installing slotted culvert



Figure 15. A diverted stream channel caused by a blocked 48 inch culvert in the Six Rivers National Forest (photo from Carolyn Cook, Six Rivers National Forest).

inlet snorkels or “slotted risers” at the pipe inlet for emergency overflow protection (Weaver et al. 2015).³⁸

Road surface runoff and surface erosion need to be considered during design of the crossing and approaches to minimize the potential for sediment delivery to the watercourse being crossed. Where the road surface is outsloped or flat as it approaches and passes over the crossing fill, the downslope discharge area will need to be designed to prevent surface erosion. Where the road surface is insloped, the inboard drainage ditch must be resistant to erosion and avoid discharging fine-grained sediment into the crossing inlet where feasible (i.e., hydrologic disconnection) (see 14 CCR § 923.2 [943.2, 963.2] (a)(5) and BOF Technical Rule Addendum No. 5). Additional erosion control structures, such as rolling dips, waterbars, or ditch relief cross-drains, placed close to crossing fills are useful in minimizing the amount of road surface runoff and sediment delivery

³⁸ For example, snorkels or slotted risers may be appropriate in recently burned watersheds and for high risk crossings in areas with limited winter maintenance. Greater detail is provided by Weaver et al. 2015.

to the watercourse from the approaches. The location and spacing of the ditch relief culvert outlet (or other drainage structure outlet) should be placed where an adequate filter strip exists to dissipate the flow and trap sediment before the water (and entrained sediment) enters the stream channel (Kramer 2001).

When a crossing is reconstructed, crossing-induced sediment accumulations in the channel upstream of the culvert inlet must be carefully removed or stabilized before installation of the new culvert [14 CCR § 923.9 [943.9, 963.9] (n)].³⁹ This will allow the new culvert to be installed closer to the original channel grade, thereby facilitating sediment transport through the culvert (minimizing the potential for sediment accumulation at the inlet and plugging) and reducing the likelihood of post-reconstruction headcutting through the sediment that had accumulated immediately upstream of the crossing. The reconstructed channel and newly installed culvert gradient should be consistent with the natural channel gradient both upstream and downstream of the crossing. If a new culvert is being installed, the gradient of the culvert should be designed so that the flow velocity through the culvert does not result in inlet deposition or outlet scour. Culvert slopes of less than three percent may be prone to bedload sediment accumulation and reduced efficiency (Beschta 1984). **Additionally, a minimum diameter of 24 inches is recommended for watercourse crossings to reduce plugging potential in channels that receive flood flows** (i.e., not crossings receiving discharge solely from small seeps or springs).

V.3 Fish Passage at Watercourse Crossings

Natural stream bottoms are much better at facilitating fish passage than hydraulically smooth culverts that may exhibit high flow velocities or shallow flow depths. Therefore, bridges and other natural-bottomed watercourse crossing structures, such as open bottom arches and round or pipe-arch culverts buried with 20 to 40 percent of their diameter embedded into the channel bed, should be installed in fish-bearing channels. This recommendation applies to new crossing installations, and also to culvert replacements where standard non-embedded culverts may have previously existed. If a round pipe is significantly embedded in to the stream channel, it will have to be sized appropriately to maintain the ability to pass the anticipated 100-year flood flow plus associated sediment and debris (see Figure 16 for an approach that can be used to determine the pipe size increase needed). Round and pipe-arch culverts with 20 to 40 percent of their diameter buried can be sized using readily available design nomographs with inlet control (e.g., Normann et al. 2005, Schall et al. 2012).

Where fish passage is a concern, an approach that incorporates geomorphic, hydraulic, and ecological requirements for fish passage, such as stream

³⁹ If a culvert is removed, or replaced with a bridge or open-bottomed arch, grade control structures may be required to prevent widespread channel adjustment (i.e., culvert replacement or removal may allow channel incision to progress upstream). Castro (2003) provides guidance on approaches to limit this potential problem. Grade control structures are generally not recommended for use in fish bearing watercourses.

simulation, should be identified and applied.⁴⁰ Unlike traditional hydraulic design approaches that exclusively address flow capacity, stream simulation incorporates additional design elements that consider fish passage and passage of woody debris. Stream simulation designs seek to maintain continuity in channel morphology and flow hydraulics through the watercourse crossing. Design elements include maintaining a natural channel substrate through the watercourse crossing and ensuring continuity of the channel cross-section through the crossing structure, with dimensions and characteristics similar to the adjacent natural channel (USFS 2008). Stream simulation structures are typically designed with HW/D ratios of between 0.5 and 0.7 for the 100-year flood flow. This makes these types of crossings less prone to contraction scour that can flush the pipes of the aggraded sediment needed to maintain fish passage, and sediment and debris accumulations at the pipe inlet that may lead to crossing failure (Gillespie et al. 2014). For example, in Vermont damage was largely avoided following a large flood event at two stream crossings where stream simulation design was implemented, while damage was extensive at multiple road–stream crossings constructed using traditional undersized hydraulic designs (Gillespie et al. 2014).

For detailed discussions of culvert design criteria for fish passage, refer to USFS (2000), NMFS (2001), ODF (2002a), Flosi et al. (2003), WDFW (2003), USFS (2008), DFW (2010), Kilgore et al. (2010), and Barnard et al. (2013).

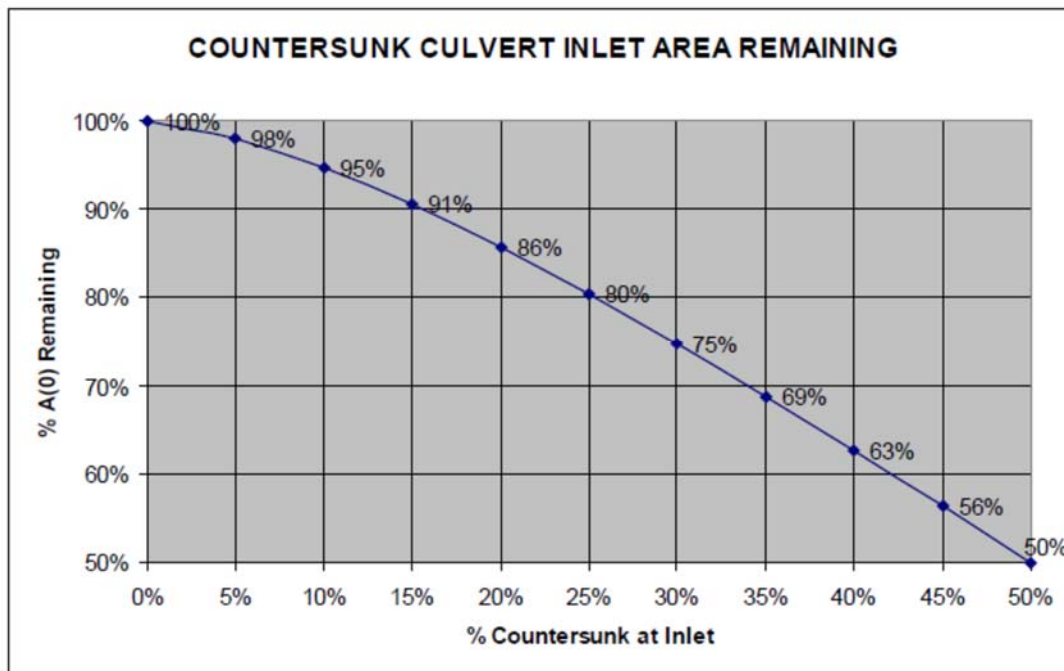


Figure 16. Percent of round culvert area at the inlet remaining open (% A(O)) versus percent of countersunk (WADNR 2013).

⁴⁰ These approaches and others are described in Part XII, Fish Passage Design and Implementation, California Salmonid Stream Habitat Restoration Manual, 4th ed. (DFW 2010), and in Barnard et al. 2013.

VI. Post-Fire Considerations for Flow, Sediment, and Debris

Runoff can increase substantially following wildfire. Runoff increases are associated with the alteration of several hydrologic processes, including (1) reduced interception and evapotranspiration, (2) reduced ground cover, (3) reduced infiltration and increased overland flow, and (4) potentially increased snow accumulation (Neary et al. 2005). Increased runoff may result from the creation of hydrophobic (water repellent) soils, but the magnitude of fire-induced repellency depends on the fire severity, type of vegetation present, soil texture, and water content of the soil (DeBano 2000).

As a result, peak flows in watersheds burned at moderate and high burn severity may increase by several orders of magnitude relative to flow in unburned watersheds (Foltz et al. 2009). The magnitude of augmented runoff increases with decreasing recurrence interval and with decreasing drainage area (Foltz et al. 2009). Post-fire runoff increases generally recover after approximately four years (Robichaud et al. 2010). Measured post-fire changes in peak discharge have been much larger than measurements of post-fire changes in annual runoff (Moody and Martin 2001).

Detailed descriptions of methods for post-fire peak flow predictions are beyond the scope of this document.⁴¹ However, runoff increases from fire can be accounted for in the Rational Method by increasing the runoff coefficient (Easterbrook 2006) and decreasing the time of concentration due to diminished hillslope roughness (Moody and Kinner 2005). The USGS Magnitude and Frequency Method can be used to predict post-fire runoff by multiplying the pre-fire 100-year flood flow by an area-weighted modifier that takes into account the percent runoff increase due to fire (Foltz et al. 2009). The percent increase in post-fire runoff used in the modifier is obtained from existing literature and past studies, or is estimated with best professional judgment. Computer software such as the USACE HEC-HMS and the NRCS TR-55 programs can also be adapted to post-fire situations (Cydzik and Hogue 2009, Kinoshita et al. 2013).

High severity wildfire can increase erosion rates by several orders of magnitude (Robichaud et al. 2010). Sediment is removed from hillslopes via surface erosion and mass wasting, and from the headwater channels through scouring by debris flows and gullying (Benda et al. 2005). Entrained (bulked) sediment can increase peak flows from 0.5 to 3 times clear water flow, particularly in the first post-fire winter, elevating flooding risk (Schuirman and Slosson 1992, Hamilton and Fan 1996, West Consultants, Inc. 2011). Additionally, large woody debris transport is significantly increased following wildfire (Benda and Sias 2003). Altogether, the greatly increased fire-induced fluxes of flow, sediment, and debris can increase the potential for culvert blockage and the overtopping of watercourse crossings (e.g., Bachmann et al. 2014, Figure 17). These factors should be considered

⁴¹ For additional information on post-fire peak flow predictions in forested watersheds, consult Mai 2003, Kaplan-Henry 2007, Seibert et al. 2010, Moody 2012, Kinoshita et al. 2013, and Yochum and Norman 2015.

when inventorying existing stream crossings during post-fire assessments and designing upgraded structures in high risk situations. Post-fire emergency protection measures for crossings may include installation of oversize culverts, emergency overflow pipes, slotted culvert risers, and flared metal end sections, as well as removal of undersized pipes followed by construction of armored crossings designed to accommodate debris flows.



Figure 17. Plugged culvert located in the 2015 Butte Fire area (Calaveras County) during winter storms in March 2016 (photo provided by Cheryl Hayhurst, CGS).

VII. Evaluating Existing Watercourse Crossings for Risk of Failure

Many of the concepts used for sizing new culverts can also be used for evaluating existing culverts to determine which ones are presently at high risk for failure. Hillslope monitoring efforts completed over the past two decades on Timber Harvesting Plans (THPs) throughout California on non-federal commercial forestland suggest that numerous existing crossings are at high risk for failure. Frequently documented problems associated with watercourse crossings have included culvert plugging, stream diversion potential, fill slope erosion, scour at the outlet, and ineffective road surface drainage immediately up road of crossings (Cafferata and Munn 2002, Brandow et al. 2006, Brandow and Cafferata 2014). Approximately 35% of the watercourse crossings and road approaches that were randomly selected for past monitoring programs exhibited

significant effectiveness problems from 1996-2001, 20% from 2001-2004, and 15% from 2008-2013 (Brandow and Cafferata 2014). Bundros et al. (2003) classified 20% of 2,300 evaluated stream crossings in the Redwood Creek watershed as “critical crossings,” which were defined as having diversion potential, an undersized culvert, and a moderate or higher plugging potential.

Crossing inventories are an important component of a road management plan that aims to reduce sediment delivery to watercourses and to prevent road damage (see Flanagan et al. (1998), Flanagan and Furniss (1997), DFW (2006), and Weaver et al. (2015) for additional information). Examples of items to consider as part of a crossing inventory include the situations listed below.

Crossings at high risk from wood and sediment plugging:

- culvert diameter divided by active channel width is less than 0.7 ⁴²
- poor culvert inlet alignment with the stream channel
- HW/D ratio greater than 0.67 with concentrations of mobile woody material upstream of a culvert inlet
- crushed and/or plugged pipe inlet
- unusually wide areas, including basins constructed for water drafting or trapping sediment, near the inlet of the pipe
- alluvial sediment deposits upstream from the culvert inlet, indicating the culvert has plugged or been exceeded in the past, and sediment was deposited in standing water, then scoured when flows receded
- strandline of deposited woody debris marking a previous high water surface above the culvert inlet
- a stream diversion gully found nearby or down the road that can be traced back to the crossing, indicating the culvert has plugged or been exceeded in the past
- piles of excavated sediment nearby, indicating a plugged culvert inlet was reopened using a backhoe or excavator
- pipe located in a channel with unusually high mobile wood and/or sediment loading
- culvert gradient less than 3 percent
- culvert gradient less than the natural stream channel gradient
- culvert inlet downstream of active mass wasting or in a channel that is prone to debris flows or hyper-concentrated flows
- undersized culverts that pond flow (backwater) and cause accelerated sediment deposition upstream of the culvert inlet during high flows
- culvert undersized in an area with high bedload transport

Crossings at high risk for hydraulic capacity exceedance:

- existing pipe capacity has less than 100-year flow capacity
- crushed or plugged pipe inlet

⁴² Research conducted in northwestern California showed that culverts sized at 0.7 times the mean stream bed width will pass, on average, 95% of fluviually transported wood greater than 12 inches long (Flanagan 2004).

- evidence of insufficient hydraulic capacity is present. Examples include:
 - deposits of sediment immediately upstream of the crossing (aggradation)
 - evidence of overtopping of the crossing by peak flows

Crossings at high risk for causing significant erosion (e.g., gullyng, landsliding):

- diversion potential exists (the road grade through the crossing is such that a stream will leave its natural channel and flow down the road if the crossing plugs or its capacity is exceeded)
- the low point of the crossing is located over the axis of a deep fill without an armored spillway

Crossings in need of replacement due to age-related deterioration or wildfire:⁴³

- the length of time the culvert has been installed is approaching or has exceeded the expected service life for a given region⁴⁴
- moderate or high degree of steel pipe abrasion and/or corrosion
- all or part of a HDPE plastic culvert is missing due to melting from burning

Crossings with fish passage limitations [design criteria for fish passage are described in USFS (2000), NMFS (2001), ODF (2002a), Flosi et al. (2003), WDFW (2003), USFS (2008), and DFW (2010)]:

- outlet is elevated greater than juvenile or adult fish jumping ability
- excessive culvert gradient, resulting in water velocities that exceed fish swimming ability and endurance
- insufficient water depth in the culvert for fish passage
- the bottom 20 percent or more of the culvert is not buried in stream gravels (note that this does not ensure fish passage if present)

Following the completion of the field inventory, a schedule prioritized by risk level should be developed and funding secured to make needed corrections. For an example stream crossing assessment methodology, data form, site prioritization strategy, and best management practices, see DFW (2006).

⁴³ In some situations, trenchless technologies may be advantageous for rehabilitating or replacing corrugated metal pipes, see Matthews et al. (2012) for detailed information.

⁴⁴ The service life of a steel culvert varies depending on local corrosion rates, but culverts generally last at least 25 years (Pyles et al. 1989). Accelerated corrosion and low service life have been linked to (1) water with a low pH, and (2) low soil resistivity of the site and backfill materials (i.e., relative quantity of soluble salts in the soil or water), and (3) high abrasion from significant coarse bedload sediment. Service life generally ranges from 20 to 50 years (Caltrans 1999, Molinas and Mommandi 2009). The expected service life of HDPE plastic pipes is 75 years (without fire damage) (Molinas and Mommandi 2009).

VIII. High-Risk Crossings and Large Fills

A key component of evaluating the potential impacts associated with watercourse crossing failure is determining the volume of fill material in the stream channel at the crossing site. This concept has been incorporated in the California Forest Practice Rules since 2010, with the passage of the Anadromous Salmonid Protection Rules by the Board. The 2017 California Forest Practice Rules (FPRs) include 14 CCR § 923.9 [943.9, 963.9] (o), which states:

“Where crossing fills over culverts are large, or where logging road watercourse crossing drainage structures and erosion control features historically have a high failure rate, such drainage structures and erosion control features shall be oversized, designed for low maintenance, reinforced, or removed before the completion of timber operations or as specified in the plan. Guidance on reducing the potential for failure at high risk watercourse crossings may be found in “Board of Forestry Technical Rule Addendum Number 5: Guidance on Hydrologic Disconnection, Road Drainage, minimization of Diversion Potential, and High Risk Crossings” (1st Edition), hereby incorporated by reference.”

“Large” fills are not defined in the FPRs or in Board of Forestry Technical Rule Addendum Number 5. However, thresholds for what constitutes a “large” fill have been proposed. For example, the Oregon Department of Forestry’s Forest Practice Rules use a fill height of 15 feet or greater above the culvert to trigger additional work by the operator (Oregon Department of Forestry 629-625-0320), and the Central Valley Regional Water Quality Control Board (CVRWQCB) uses a fill volume of 500 cubic yards or a fill height of 25 vertical feet as triggers in their timber Waiver requiring the review of the watercourse crossing by a California Professional Engineer (PE) or Professional Geologist (PG) (Order No. R5-2017-0061). Although not explicitly indicated, it is implied that the fill height thresholds listed in both of these examples are maximum values and are measured vertically from the outside (downstream) edge of the road surface to the top of the culvert (see Figure 18). In the case of existing crossings where the culvert was not installed on grade but daylighted in the face of the fillslope above the channel, it is recommended that the fill height be measured from the outside edge of the road surface to the toe of the fill slope in the channel.

Table 4 presents maximum fill heights and estimated fill volumes over a two-foot wide channel width for various crossing configurations. Review of Table 4 reveals that in general it is reasonable to characterize “large” fill volumes as having a **15-foot maximum fill height at the outfall or 500 cubic yards of fill**, although site- specific information for a given field situation may indicate that other thresholds are appropriate. To assist RPFs in quantifying fill heights and fill volumes, simple tables and a volume estimate calculator (Excel spreadsheet) are provided (Appendix C and available at www.fire.ca.gov, respectively).

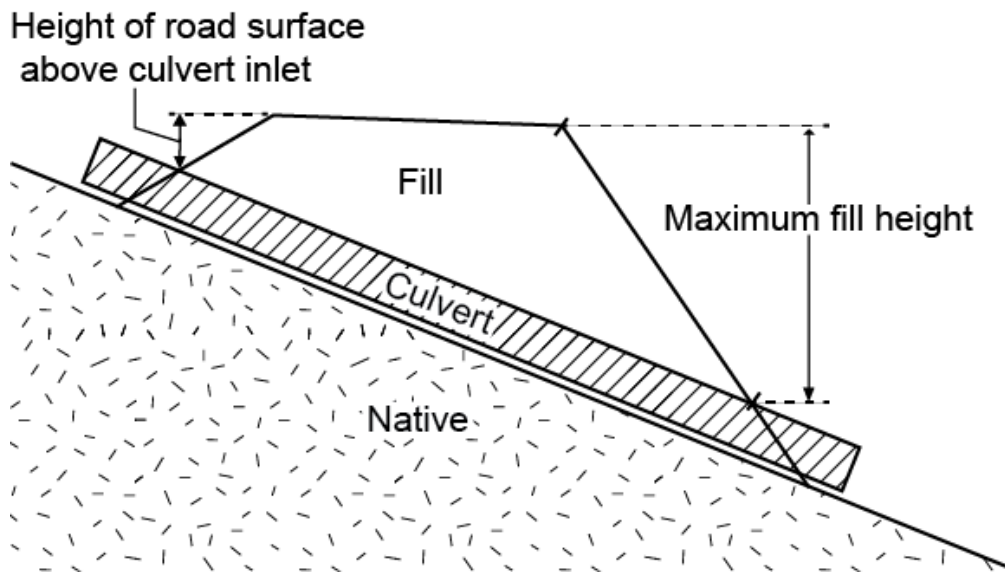


Figure 18. Fill height diagram.

Table 4. Comparison of crossing fill height to fill volume over a two-foot wide channel for different channel slope conditions. Crossings right of the stepped red line have fill heights greater than 15 feet or fill volumes greater than 500 cubic yards.

Assumes: Road Width of 16 feet; Fixed channel width of 2 ft.; Fillslope inclination of 67% (1.5h:1v); and 67% side slopes.

		Height of road surface (fill) above culvert inlet, ft.												
		2	4	6	8	10	12	14	16	18	20	22	24	26
Channel slope, percent.	0	2.0	4.0	6.0	8.0	10.0	12.0	14.0	16.0	18.0	20.0	22.0	24.0	26.0
		12	35	74	131	209	311	439	596	786	1012	1275	1580	1928
	5	3.1	5.3	7.6	9.8	12.0	14.3	16.5	18.7	21.0	23.2	25.4	27.7	29.9
		18	49	97	168	264	388	545	737	969	1243	1564	1934	2358
	10	4.4	6.9	9.4	11.9	14.4	16.9	19.4	22.0	24.5	27.0	29.5	32.0	34.5
		28	67	129	218	338	494	689	928	1216	1556	1954	2413	2937
	15	5.9	8.8	11.6	14.4	17.3	20.1	23.0	25.8	28.6	31.5	34.3	37.2	40.0
		41	93	174	288	442	640	888	1192	1557	1987	2490	3070	3732
	20	7.8	11.0	14.3	17.5	20.7	24.0	27.2	30.5	33.7	36.9	40.2	43.4	46.6
		62	132	238	388	589	847	1170	1564	2036	2594	3244	3993	4848
	25	10.1	13.9	17.6	21.3	25.1	28.8	32.5	36.3	40.0	43.7	47.5	51.2	54.9
		92	189	334	537	808	1154	1579	2103	2730	3470	4331	5323	6454
	30	13.1	17.5	21.8	26.2	30.5	34.9	39.3	43.6	48.0	52.4	56.7	61.1	65.4
		140	277	481	764	1139	1618	2214	2939	3806	4827	6015	7382	8941

= Maximum height of fill above culvert outlet, ft.
 = Volume of fill, cubic yards

There are two major considerations that should be addressed when evaluating watercourse crossing sites for new culvert installations or upgrades: (1) the risk of catastrophic watercourse crossing failure, and (2) the potential impacts downstream in the event of catastrophic failure. The risk of catastrophic failure (e.g., significant loss of the road prism) is not directly related to fill volume, but is more a function of other factors that include:

- Watercourse crossing hillslope position (upper, middle, or lower slope), since greater flow and associated stream power generally occurs at lower hillslope positions.
- Inherent landslide potential for upslope and upstream hillslopes (e.g., high risk for debris slides, debris flows, debris torrents).
- Upstream land-use practices and slope conditions (characteristics that can affect peak flow, erosion potential, large woody debris (LWD) recruitment, colluvium entrainment, landsliding, post-fire or post-land use erosion, etc.)
- Local, reach, and watershed-scale fluvial geomorphic processes (e.g., stream power potential, large woody debris loading and movement potential (Lassetre and Kondolf 2001), stream gradient, evidence of debris flow deposits).
- Local hydrologic conditions (e.g., potential for major rain-on-snow runoff events).
- Soil strength characteristics of the anticipated fill material.
- Feasibility to conduct monitoring and maintenance activities (e.g., remoteness of the crossing, ease of access during winter).
- Adequacy of construction techniques.
- Crossing design (e.g., undersized culvert, poor crossing location, or the wrong crossing type for the site conditions, such as the installation of a permanent culvert crossing when a temporary crossing or bridge would have been a better choice).
- Diversion potential.

Generally, the larger the fill volume, the more potential there is for adverse downstream impacts to occur in the event of catastrophic failure (Figure 19). Thus, fill volume is often proportional to the potential risk of adverse downstream impacts. However, there are additional factors that should be considered in evaluating the severity of a crossing's potential downstream impacts if it were to fail. Examples of potential significant downstream impacts include:

- Threats to human life, safety, and infrastructure.
- Threats to key beneficial uses of water, including municipal/domestic water supplies and fish habitat.
- Substantial sediment delivery to sensitive receptors such as a USEPA 303(d) listed waterbody, high value structure, public highway, etc.

The following matrix (Table 5) illustrates how the risk of failure and the potential severity of downstream impacts can be combined to evaluate a crossing's overall

threat as a high-risk crossing.⁴⁵ High-risk crossings (i.e., ranks 4 and 5) can be considered as presenting a high environmental and/or human life-safety risk, and require considerably more thought and attention in design and construction. Depending on the situation, other ranks may still warrant additional consideration.

Table 5. Rank of overall crossing threat (ranks 1-5) based on potential risk of failure and potential downstream impact.

		Potential Risk of Failure		
		Low	Moderate	High
Potential Downstream Impact	Low	1	2	3
	Moderate	2	3	4
	High	3	4	5



Figure 19. Example of a watercourse crossing with a large fill and a relatively small culvert.

⁴⁵ A similar risk assessment matrix has been proposed by Keller and Ketcheson (2015).

IX. Rock-Armored Crossing Design

Rock-armored crossings may be a better alternative than culverts for small headwater channels,⁴⁶ particularly where winter maintenance is difficult or debris flows are likely (Spittler 1992, Warhol and Pyles 1989, Weaver et al. 2015) (Figure 20). These types of crossings must be built using accepted practices and meet the requirements of the FPRs. For example, 14 CCR § 923.9 [943.9, 963.9] (l) of the 2017 FPRs states:

“Any necessary protective structures associated with logging road watercourse crossings such as wing walls, rock-armored headwalls, and downspouts shall be adequately sized to transmit runoff, minimize erosion of crossing fills, and prevent significant sediment discharge. **Rock used to stabilize the outlets of crossings shall be adequately sized to resist mobilization, with the range of required rock dimensions described in the plan.**”
[emphasis added]

Several alternatives for designing armored crossing outfalls to resist scour and protect the crossing exist, including use of gabion baskets (gabion mattress), concrete blocks, tieback walls, biotechnical stabilization, and various types of geosynthetic materials (e.g., geomembranes, geocells, turf reinforcement mats⁴⁷), but the most common armor used in the forest setting is rock riprap.⁴⁸

⁴⁶ Small headwater channels are considered channels with 100-year flows typically less than 100 cfs, but they may include those with higher flows where field conditions are appropriate (e.g., low gradient channel; wide, shallow channel; low volume of fill).

⁴⁷ Design methods for various outfall armor alternatives can be found at: <http://www.dtic.mil/dtic/tr/fulltext/u2/a354949.pdf> and <http://www.conteches.com/Knowledge-Center/PDH-Article-Series/Overtopping-Flow-Protection>.

⁴⁸ In rare cases log-armored crossings have been used. In the California Coast Ranges they have been constructed with sound, tight-grained redwood logs placed perpendicular to the slope in a step ladder arrangement at the outlet of a dip in an outsloped road, or at the outlet of very small, typically unclassified, drainages. In interior California they have generally consisted of a large diameter notched large log used alone where the fill height is minimal, or in conjunction with appropriately sized rock riprap below the notched log. The notched log is placed at the outboard edge of the road running surface to provide a somewhat stable outfall from a dip or small drainage. Several potential problems exist with log armored fill crossings, including (1) their expected short lifespan (particularly for non-redwood logs), (2) the high surface area and low density wood that limits its ability to resist hydrodynamic forces unless adequately anchored, (3) the difficulty to shape the outfall such that flow is not diverted to the edges of the logs and thus erode the unprotected slope, (4) the need for countermeasures to protect the base of the log outlet from scour and head cutting, and (5) the need to inspect and maintain the crossing on a regular basis and replace it when necessary. In the interior part of California, they should only be considered as temporary crossings, since the wood is expected to rapidly deteriorate (estimated to be as low as approximately 5 years, depending on log size, species, soil type, streamflow, etc.). To mitigate these potential problems, sound engineering and construction principles must be employed when these types of crossings are used. Any proposal for using log armored crossings should include specific construction details, as well as a rationale as to why other crossing alternatives are not feasible.

Simplified design methods to size rock riprap used to provide scour protection along the bed and banks of channels have been developed theoretically based on shear velocity and critical shear stress (Brown and Clyde 1989, Maynor et. al. 1989), and empirically based on field observations and laboratory testing (Racin et al. 2000). These solutions, however, are not particularly relevant to sizing rock used in riprap structures where overtopping flow occurs down steep (between 30% to 67%) channels, such as in rock-armored crossings where the crossing outfall can be as steep as 67%.

Appropriately designed rock riprap at the outfall of armored crossings should protect the crossing against scour (undercutting) and the loss of fill material downstream (Figure 21). However, an overestimation of the size and thickness of rock riprap required to protect the crossing fill can lead to excessive costs that make the project prohibitively expensive. Underestimating the size and thickness of the riprap layer can lead to negative consequences including economic losses, losses to infrastructure, and impacts to downstream beneficial uses. The following discussion introduces methods that RPFs and other licensed professionals can apply to design rock riprap in rock-armored crossings subjected to overtopping flows consistent with the requirements of 14 CCR § 923.9 [943.9, 963.9] (I) of the 2017 California FPRs.



Figure 20. Example of a rock-armored crossing installed in LaTour Demonstration State Forest, Shasta County.



Figure 21. Rock-armed crossing where the outlet rock was undersized, poorly placed, and with insufficient quantity, resulting in post-construction failure. This situation illustrates the need for appropriate sizing of the rock on the outboard edge and close oversight during construction.

These methods are intended to provide forest managers, operators, and regulators information to elevate the current standard of practice being applied in the design and construction of rock-armed crossings. They are not intended to be used as rule requirements or regulations, and the extent to which the methods are applied should be based on sound professional judgment after considering all site-specific conditions, including the potential life-safety risks and environmental impacts present.

IX. 1 Sizing Rock Riprap in Rock-Armored Crossings Subjected to Overtopping Flow

Studies by Abt et al. (1987, 1988) and Robinson et al. (1998) were conducted to assess the stability of rock riprap under overtopping flows (e.g., often shallow, rapidly accelerating flow down the fall line of slopes). These studies were performed with rock diameters of about 6 inches or less and slopes of 40% or less. Consequently, additional experiments were conducted by Mishra (1998) to evaluate the stability of rock riprap on slopes as steep as 50% (2h:1v).

Mishra (1998) recognized that the hydraulics and tractive shear forces acting to dislodge rock riprap in overtopping flows down steep slopes cannot be analyzed with standard flow and sediment transport equations traditionally used for stream bed and banks due to the shallow, highly turbulent and aerated flow over a rough surface. For this reason, Mishra (1998) developed empirically derived riprap design criteria based on large-scale flume studies. This method takes into account the material properties of the rock riprap (median size, shape, gradation, porosity, and specific gravity), the embankment slope, and the unit discharge (flow per unit width, ft^2/sec). This work was funded through a cooperative agreement between the U.S. Bureau of Reclamation (USBR) and Colorado State University (CSU), and is commonly referred to as the USBR/CSU method of

sizing rock riprap in overtopping flows. It has been adopted by the Federal Highway Administration (Lagasse et al. 2001) and the Transportation Research Board of the National Academies (Lagasse et al. 2006). Detailed information on this method is provided in the following section and Appendix D.

IX. 2. USBR/CSU Method

Steep embankments composed of homogenous material (soil or rock) subjected to overtopping flows where there is low tail water control can fail as a result of erosion that typically initiates along the slope and progresses in the upslope/upstream direction, as illustrated in Figure 22 (adapted from Chen and Anderson 1987). These conditions are present for most rock-armored crossings on moderate to steep channels (>10%). **This failure mechanism illustrates the importance of, and need for, an effective rocked transition between the base of the outfall and the natural channel.** The USBR/CSU method provides a universal riprap design equation to predict the size of riprap to be used to armor steep slopes against this type of failure caused by overtopping flows (Lagasse et al. 2006). For embankment slopes with gradients steeper than 25% (4h:1v), the USBR/CSU method assumes that all the flow is contained within the thickness of the riprap layer (interstitial flow). This method employs a step-wise approach involving two primary equations, one to calculate the average rock size, d_{50} , for which 50 percent of the rock in the riprap is smaller by weight, and the second to calculate the interstitial flow velocity, V_i . Design equations and the input parameters used in the USBR/CSU method are discussed in Appendix D, Part A, and a completely worked sample problem to illustrate the design procedure is provided in Appendix D, Part B.

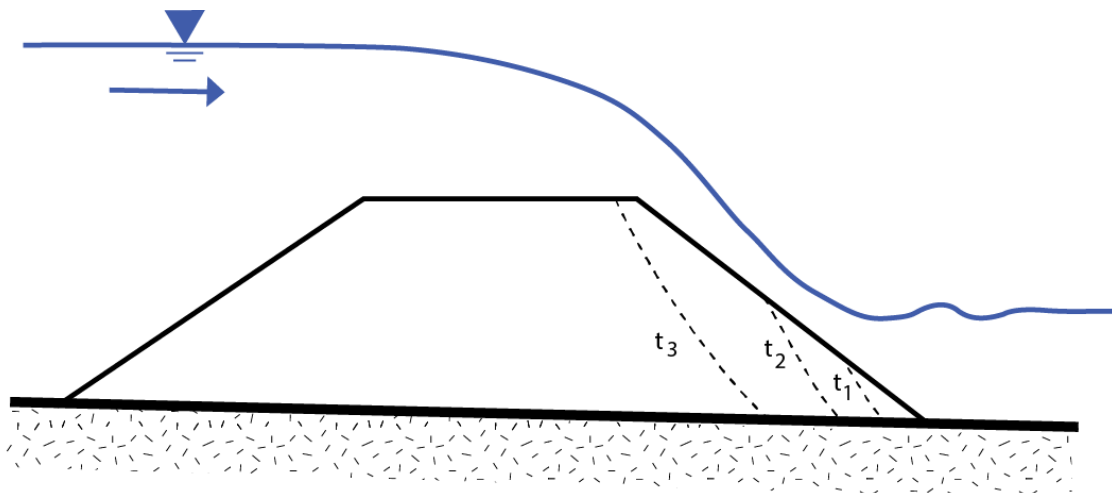


Figure 22. Typical embankment erosion pattern with free flow (t =time), adapted from Chen and Anderson (1987).

RPFs and other licensed professionals are encouraged to perform the USBR/CSU rock riprap design based on site-specific conditions for proposed rock-armored crossings that are determined to have a high environmental or human safety risk or have unique physical or hydrological characteristics that must be considered (see Table 5). A spreadsheet is provided at the following CAL FIRE web address to assist licensed professionals in performing these calculations:
http://calfire.ca.gov/resource_mgt/resource_mgt_forestpractice_pubsmemos_public.php

IX. 3 Simplified Approach to Design Rock Riprap Under Overtopping Flows

To provide RPFs with a simple approach to estimate the size of rock riprap needed for rock-armored crossings, a nomograph has been developed that allows the design of rock riprap based on the anticipated 100-year flood flow at the crossing site and on the anticipated embankment slope ranging from 33% (3h:1v) to 67% (1.5h:1v) (see Figure 23).

To use the nomograph, draw a horizontal line extending the entire width of the graph along the y-axis that corresponds to the 100-year flood flow estimated at the site. Where the horizontal line intersects the plotted curves, extend lines vertically to the X-axis and record the values for the parameters required to complete a rock-armored crossing design, including the d50 rock size, the thickness of the rock riprap layer measured normal to the fillslope, and the minimum width and depth of the rock chute/outfall. An example crossing design using the simplified nomograph is provided in Appendix D, Part C.

This simplified nomograph is intended for use in the forested setting and is developed based on the USBR/CSU design method with the following assumptions. **If any of the following assumptions do not apply, then use of the full USBR/CSU design method described in Appendix D, Part A may be more appropriate to size rock riprap for the armored crossing.**

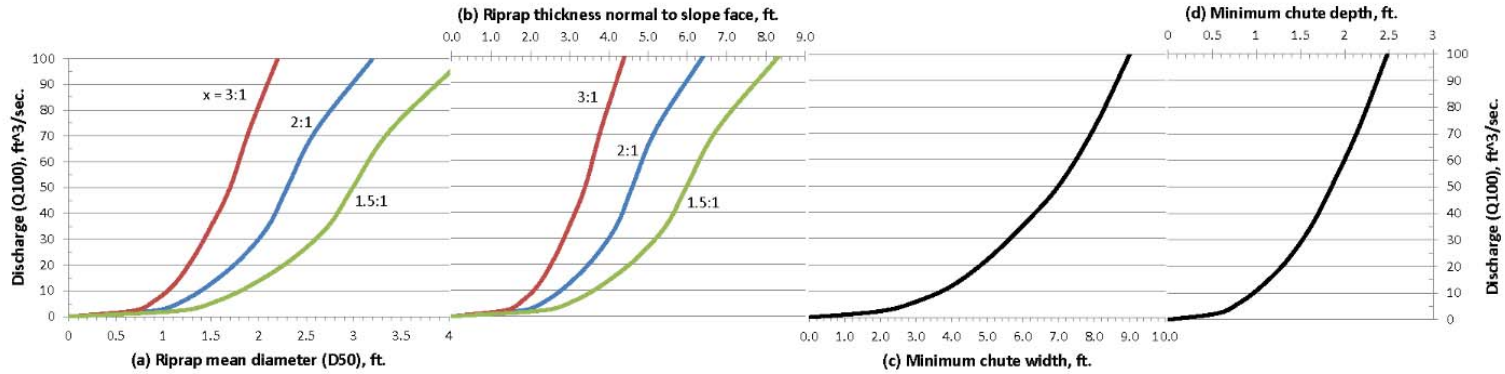
Assumption 1: The proposed crossing has anticipated 100-year flood flows of **100 cfs** or less.

Assumption 2: The rock riprap used to armor the crossing is selected to represent 'typical' material obtained from pit run sources and sorted to be used as rock riprap. It has the following general properties: the material is angular with an angle of repose (in this case equal to the angle of internal friction), **ϕ , of 40 degrees**; is well-graded with a uniformity coefficient, **C_u , equal to 1.75** and a porosity, **n , of 0.40**; and has a specific gravity, **S_g , of 2.65**.

Assumption 3: The highest unit discharge occurs near the toe of the outfall where flows are re-concentrated into the natural channel and is based on an assumed chute/outfall width roughly equivalent to 2 times the diameter of an appropriately sized, inlet-controlled culvert flowing full (HW/D=1) with the 100-year flood flow at the site.

The 2 times equivalent culvert diameter approach was identified as a reasonable surrogate to estimate the width used to calculate the unit discharge for the following reasons: culverts sized for the 100-year flood flow generally fit the active channel width in confined, moderate to steep (>10% grade) mountainous streams where most rock-armored crossings are proposed, and although it has not been thoroughly tested, preliminary field review of several crossings found that the approach reasonably represents the width of concentrated flow down rock-armored crossings during high flow events. Moreover, the resulting peak unit discharge calculated using the 2 times equivalent culvert diameter approach produced results of approximately 10 cfs/ft or less, which are within the range originally tested with the USBR/CSU method. The only exception is for 100 cfs flows at the top of the scale, which equate to a slightly higher unit discharge of about 11 cfs/ft. Thus, additional caution should be considered based on site specific conditions as flows approach 100 cfs.

Simplified design of rock-armored crossings



Note: Slopes armored at a 1.5:1 slope inclination shall be less than 15 feet in height, and be constructed by placing rock in a running bond with at least 3 points of contact and with the long axis of the rock placed horizontal and facing into the slope.

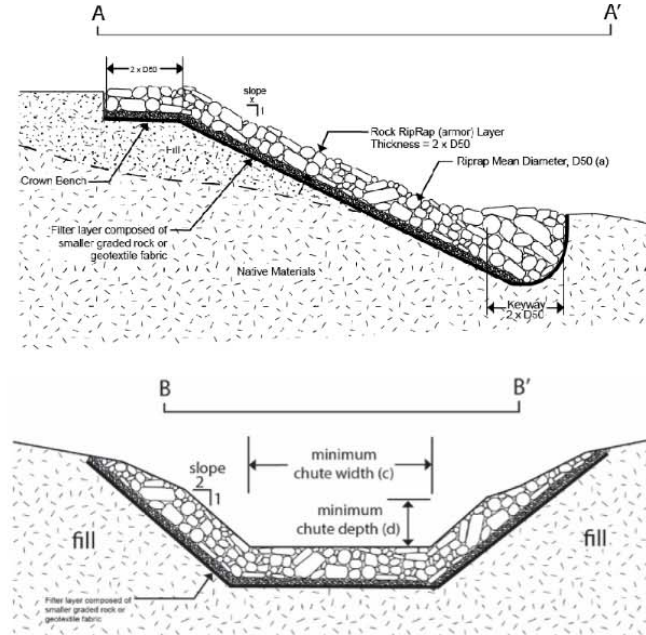
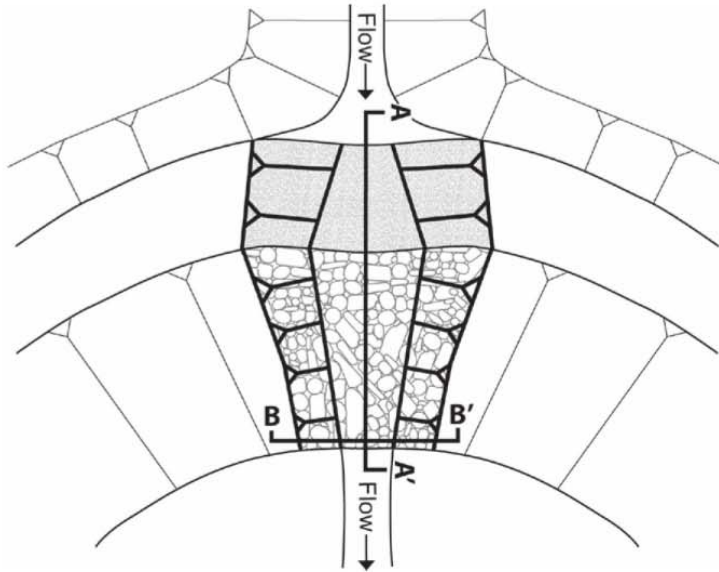


Figure 23. Simplified design nomograph (may be printed in an expanded 11 x 17 inch format).

Assumption 4: The rock chute/outfall down the fillslope has a minimum cross-section geometry equal to a trapezoid with a minimum base width equivalent to 2 times the diameter of an appropriately sized, inlet-controlled culvert flowing full ($HW/D=1$) with the 100-year flood flow and a minimum depth calculated using the broad-crested weir formula (Appendix D, Part A, Eq. 3), and side slopes inclined at 2h:1v.

Using the broad-crested weir formula to estimate the depth of water in the trapezoidal chute is conservative, particularly since it assumes all flow is constricted into the inlet of the chute before the flow goes through a hydraulic drop as it accelerates down the chute and, by design, the flow down the rock chute should all be within the rock riprap (i.e., flowing interstitially). However, placing a minimum depth dimension on the chute/outfall will help account for any concentrated flows that may occur down the armored fillslope and ensure that the flows are not easily diverted outside the protection provided by the rock riprap if the interstitial voids within the riprap layer eventually become filled with sediment and debris.

Assumption 5: Crossings with fillslopes as steep as 67 percent are stable against overtopping flows, provided they (1) have less than 15 feet of maximum fill height, and (2) are constructed with angular rocks that are machine placed (as opposed to randomly dumped) and oriented in a running bond pattern with at least three points of contact and with the long axis of the individual rocks sloping slightly into the fillslope.

Slopes steeper than 50 percent are outside the range of experimental data used in the USBR/CSU study and are critically close to the angle of repose of most rock riprap; thus, theoretically, it would not be appropriate to apply the USBR/CSU design method on slopes as steep as 67 percent. However, it is recognized that the rock riprap used in the USBR/CSU flume tests was randomly placed, resulting in riprap that is less stable than rock that is placed with purpose. Randomly placed rock shows traditional limit-equilibrium stability characteristics and fails as the fillslope approaches the angle of repose, while rock that is placed in a running bond pattern attains stability characteristics similar to those observed in dry-stacked rock walls, where the stability of the wall is best expressed through rock mechanics. For this reason, we consider it reasonable to apply the USBR/CSU method to low- to moderate-risk crossings with slopes as steep as 67 percent, provided that flows are ≤ 100 cfs and an appropriate factor of safety is applied to increase the rock size. Consequently, the d50 rock size shown in the nomograph for 67 percent slopes was calculated by applying a factor of safety of 1.3 to the d50 rock size calculated using the same material properties listed above.

Assumption 6: The rock-armored crossings are constructed following the sound construction standards outlined in the following section.

IX. 4 Recommended Construction Standards for Rock-Armored Crossings

Rock-armored crossings designed using the USBR/CSU method should be constructed in general agreement with Figures 24, 25, and 26.

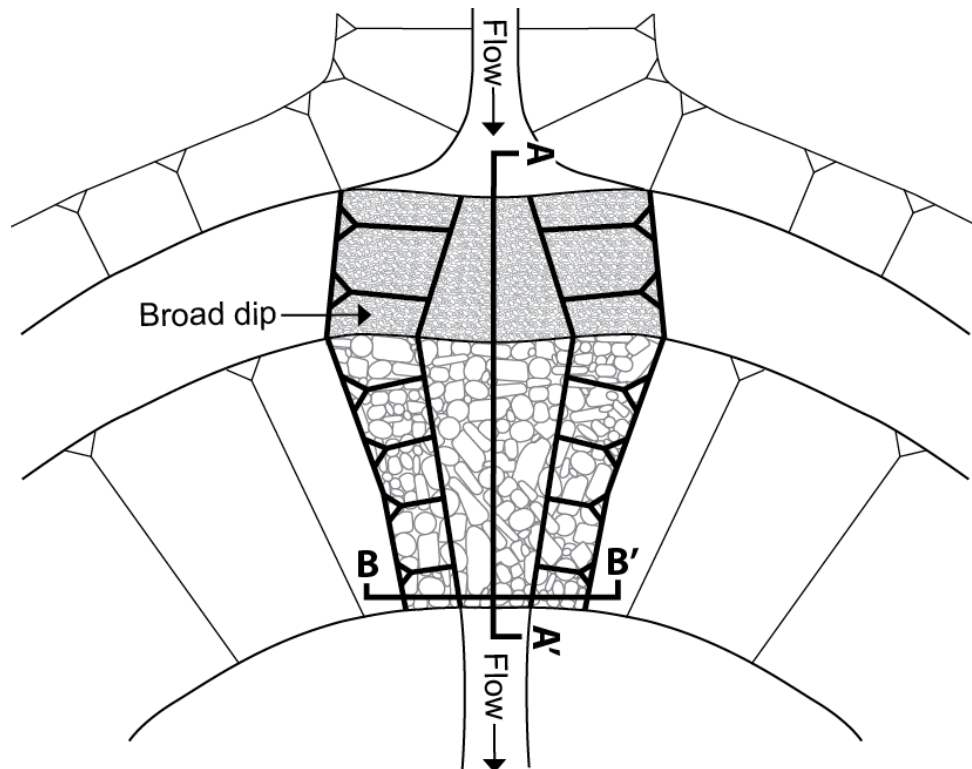


Figure 24. Design components of a rock-armored crossing—plan view, showing cross-sections A-A' and B-B'.

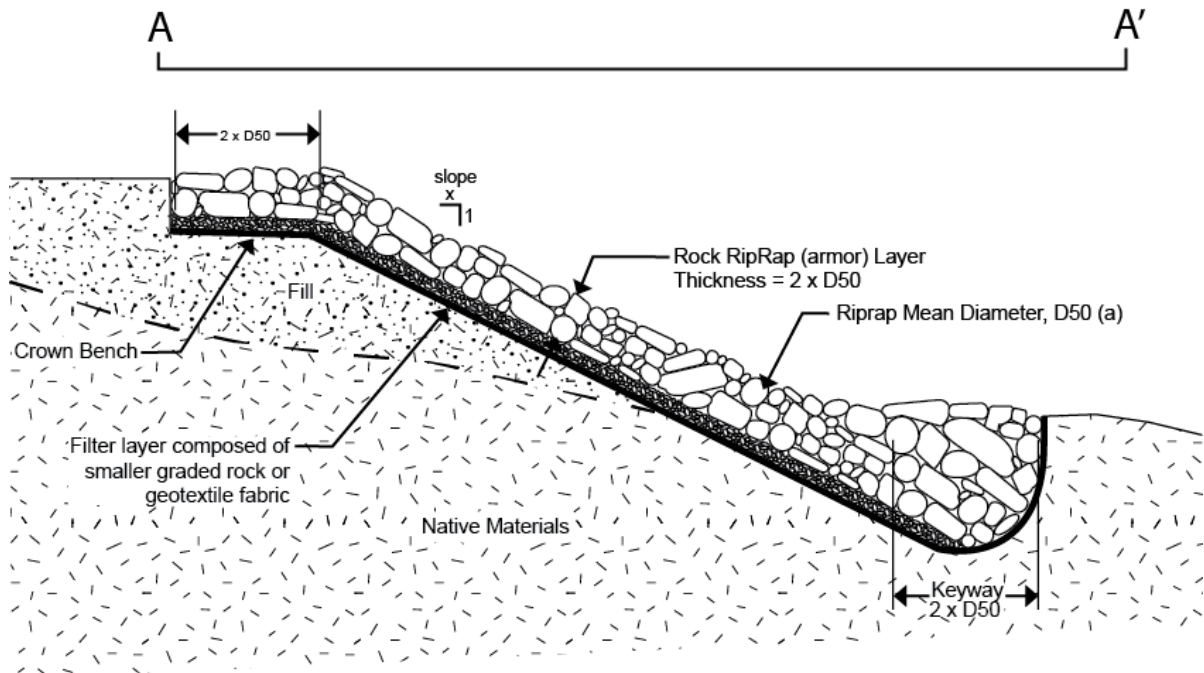


Figure 25. Rock-armored crossing—profile A-A' view.

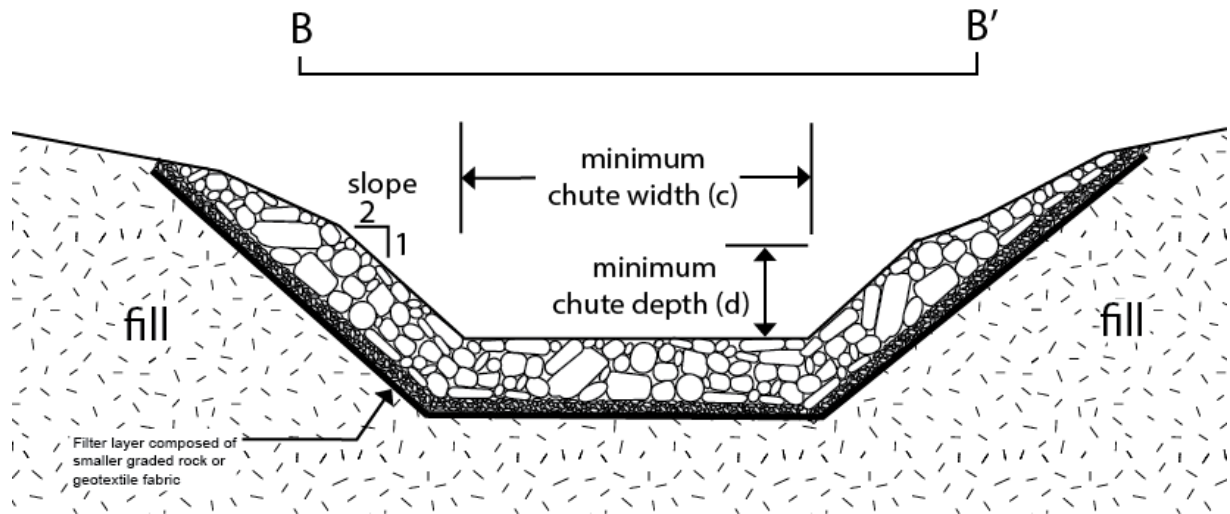


Figure 26. Rock-armored crossing—profile B-B' view.

As illustrated in Figures 24, 25, and 26, the primary components of a rock-armored crossing include:

Rock riprap (armor) layer: The rock riprap layer should be composed of competent, angular, narrow to well-graded material where about half the particles are larger and half are smaller (i.e., median size) than the d_{50} rock size determined using the USBR/CSU method to be hydraulically stable for the anticipated flow.

Filter layer: The filter layer (also commonly referred to as the backing layer) is placed between the outside riprap layer and the fill, and can be composed of smaller, well-graded aggregate or geotextile fabric. The filter layer prevents migration of fine soil in the fillslope through voids in the coarse outside riprap layer, distributes the weight of the riprap layer to provide more uniform settlement, and permits relief of hydrostatic pressures within the fillslope. If the filter layer is omitted, not installed correctly, or is not sized correctly (e.g., too small or not thick enough), excessive piping through the riprap layer can cause erosion and failure of the fillslope. Design and construction recommendations for both aggregate and geotextile fabric filter layers are provided in Brown and Clyde (1989). In general, aggregate filter layers should be a minimum of 6 inches thick or 4 times the d_{50} of the filter stone, whichever is greater. Additionally, geotextile fabric layers should be resistant to ultraviolet light, be durable (e.g., high grab strength, tear strength, and puncture resistance properties), and have appropriate hydraulic properties and apparent opening size characteristics for the anticipated site conditions. A common geotextile used for this purpose is a 6 to 8 ounce/square yard needle-punch non-woven fabric. Openings, designated by AOS (apparent opening size) and permittivity, depend on the soil type in accordance with American Association of State Highway and Transportation Officials (AASHTO) specification M-288.⁴⁹

⁴⁹ AASHTO Standard Specifications for Geotextiles – M-288, latest version available at: <http://www.transportation.org/>

Crown bench: The crown bench is constructed along the outside edge of the fillslope and provides a reinforced zone where flows start their cascade down the rock-armored slope. For added road width, the coarse riprap layer placed in the crown bench can be topped with smaller, sacrificial rock fill designed to be carried away during high flows, but smooth enough to accommodate tire traffic.

Crossing dip: The road grade is dipped across the crossing and is wide and deep enough to accommodate the expected 100-year flood flow and prevent stream diversion. The crossing dip must be sloped to be free-draining to avoid ponding water at the upstream edge of the crossing. If site conditions require a raised fill prism through the crossing that would act as a dam and create standing water upstream of the crossing, then additional considerations should be evaluated, including the stability of the fillslope under saturated soil conditions, soil piping, and the potential to create wetland habitat.

Armored chute: The armored chute conveys flows down the fillslope and ideally extends the full width of the crossing fill. However, constructing the armored chute the full width of the crossing fill can be prohibitively expensive and is not always necessary. Thus, at a minimum, the width of the armored chute must extend below the dipped road the full width of anticipated flood flows or be confined to a trapezoidal chute sized to accommodate the 100-year flood flow.

Fill: Fill is material that is mechanically placed and built up in compacted lifts to establish grade through the crossing and to create a firm, stable road bed. Fill material can be sourced locally or imported and must be free of low-density topsoil and organic material.

Keyway: A keyway is constructed at the toe of the fillslope to anchor the riprap, prevent scour at the transition between the chute and native channel, and reduce the potential of head cutting extending upstream through the crossing.

IX. 5 Suggested Rock-Armored Crossing Construction Techniques

Constructing a rock-armored crossing involves a multi-step process that generally includes:⁵⁰

Step 1: Clearing and removing all organic material from within the limits of the proposed road crossing, including cut and fill slopes.

Step 2: Preparing the slopes to receive fill by constructing either a keyway or bench at the base of the fillslope and subsequent benches as the fillslope progresses upslope.

⁵⁰ Detailed construction methods for rock-armored crossings are also covered in DFW (2006) and Weaver et al. (2015).

Step 3: Constructing the fillslope from the toe upslope in successive 1-foot thick, compacted lifts.

Step 4: Constructing a broad dip in the roadbed, aligned with the original channel, that can accommodate the 100-year flood flow without being diverted.

Step 5: Over-excavating the outside fillslope and constructing a keyway at its toe to receive rock fill. The over-excavation may need to extend to the specified depth of rock riprap such that the finished rock surface will be at the elevation of the surrounding slopes. Otherwise the entire fillslope can be faced with rock riprap to the specified depth.

Step 6: Placing the specified rock riprap within the keyway and extending upslope. The primary rock riprap must be either placed on a filter bed formed from a layer of specified smaller rock or on a layer of geosynthetic filter fabric. Placement of rock riprap should follow immediately after placement of the filter layer. It is good practice to place the larger diameter rock available in the keyway first and build up from there. Be sure, however, to have a homogenous mix of riprap with large-diameter rocks evenly distributed throughout. It is also critical to ensure that the rock in the keyway provides an effective transition from the slope of the crossing to the natural channel.

Step 7: Constructing the rock riprap fill face to have a concave or trapezoidal shape to naturally concentrate and maintain flows centrally within the rock riprap layer.

Step 8: Constructing the upper crown bench, installing the appropriate filter layer, and backfilling with rock riprap. Ensure the finished rock surface blends with the surrounding road grade and does not impede surface flows across the road and down the rock-armored chute/outfall.

Step 9: If necessary, place a layer of smaller rock or crushed aggregate across the running surface of the dip to reduce the generation of sediment on the running surface and to provide a suitable surface to support truck traffic.

The steps outlined above are for fillslope heights of about 15 feet and less that can be constructed with conventional equipment (e.g., excavator with standard length boom and dipper arm) from a position near the final road surface elevation (Figure 27). For crossings with fillslopes greater than about 15 feet in height that exceed the reach of most conventional construction equipment, the fillslope and rock riprap layer shaped to final grade must be constructed somewhat in unison from the toe of the fillslope to the finished road surface elevation.



Figure 27. A rock-armed crossing with well-defined armored outfall looking downslope (upper photo) and upslope (lower photo), located in northeastern California.

X. Permanent Bridge Design

To preserve water quality and the beneficial uses associated with aquatic species, project proponents are increasingly installing temporary and permanent bridge crossing structures, particularly on fish-bearing watercourses (Figure 28). If they are properly designed and constructed, bridges typically provide low overall environmental impact, provide clearance for extreme floods and floating debris, provide year-round vehicle passage, and generally meet stringent fish passage requirements.

It is the RPF's responsibility to determine if the services of professionals with the appropriate expertise, including but not limited to licensed professional engineers or licensed professional geologists, are required, where such expertise is called for by Professional Foresters Law, Public Resources Code (PRC) § 750 et seq., particularly § 758; the Business and Professions Code § 6700 et seq. (Professional Engineers Act); and/or § 7800 et seq. (Geologists and Geophysicists Act). **In accordance with § 6731 of the California Professional Engineers Act, permanent bridge structures are to be engineered by a California Registered Civil Engineer.**

However, in accordance with the Professional Foresters Law (PRC § 750 et sec.), RPFs often identify the location, develop preliminary plans for the crossing based on field conditions, and propose a conceptual crossing design for agency review



Figure 28. Permanent bridge installed in Shasta County. The bridge spans the entire channel and is positioned above the anticipated flood flow elevation.

and comment. In order to achieve a viable design, RPFs should have a working knowledge of fundamental design standards and concepts for permanent bridge crossings.

In this chapter, we expand on several key design elements that should be considered by RPFs to ensure that public safety and impacts to resources are adequately addressed. The key design elements include channel geomorphology, foundation considerations, and hydraulic capacity. The design considering these elements is submitted to CAL FIRE as part of a THP or other type of commercial harvesting plan.

Design drawings showing site topography; control points; dimensions of the bridge structure in plan, elevation, longitudinal profile, and cross-sectional views; and key component details may be required by the reviewing agencies. Additionally, the geomorphic setting of the bridge, the potential for debris jams at the bridge site, and potential scour problems should be considered when designing a bridge or other structure in a watercourse with state and federally listed anadromous salmonids (see the site examination form provided in Appendix E, Part B).

Where necessary, RPFs designing a permanent bridge crossings are encouraged to consult with professionals with expertise in key disciplines, such as engineers, geologists, fisheries and wildlife biologists, hydrologists, botanists, and archeologists.

X.1 Channel Geomorphology and Bridge Siting

As with culverts, the geomorphology of the channel plays a key role in the siting of bridges. For example, a low-energy, incised narrow channel with stable bedrock side slopes would have far fewer design challenges than a braided channel that is prone to scour and lateral migration. Consequently, to reduce costs and to maximize success, the RPF should consider the following questions in selecting the most favorable location to site a bridge crossing (adapted from Groenier and Gubernick 1989):

- **Where is the crossing location in the watershed, and how does the stream transport water, sediment, and wood at that location?**
Locate bridges where the stream channel is narrow, straight, has a uniform profile, and unobstructed flow. Avoid constricting the natural channel or overflow channels with the placement of abutment foundations or mid-stream foundations (bents or piles). Consider upstream landslide potential and large woody debris recruitment and transport potential that could impact the structure.
- **How is the channel configured?**
 - **What is the degree of channel confinement?**
 - **Is there floodplain conveyance? If so, how much, and are there multiple channels, side channels, or backwater alcoves?**
 - **Does the channel contain evidence that the stream may migrate (move laterally) and affect the foundation system?**

- **What is the range of vertical fluctuation (either due to aggradation or degradation) of the streambed?**

Locate bridges to avoid problem areas such as deltas, alluvial fans, actively aggrading or degrading sections, sharp curves, multi-threaded channels, wetlands, and floodplains. Bed and bank soil properties should be evaluated to determine whether they present unique characteristics that will influence the long-term stability of the channel and the structure. For example, well-armored (coarse-grained cobble and boulder dominated), moderate (5-10%) gradient channels (i.e., transport reaches) are typically more rigid and less prone to large scale aggradation and degradation compared to less-armored (fine-grained silts, sands, and gravels), low-gradient (<3%) channels (i.e., response reaches) found in valley bottoms.

How well are the road and bridge aligned with the stream channel?

Where possible, locate bridges perpendicular to the channel to minimize the required span and to reduce ground disturbance adjacent to the watercourse. Place the bridge deck horizontal (sloping decks can present a safety hazard due to the increased sliding potential and loads imposed on the structure). Place the bridge deck slightly higher in elevation compared to the adjacent road approaches such that approaches on either side of the bridge direct drainage away from the watercourse and bridge deck. Avoid road alignments that would require a curved bridge deck.

- **Is the channel stable, or is it adjusting to recent large-scale disturbances?**

Assess the channel for presence of potential headcuts that can migrate upstream and undermine foundations and for aggradation that may affect channel capacity at the site. Longitudinal profiles (see Bridge Crossing Hydraulics below) are particularly useful in determining the presence of a significant headcut.

- **Are there special site-specific conditions or construction limitations that should be considered?**

Factors such as equipment access, ability to excavate, clearing limits, erosion control measures, and water management requirements should be considered as part of the bridge layout. As with other types of crossings, in many cases State and Federal agencies, including DFW, RWQCBs, USACE, and NOAA, require spill prevention and erosion control measures, temporary crossing provisions, and stream diversion methods through or around the construction site to be addressed as part of proposed bridge crossing designs.

Groenier and Gubernick (1989), Barnard et al. (2013), USACE (1994), B.C. Ministry of Forests (2002), Flosi et al. (2010), and Richardson et al. (2001) provide additional information on these questions and other design considerations related to channel geomorphology and bridge siting criteria.

X.2 Foundation Considerations

Foundations used to support the superstructure of a bridge should be evaluated by an appropriately licensed professional working within their area of expertise. Although RPFs may not be ultimately responsible for the design of the foundation system, at a minimum they typically perform a site reconnaissance and provide input on the preliminary design and layout of the foundation. To minimize the need for complex and expensive foundation designs, such as deep foundations (e.g., driven piles or drilled shafts) or tall retaining walls, the following list provides a few general concepts that should be considered when conducting a reconnaissance of the bridge site and foundation layout.

- **Where possible, the bridge should span the entire width of the channel above the anticipated 100-year flood flow elevation (see discussion below on Bridge Hydraulics) and the foundation elements should not encroach into the 100-year floodplain.** Foundation elements that encroach into the 100-year floodplain can affect the hydraulic capacity of the structure, can generate areas of concentrated scour (e.g., contraction scour), and can limit passage of floating debris.
- **Footings should be founded on firm soil or bedrock that is capable of supporting the anticipated loads.** Expansive soils (e.g., clay-rich soil) and weak or collapsible soils (e.g., organic-rich soil or loose, low-density silts and sands) should be avoided. As mentioned above, avoid deep foundations where possible. However, poor site conditions may necessitate deep foundations such as piles, drilled shafts, or caissons. Costs of the substructure and foundation work can be a large part of the overall cost of a bridge.
- **Foundations that function as tall retaining structures should be avoided.** When there is a difference in elevation of the backfill on either side of the foundation, then unbalanced lateral earth pressures can develop and the foundation acts as a retaining structure. To minimize design and detailing requirements for retaining structures, foundations (e.g., footings and/or stem walls) that have more than about 4 feet of unbalanced fill height from the front to the back of the foundation should be avoided without detailed engineering design (see Figure 29).

Avoid placing shallow footings on or adjacent to descending slopes steeper than 3h:1v (units horizontal, h, to units vertical, v), or about 33 percent. Foundations on or adjacent to descending slopes should be founded in suitable material with an adequate embedment depth and set back from the descending slope surface sufficient to provide vertical and lateral support for the foundation. Where the slope is steeper than 1h:1v, or about 100 percent (45 degrees), the required setback should be measured from an imaginary plane which is a maximum 45 degrees to the horizontal, projected upward from the toe of the slope (see Figure 30). Alternate setbacks and clearances from those shown in Figure 30 may be applied based on soil and/or rock competency, and may require input from an appropriately licensed professional. In some cases,

revetment may be necessary along the toe of the slope to prevent scour and ensure the long-term stability of the slopes supporting the foundation.

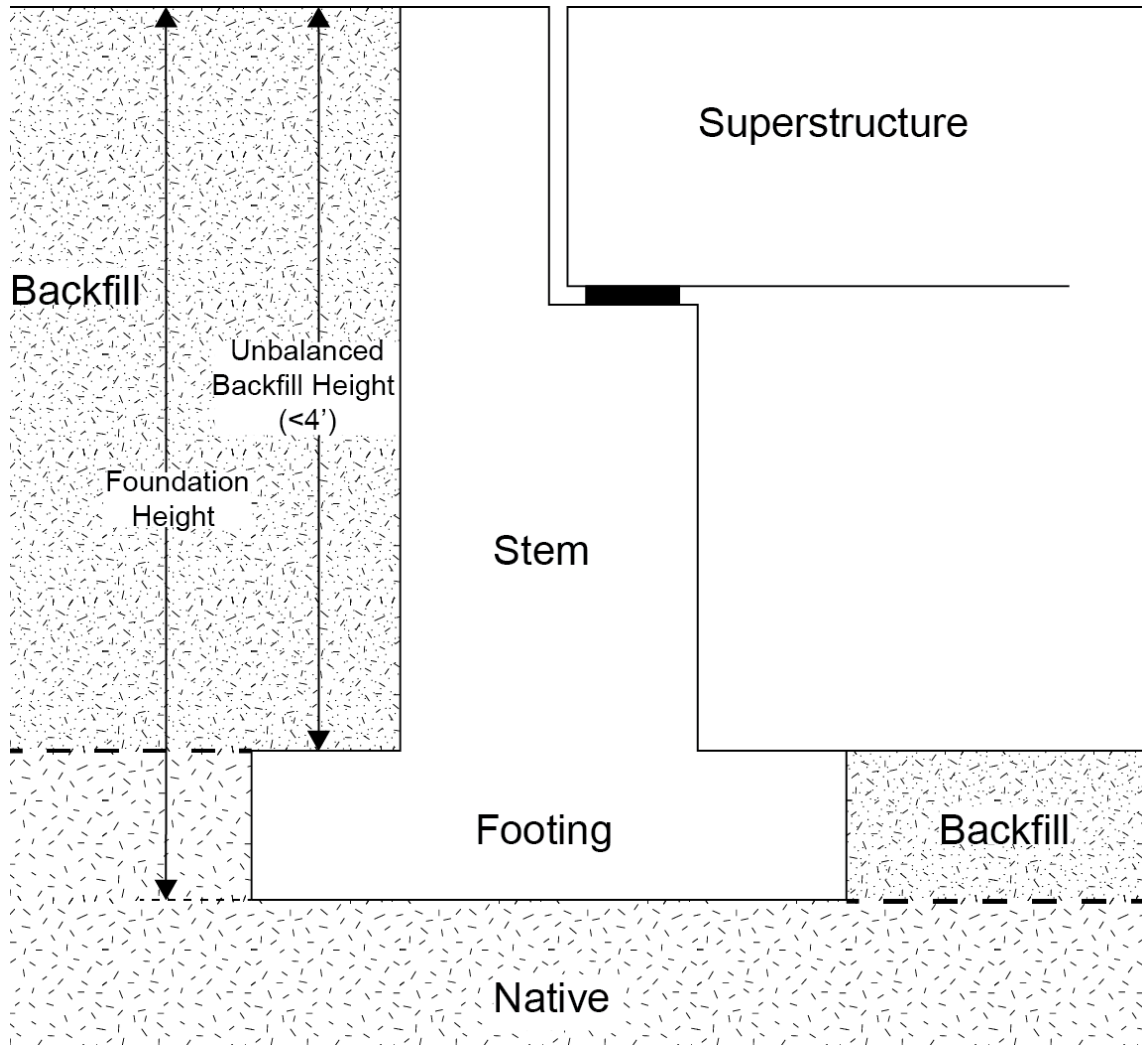


Figure 29. Bridge foundation showing unbalanced fill height.

Where retaining wall abutments cannot be avoided, there are a number of economical abutment approaches that design professionals may consider, including mechanically stabilized earth (MSE) walls (Berg et al. 2009), segmental concrete block and pre-cast cantilevered walls (www.redi-rockblock.com and www.contech.com, respectively), and geosynthetic reinforced soil (GRS) integrated bridge systems (Figure 31) (Adams et al. 2011). These types of retaining wall abutments benefit by being relatively quick and easy to install, and do not generally require specialized equipment to construct.

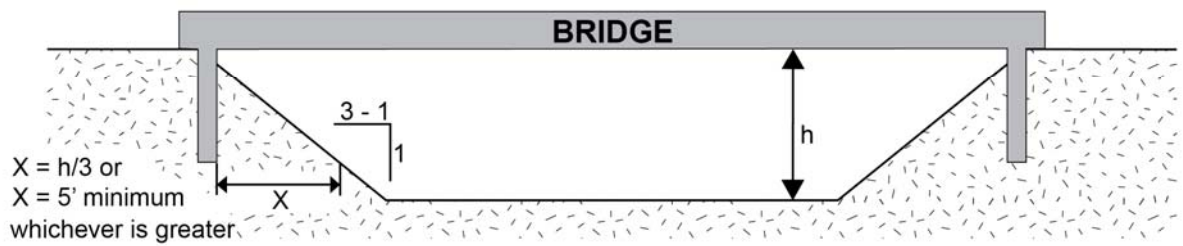
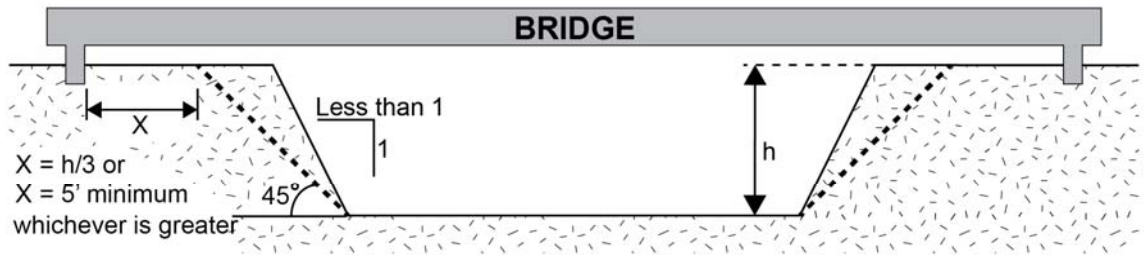
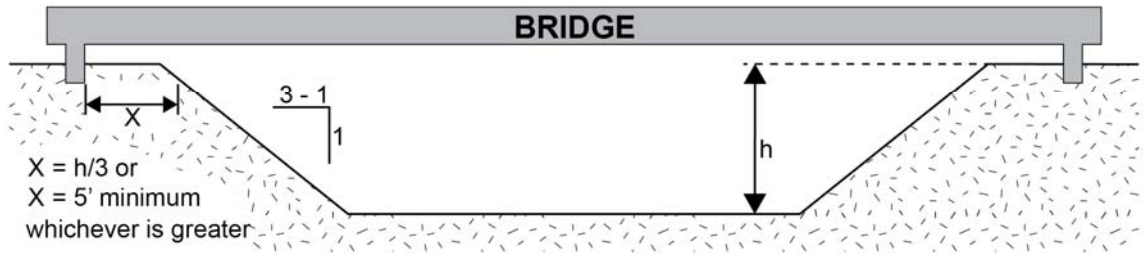


Figure 30. Example of foundation setback limits (adapted from CBC 2013).



Figure 31. GRS bridge abutments founded above the Q_{100} water surface elevation, Mendocino County (photo provided by Gordon Keller, USFS (retired)).

X.3 Bridge Crossing Hydraulics (100-year flood flows)

In accordance with FPR 14 CCR § 923.9 [942.9, 963.9] (f), all permanent watercourse crossings that are constructed or reconstructed shall accommodate the estimated 100-year flood flow, including debris and sediment loads. To allow for the passage of floating debris, it is recommended that a minimum of 3 feet of free board measured from the anticipated 100-year flood flow elevation to the bottom of the bridge superstructure be applied (ODF 2002b, Wenger 1984, WADNR 2013, Barnard et al. 2013). Alternately, high-risk bridges can be constructed with armored overflow areas near the structure if necessary (Keller and Ketcheson 2015). The following discussion introduces a simple, multi-step process to size a bridge crossing to pass the 100-year flood flow, including debris and sediment loads. In bridge design, it is always preferable to keep the bridge and its abutments out of the 100-year flood zone, and to avoid altering the natural channel bed and banks beneath the bridge.

Step 1: Survey a longitudinal profile of the channel extending upstream and downstream of the crossing. The scale, or length, of the longitudinal profile surveyed should reflect the scale of the watercourse, the channel geomorphology, and the crossing characteristics. In many cases the length of the longitudinal profile should extend upstream and downstream of the crossing approximately 10 to 30 times the

active channel width. The longitudinal profile should be surveyed along the thalweg of the channel and can be conducted using simple techniques (e.g., hand level, tape, and stadia rod). Methods to survey longitudinal profiles are detailed in Harrelson et al. (1994).

Information obtained from the longitudinal profile includes the average channel slope, roughness characteristics based on the channel substrate, channel morphology, and upper and lower limits for vertical channel adjustment (e.g., limits of channel aggradation and degradation). The roughness of the channel and its floodplain can affect streamflow by influencing water turbulence; the rougher the channel, the more resistance there is to water flow. The roughness of a channel is expressed using Manning's roughness coefficient, n . Suggested values of Manning's n for mountain stream channels can range from 0.04 to >0.07 . Roughness characteristics of natural channels are given by Barnes (1967), who presents photographs and cross-sections of typical rivers and creeks and their respective n values. A guide by Arcement and Schneider (1989) presents step-by-step procedures for selecting Manning's n values for natural channels and floodplains. Chow (1959) provides formulas to calculate a "composite" Manning's n value for the entire cross-section using different roughness values for different sections of the cross-section wetted perimeter. Appendix E, Part A provides a list of accepted Manning's n values based on stream type.

Step 2: Survey and draw a scaled cross-section perpendicular to the channel through the centerline of the proposed bridge crossing. The cross-section should extend the full width of the channel, and at a minimum include the existing and proposed channel surfaces, proposed foundation (e.g., abutment fills, walls, footings, etc.), deck components, and road approaches. Methods to survey a channel cross-section are detailed in Harrelson et al. (1994).

Step 3: On the scaled cross-section drawing, project a horizontal line 3 feet below the bottom surface of the proposed bridge deck. This line represents the water surface elevation under the bridge that would maintain a minimum of 3 feet of freeboard for woody debris passage (freeboard may need to be increased in larger watercourses that could pass larger logs). Next measure the distance, in feet, along the channel surface that is below the projected horizontal line. This measurement represents the "wetted perimeter", or WP.

Now calculate the wetted cross-section area of the channel below the horizontal line, being sure to account for the area occupied by riprap, if it is to be applied as revetment on the channel banks beneath the bridge. If the channel cross-section has a common geometric shape such as a rectangle, trapezoid, or a semi-circle, then it is possible to calculate the cross-section area and wetted perimeter using common equations found in most hydraulics text books and on line at: http://www.engineeringtoolbox.com/flow-section-channels-d_965.html. However, if the channel cross-section is irregular in shape, as most natural stream channels are, then the cross-section area can be approximated. A simple method to approximate the cross-section area is to divide the cross-section into unique segments, then sum the product of the width, w , and average depth, d , for each segment (see Figure 32 for an example).

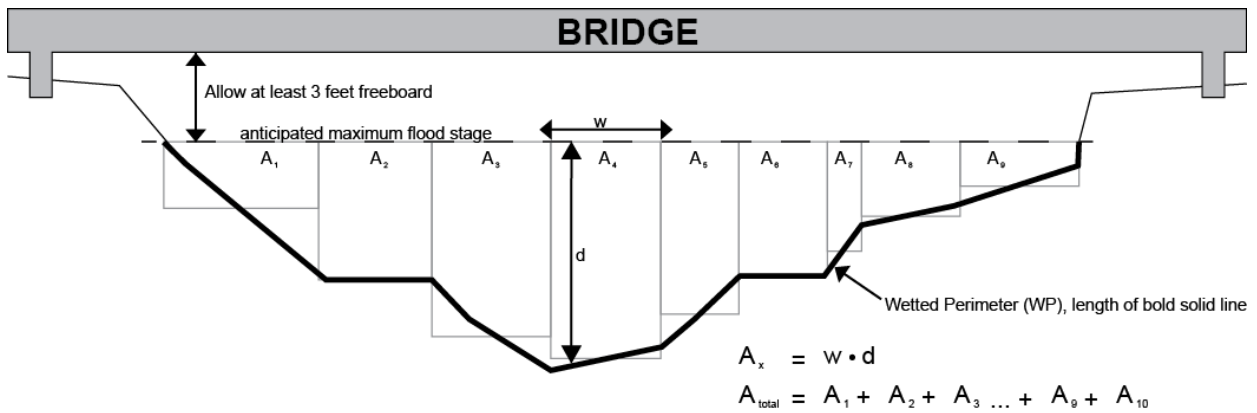


Figure 32. Cross-section beneath a bridge showing the wetted perimeter and an example area calculation.

Step 4: Calculate the average water velocity of the channel beneath the bridge.

Knowing the average channel slope, S , and Manning's roughness coefficient, n , obtained from the longitudinal profile (Step 1), the wetted perimeter, WP , and the cross-section area, A , of the channel beneath the proposed structure obtained from the cross-section survey and scaled drawing (Steps 2 and 3), the next step is to calculate the average water velocity of the channel below the bridge using Manning's Equation:

$$v = \left(\frac{1.49}{n}\right) \left(\frac{A}{WP}\right)^{2/3} S^{1/2} \quad \text{Eq. 1}$$

Where:

- v = average flow velocity (ft/s)
- n = Manning's roughness coefficient
- S = channel slope (e.g., slope of energy grade line) (ft/ft)
- A = wetted cross-section area (ft²)
- WP = wetted hydraulic perimeter (ft)

Step 5: Calculate the hydraulic capacity of the bridge. The hydraulic capacity of the bridge (Q) is the product of the mean channel velocity (v) calculated using Manning's Equation (Step 4) and the channel cross-sectional area (A) calculated in Step 3:

$$Q = vA \quad \text{Eq. 2}$$

Where:

- Q = flow (cfs)
- v = average flow velocity (ft/sec)
- A = wetted cross-section area (ft²)

Step 6: Compare the hydraulic capacity of the bridge to the estimated 100-year flood flow. Provided the hydraulic capacity of the bridge (Q) (Step 5) is greater than the estimated 100-year flood flow (calculated previously, refer to Chapter 2), the crossing is adequately sized. However, if the hydraulic capacity of the bridge is less than the estimated 100-year flood flow, then the crossing may be inadequately sized and an alternative design should be proposed or the existing design justified as being appropriate. Examples justifying an inadequately-sized crossing (i.e., where Q of the bridge is less than 100-year flood flow) may include designing the structure to be overtopped (Figure 33), or demonstrating that alternative flow paths around the structure and its abutments exist that would allow flood flows to pass without substantially damaging the bridge, its foundation system, or surrounding resources. Dipped or vented (culverted) approach fills have commonly been used where there is a floodplain to provide an alternate flow path around the bridge (see Chapter V, Section V.1).

Alternatively, the elevation of the 100-year flood flow at the crossing can be approximated by performing a step-wise or iterative process of changing the location of the horizontal line in Step 3 and performing the hydraulic calculations outlined in Step 4 and Step 5 until the hydraulic capacity (Q) matches the anticipated 100-year flood flow. Knowing the hypothetical elevation of the 100-year flood flow is particularly useful in locating the proposed foundation elements outside the 100-year flood zone and determining the elevation to install revetment used to provide scour protection along the banks of the watercourse.



Figure 33. Heavily reinforced concrete bridge with sacrificial wood railings that is designed to be overtopped in the event of a flood event, located in Santa Cruz County.

X.4 Bridge Plan Detailing

The RPF should provide sufficient information in adequate detail for the proposed crossing in the plan to allow the public and the State Review Team agencies to determine the appropriateness of the proposed design and its potential to maintain downstream beneficial uses. The following list provides specific details to consider. Depending on the site conditions, this list may be modified as appropriate. Appendix E, Part B provides a generalized form that can be used as a checklist to ensure the basic information needed for preliminary site assessment is collected and provided in the design.

- **Watercourse classification.**
- **Estimated 100-year flood flow calculated for the site (hydrology).**
- **Hydraulic capacity of the bridge, including the input parameters used to calculate the hydraulic capacity.**
- **Relative site plan and profile (longitudinal and cross-section) drawings drawn to scale showing current conditions (e.g., existing crossing structures and aggraded materials), proposed conditions (e.g., proposed slope gradients and grade control structures), estimated 100-year flood flow elevation, and the proposed foundation and superstructure elements relative to the channel.**
- **Description of the channel geomorphology, including bed and bank substrate, channel stability, and vegetative cover.**
- **Description of foundation soil conditions.**
- **Grade control structures, if necessary (including their impact on fish passage).**
- **Material descriptions and construction specifications, if required.**
- **Details of debris passage or management strategies, if required.**
- **Special provisions, such as dewatering plans, seismic design, equipment restrictions, and erosion control measures, that may be required as part of separate State and Federal agency permits and agreements.**⁵¹

Appendix E, Part C provides an example of a hypothetical bridge design from start to finish, with the necessary information provided, detailed, and presented in a form that would be appropriate for public and agency review.

⁵¹ Note that bridges on privately owned roads may require building permits that are consistent with the applicable building codes for a given location.

XI. Conclusions

Several office techniques based on flood flow measurement records and empirical relationships between precipitation, watershed characteristics, and runoff are available to determine an estimated 100-year flood flow. However, any office-based results should be checked against field observations. For instance, if office-based equations indicate that a 24-inch culvert would pass the 100-year flood flow but the bankfull cross-section is more than one square foot in coastal northwestern California, the culvert may be too small for stream discharge. Wood and sediment passage requirements would likely further increase initial estimates of pipe diameter. Anticipated increases in flow, sediment, and debris must be considered in post-fire situations. Incorporating climate change into the design of watercourse crossings may also be necessary in the future (Wilhere et al. 2016).

Culvert sizes specified as part of a permitted project in California, such as a THP or other type of harvesting plan, should be based on defensible, accepted methods, such as those discussed above, with documentation for the input values, appropriate maps, data sources, field observations, and calculations.⁵² A spreadsheet is available for calculating discharges using the Rational Method and the updated USGS Magnitude and Frequency Method, which can be cited in the plan (Figure 34).⁵³ This level of information assists both agency review of plans and reduces the need for follow-up questions.

While culvert installation still occurs very frequently in California's forested watersheds, rock fords, rock-armored crossings, bridges, and open-bottomed arch installations have become more common in the past 15 years. These types of permanent crossings are excellent alternatives to culverts, and should be used where appropriate based on the site-specific conditions present. These may include fish passage requirements, winter maintenance issues, the presence of large amounts of mobile wood, and crossing sites in landslide prone terrain. Design of these types of permanent crossings must be based on sound engineering principles, involve licensed professionals where necessary, and take into consideration both the risk of failure and the potential downstream impacts. In many situations, a temporary crossing is preferable to installing a permanent structure.

While we cannot completely avoid watercourse crossing failures, we can reduce failure potential through careful crossing design that accommodates water, wood, and sediment, and that reduces potential erosional consequences if they do fail.

⁵² California FPR 14 CCR § 923.9 [943.9, 963.9] (e) specifies that the minimum diameter and the method(s) used to determine the culvert diameter must be specified in the plan.

⁵³ Moore et al. (1999) provide a review of software tools available for culvert design and analysis.

Location: THP

(Enter data in fields with red-colored headings. Other data fields will be calculated automatically.)

Magnitude and Frequency Method for 100-year flood flow (A > 50 acres)

No.	Crossing	Area (acres) A	Basin maximum elevation (ft)*	Crossing elevation (ft)*	Area (mi ²) A	Avg. Annual Precipitation (in/yr) P	Average Basin Elevation H	100-yr flood flow Q ₁₀₀ (cfs)			
								North Coast ⁽¹⁾ (NC)	Sierra ⁽²⁾ (S)	North-east ⁽³⁾ (NE)	Central Coast ⁽⁴⁾ (CC)
1					0.000		0	0.0	#DIV/0!	0.0	0.0
2					0.000		0	0.0	#DIV/0!	0.0	0.0
3					0.000		0	0.0	#DIV/0!	0.0	0.0
4					0.000		0	0.0	#DIV/0!	0.0	0.0
5					0.000		0	0.0	#DIV/0!	0.0	0.0
6					0.000		0	0.0	#DIV/0!	0.0	0.0
7					0.000		0	0.0	#DIV/0!	0.0	0.0
8					0.000		0	0.0	#DIV/0!	0.0	0.0
9					0.000		0	0.0	#DIV/0!	0.0	0.0
10					0.000		0	0.0	#DIV/0!	0.0	0.0

See below for M&F equations

Rational Method for 100-year flood flow (A < 200 acres, best < 100 acres)

No.	Crossing	T _c = 60((11.9 X L ³)/H) ^{0.385}			Q ₁₀₀ = CIA				Magnitude & Frequency Q ₁₀₀ equations
		Channel length (to top of basin) (mi) L	Elevation difference (ft) H	Concentration time (min) T _c	Runoff coefficient C	100-year Return-Period Precipitation (in/hr) I*	Area (acres) A	100-yr flood flow (cfs) Q ₁₀₀	
1			0	#DIV/0!			0	0.0	NC (1) Q ₁₀₀ = 48.5(A) ^{0.866} (P) ^{0.558} S (2) Q ₁₀₀ = 20.6(A) ^{0.874} (P) ^{1.24} (H) ^{0.250} NE (3) Q ₁₀₀ = 0.713(A) ^{0.731} (P) ^{1.56} CC (4) Q ₁₀₀ = 11.0(A) ^{0.84} (P) ^{0.994}
2			0	#DIV/0!			0	0.0	
3			0	#DIV/0!			0	0.0	
4			0	#DIV/0!			0	0.0	
5			0	#DIV/0!			0	0.0	
6			0	#DIV/0!			0	0.0	
7			0	#DIV/0!			0	0.0	
8			0	#DIV/0!			0	0.0	
9			0	#DIV/0!			0	0.0	
10			0	#DIV/0!			0	0.0	

*Use 100-yr precipitation of duration similar to T_c or for 10 min, whichever is larger, convert to in/hr for input as "I"

Figure 34. Spreadsheet available for determining estimated water discharge associated with a 100-year recurrence interval flood event using either the Rational Method or the updated USGS Magnitude and Frequency Method. Created by Dr. Michael Wopat in 2003; updated in 2014. Available at: http://calfire.ca.gov/resource_mgt/resource_mgt_forestpractice_pubsmemos_pubs.php

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Rational Method

Known Information:

Drainage area (A) = 96 acres (Henry 1998) for HEN
100-yr 10 minute NF Caspar Creek rainfall = 0.698 inches (NOAA website)
100-yr 30 minute NF Caspar Creek rainfall = 1.16 inches (NOAA website)
100-yr 60 minute NF Caspar Creek rainfall = 1.64 inches (NOAA website)
Channel length = 0.5 miles from the ridge to the gaging station
Hillslope length = 400 feet
Average basin slope = 21%
Difference in elevation = 550 feet from the ridge to the gaging station
Soil type = loam
Subsurface flow/saturation overland flow rate = 0.1 ft/sec (Dunne and Leopold 1978)
Channel flow rate = 6 ft/sec (Dunne and Leopold 1978)

Calculate:

$$Q_{100} = CIA$$

Time of Concentration (using the California Culvert Practice or modified Kirpich Equation):

$$T_c = \left(\frac{11.9(L)^3}{H} \right)^{0.385}$$

where:

T_c = time of concentration (hours)

L = length of the channel in miles from the head of the watershed to the crossing point

H = elevation difference between the highest point in the watershed and the crossing point (feet)

$$T_c = [(11.9 (0.5 \text{ miles})^3 / 550 \text{ feet})]^{0.385}$$

$$T_c = 0.103 \text{ hours or 6 minutes}$$

T_c = 6 minutes. Use 10-minute rainfall-depth-duration-frequency data from the NOAA website
0.698 inches/10 minutes x 60 minutes/hour = 4.19 inches/hour

$$I = 4.19 \text{ inches/hour}$$

$$C = 0.3 \text{ (loam soil, Table 1)}$$

$$Q_{100} = 0.3 \times 4.19 \text{ inches/hour} \times 96 \text{ acres}$$

$$Q_{100} = 120.7 \text{ or } 121 \text{ cfs}$$

Pipe diameter = 62 inches (assumes HW/D = 1.0 and projecting pipe)

Pipe diameter = 80 inches (assumes HW/D = 0.67 and projecting pipe)

Time of Concentration (using the Airport Drainage Formula):

$$T_c = ((1.8) (1.1 - C) (D^{0.5})) / (S^{0.33})$$

where:

T_c = time of concentration in minutes

C = runoff coefficient (dimensionless, $0 < C < 1.0$)

D = distance in feet from the point of interest to the point in the watershed from which the time of flow is the greatest

S = slope in percent

$$T_c = ((1.8) (1.1 - 0.3) (2640^{0.5})) / (21^{0.33})$$

$$T_c = 27 \text{ minutes, or approximately 30 minutes}$$

$$1.16 \text{ inches/30 minutes} \times 60 \text{ minutes/hour} = 2.32 \text{ inches/hour}$$

$$I = 2.32 \text{ inches/hour}$$

$$C = 0.3 \text{ (loam soil, Table 1)}$$

$$Q_{100} = 0.3 \times 2.32 \text{ inches/hour} \times 96 \text{ acres}$$

$$Q_{100} = 66.8 \text{ or } 67 \text{ cfs}$$

Pipe diameter = 50 inches (assumes HW/D = 1.0 and projecting pipe)

Pipe diameter = 64 inches (assumes HW/D = 0.67 and projecting pipe)

Time of Concentration (using the Estimated Travel Times Method—modified from Cafferata and Reid 2013):

$$T_c = (HL/V_1) + (CL/V_2)$$

where:

T_c = time of concentration in seconds

HL= hillslope length

CL= channel length

V_1 = subsurface flow/saturation overland flow rate (ft/sec)

V_2 = channel flow rate (ft/sec)

$$T_c = 400 \text{ ft}/0.1 \text{ ft/sec} + 2400 \text{ ft}/6 \text{ ft/sec}$$

$$T_c = 4400 \text{ seconds or } 73 \text{ minutes (use } T_c = 1 \text{ hr)}$$

$$I = 1.64 \text{ inches/hr (using the NOAA website data)}$$

$$C = 0.3 \text{ (loam soil, Table 1)}$$

$$Q_{100} = 0.3 \times 1.64 \text{ inches/hour} \times 96 \text{ acres}$$

$$Q_{100} = 47.2 \text{ or } 47 \text{ cfs}$$

$$\text{Pipe diameter} = \underline{42 \text{ inches}} \text{ (assumes HW/D} = 1.0 \text{ and projecting pipe)}$$

$$\text{Pipe diameter} = \underline{54 \text{ inches}} \text{ (assumes HW/D} = 0.67 \text{ and projecting pipe)}$$

Time of Concentration (using the BC Empirical Chart—BC Government 1991):

T_c = value determined from Figure 1, Chapter 7 in BC Government 1991 document (plot of T_c in hours vs square root of drainage area (km^2); use curve for steep slope, since average basin slope $\geq 10\%$)

$$T_c = 0.72 \text{ hours or } 43 \text{ minutes (use } T_c = 1 \text{ hr)}$$

$$I = 1.64 \text{ inches/hr (using the NOAA website data)}$$

$$C = 0.3 \text{ (loam soil, Table 1)}$$

$$Q_{100} = 0.3 \times 1.64 \text{ inches/hour} \times 96 \text{ acres}$$

$$Q_{100} = 47.2 \text{ or } 47 \text{ cfs}$$

$$\text{Pipe diameter} = \underline{42 \text{ inches}} \text{ (assumes HW/D} = 1.0 \text{ and projecting pipe)}$$

$$\text{Pipe diameter} = \underline{54 \text{ inches}} \text{ (assumes HW/D} = 0.67 \text{ and projecting pipe)}$$

Updated USGS Magnitude and Frequency Method⁵⁵

Known Information:

$$A = 0.15 \text{ miles}^2$$

$$P = 46.85 \text{ inches/year (Henry 1998)}$$

Calculate:

$$Q_{100} = 48.5 A^{0.866} P^{0.556}$$

$$Q_{100} = 48.5 (0.15)^{0.866} (46.85)^{0.556}$$

$$Q_{100} = 79.6 \text{ or } 80 \text{ cfs}$$

$$\text{Pipe diameter} = \underline{53 \text{ inches}} \text{ (assumes HW/D} = 1.0 \text{ and projecting pipe)}$$

$$\text{Pipe diameter} = \underline{67 \text{ inches}} \text{ (assumes HW/D} = 0.67 \text{ and projecting pipe)}$$

⁵⁵ The USGS National Streamflow Statistics Program (NSS, Version 6, available at <http://water.usgs.gov/software/NSS/>) uses the updated USGS Magnitude and Frequency regional regression equations to estimate flood flows in California. NSS shows there to be a standard error of prediction (SE) of 44% (= 35 cfs) for the 80 cfs Q_{100} estimate. $Q_{100} \pm 1 \text{ SE} = 80 \text{ cfs} \pm 35 \text{ cfs}$, resulting in a ± 1 -SE range of 45 cfs to 115 cfs. Because the range ± 1 SE encompasses the central 68 percent of the range of the estimated discharge, there is a 68 percent chance that the true Q_{100} lies within the range defined by $Q_{100} \pm 1 \text{ SE}$, that is, between 45 cfs and 115 cfs. Additionally, the NSS output reveals that the 90% prediction intervals are 39 cfs and 164 cfs.

Flow Transference Method (Waananen and Crippen 1977)

Known Information:

A = 96 acres (Henry 1998) for HEN; 1168 acres for the North Fork
Q_{100g} for NF Caspar Creek is 352.3 cfs (using USGS PeakFQ program)

Calculate:

$$Q_{100u} = Q_{100g} (A_u/A_g)^b$$

$$Q_{100u} = 352.3 \text{ cfs } (96 \text{ acres}/1168 \text{ acres})^{0.87}$$

$$Q_{100u} = 40 \text{ cfs}$$

Pipe diameter = 40 inches (assumes HW/D = 1.0 and projecting pipe)

Pipe diameter = 52 inches (assumes HW/D = 0.67 and projecting pipe)

Direct Flow Transference Method (Skaugset and Pyles 1991)

Known Information:

A = 96 acres (Henry 1998) for HEN; 1168 acres for the North Fork
Q_{100g} for NF Caspar Creek is 352.3 cfs

Calculate:

$$Q_{100u} = Q_{100g} (A_u/A_g)$$

$$Q_{100u} = 352.3 \text{ cfs } (96 \text{ acres}/1168 \text{ acres})$$

$$Q_{100u} = 29 \text{ cfs}$$

Pipe diameter = 34 inches (assumes HW/D = 1.0 and projecting pipe)

Pipe diameter = 45 inches (assumes HW/D = 0.67 and projecting pipe) (see Figure A-5 for an example of using the culvert sizing nomograph for a discharge of 30 cfs)

3 X Bankfull Area Method (see Figure A-4)

Known Information (based on measurements made at 3 cross-sections):

Average channel depth at HEN is 0.95 feet
Average bankfull stream channel width (W₁) at HEN is 5.6 feet
Average active stream channel width (W₂) at HEN is 4.4 feet
Combined average stream channel width at HEN is 5.0 feet
Bankfull cross-sectional area above HEN is 4.75 feet²

Calculate:

$$D \approx 2[(bfa)^{1/2}]$$

$$D = 2[(4.75 \text{ feet}^2)^{1/2}]$$

$$\text{Pipe diameter (D)} = 4.35 \text{ feet} \times 12 = \underline{52 \text{ inches}}$$

Active Channel Width Method

Known Information:

Average channel width above HEN is 4.4 feet (use W₂ width)

Calculate:

$$\text{culvert diameter/channel width} = 1.0$$

$$\text{culvert diameter} = 1.0 \times \text{channel width}$$

$$\text{culvert diameter} = 1.0 \times 4.4 \text{ feet}$$

$$\text{Pipe diameter (D)} = 4.4 \text{ feet or } \underline{53 \text{ inches}}$$

Flow Frequency Analysis Method

Known Information:

Table A-1. Annual peak discharges for station HEN for water years 1986 through 2013.

Year	Peak Q (cfs)	Rank
1986	12.7	7
1987	4.2	23
1988	7.6	16
1989	4.9	20
1990	12.2	8
1991	1.6	28
1992	4.3	22
1993	17.0	2
1994	4.0	24
1995	14.7	5
1996	11.4	9
1997	15.6	4
1998	13.0	6
1999	16.5	3
2000	6.6	18
2001	5.8	19
2002	10.0	13
2003	11.3	10
2004	10.0	13
2005	3.5	25
2006	18.9	1
2007	2.9	26
2008	11.0	11
2009	2.5	27
2010	7.4	17
2011	6.6	18
2012	8.8	14
2013	4.8	21
2014	3.5	25
2015	7.8	15
2016	10.8	12

Calculate:

Table A-2. Estimated discharges for various recurrence intervals (RIs), including the 10-year RI discharge (used in Part B) and the 100-year flood flow (discharges estimated by the USGS PeakFQ program).

RI (yr)	Q (cfs)	95% Confidence Limits	
		Lower	Upper
2	7.8	6.4	9.4
5	12.7	10.4	16.3
10	16.0	12.9	21.5
25	20.2	15.8	28.4
50	23.2	17.8	33.6
100	26.1	19.7	38.8

$Q_{100} = 26 \text{ cfs}$
 Pipe diameter = 33 inches (assumes HW/D = 1.0 and projecting pipe)
 Pipe diameter = 43 inches (assumes HW/D = 0.67 and projecting pipe)

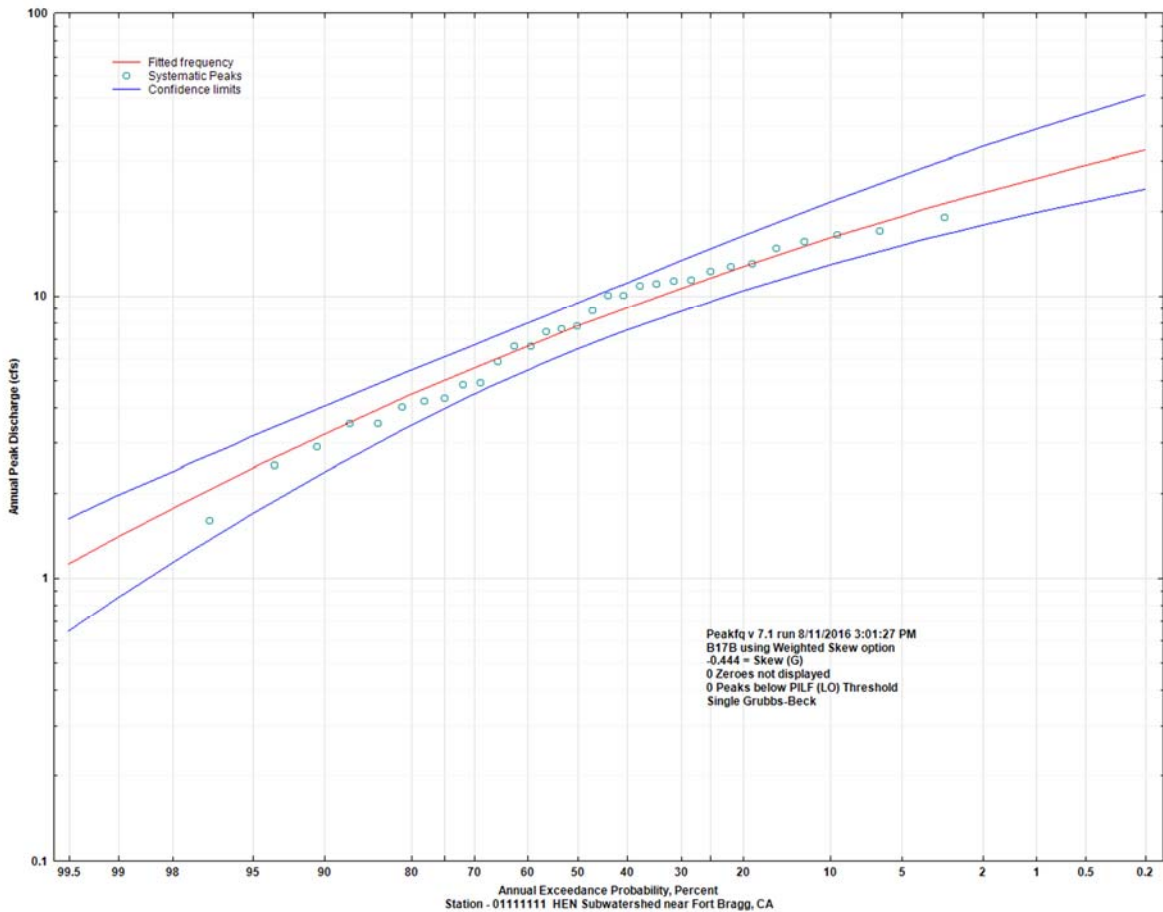


Figure A-3. USGS PeakFQ plot of annual exceedance probability vs. annual peak discharge for North Fork Caspar Creek subwatershed HEN.⁵⁶

The Rational Method is recommended for basins less than 100 acres, while the USGS Magnitude and Frequency Method is preferred over the Rational Method for drainage areas larger than 25-100 acres, depending on the region being considered (25 acres for the North Coast region). The direct flow transference method is preferred over both of these methods for HEN, however, since (1) there are 53 years of discharge data for the downstream North Fork Caspar Creek gaging station available (Tables A-1 and A-2, Figure A-3), (2) the subwatershed is within approximately one order of magnitude in size of the North Fork station (<2,500 ac), and (3) local data are more likely to represent the drainage-basin characteristics in terms of slopes, geology, soils, and climate than the more general regional equations or empirical relationships. We utilized the direct flow transference method with a HW/D ratio of 0.67, since there is considerable mobile wood in this channel, and the 3 X bankfull area method as a field check to determine the best estimate of required pipe diameter for a hypothetical crossing at the bottom of the HEN watershed. The active channel width method was used to allow for wood passage. Based on

⁵⁶ Note that recurrence interval (RI) in years is the inverse of annual exceedance probability (p), where $RI = 1/p$, and p has a value between 0 and 1. For example, a discharge with a 0.01 annual exceedance probability (or 1 percent) has a recurrence interval of 100 years.

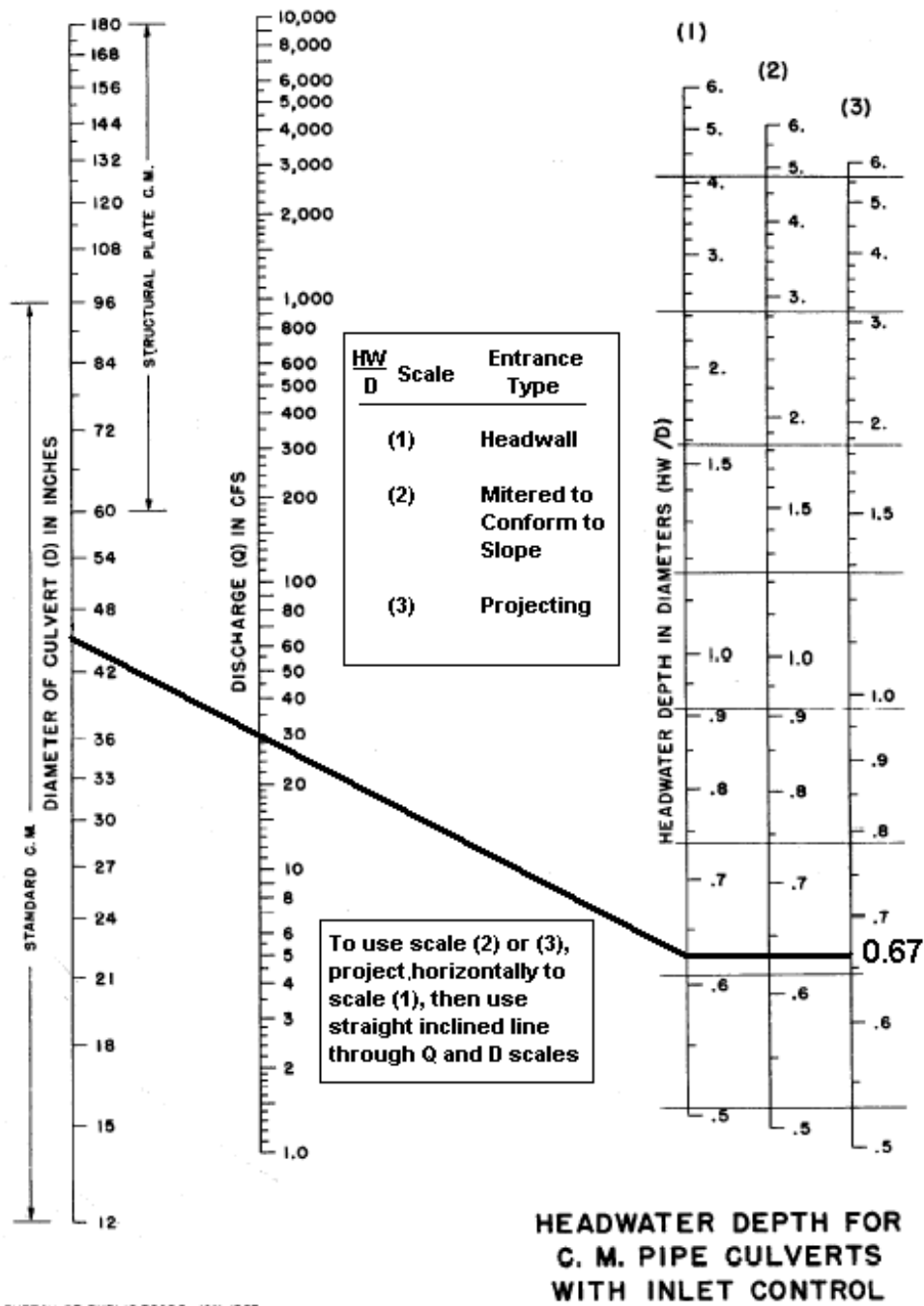
the results from these office methods and the field cross-sectional measurements, we recommend the selection of a **54 inch CMP (rounds to 60 inches, a size easily available) (Table A-3)**. The flow frequency analysis confirms that this is a reasonable estimate for this small watershed.

Table A-3. Summary of the results using all the crossing sizing methods for determining the 100-year flood flow and pipe diameters for subwatershed HEN.

Method	Predicted 100-Year Recurrence Interval Flood Flow (cfs)	Pipe Diameter—assuming HW/D Ratio = 0.67 for office-based methods (in)
Rational—CA Culvert Practice (modified Kirpich)	121	80
Rational—Airport Drainage	67	64
Rational—Estimated Travel Times	47	54
Rational—BC Empirical Chart	47	54
USGS Magnitude and Frequency	80	67
Flow Transference	40	52
Direct Flow Transference	29	45
3 X Bankfull Area	--	52
Active Channel Width	--	53
Flow Frequency Analysis	26	42



Figure A-4. Clay Brandow, CAL FIRE Sacramento (retired), measuring Caspar Creek sub-watershed HEN channel width for the 3 X bankfull area calculation.



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Figure A-5. Schall et al. (2012) culvert sizing nomograph for a round pipe with inlet control. For the watershed HEN example, using the direct transference method result of 29 cfs, a projecting pipe inlet, and a HW/D ratio of 0.67, the culvert size is 45 inches (round to 48 in).

Part B. Predicting a 10-yr Recurrence Interval Event at North Fork Caspar Creek Subwatershed HEN and Comparing the Results to the 10-yr Discharge Determined with the Flow Frequency Analysis

To date, the largest flow documented in the HEN subwatershed is approximately a 20-year recurrence interval event based on the flow frequency analysis presented in Part A of the Appendix A (see Tables A-1 and A-2). **While this document was written to provide assistance in designing crossings for 100-year flood flows (including wood and sediment passage), Part B is included to provide information on how the various methods performed compared to actual gaging station data (as calculated by the 10-year flood flow using the flow frequency analysis).** It is assumed that: (1) the 31 years of record at HEN are long enough to adequately determine a reasonable estimate of the 10-year discharge, and (2) the techniques that come the closest to predicting the 10-year event for subwatershed HEN based on the flow frequency analysis would therefore likely provide the best estimate of a 100-year event for this small basin.⁵⁷

Rational Method

Known Information:

- Drainage area (A) = 96 acres for HEN
- 10-yr 10 minute NF Caspar Creek rainfall = 0.450 inches (NOAA website)
- 10-yr 30 minute NF Caspar Creek rainfall = 0.749 inches (NOAA website)
- 10-yr 60 minute NF Caspar Creek rainfall = 1.06 inches (NOAA website)
- Channel length = 0.5 miles from the ridge to the gaging station
- Difference in elevation = 550 feet from the ridge to the gaging station
- Soil type = loam

Calculate:

$$Q_{10} = CIA$$

Time of Concentration (using the California Culvert Practice or modified Kirpich equation):

$$T_c = \left(\frac{11.9(L)^3}{H} \right)^{0.385}$$

where:

T_c = time of concentration (hours)

L = length of the channel in miles from the head of the watershed to the crossing point

H = elevation difference between the highest point in the watershed and the crossing point (feet)

$$T_c = [(11.9 (0.5 \text{ miles})^3 / 550 \text{ feet})]^{0.385}$$

$$T_c = 0.103 \text{ hours or } 6 \text{ minutes}$$

T_c = 6 minutes. Use 10-minute rainfall-depth-duration-frequency data from NOAA website

$$0.45 \text{ inches}/10 \text{ minutes} \times 60 \text{ minutes}/\text{hour} = 2.70 \text{ inches}/\text{hour}$$

$$I = 2.70 \text{ inches}/\text{hour}$$

$$C = 0.3 \text{ (loam soil, Table 1)}$$

$$Q_{10} = 0.3 \times 2.70 \text{ inches}/\text{hour} \times 96 \text{ acres}$$

$$Q_{10} = 77.8 \text{ or } 78 \text{ cfs}$$

Time of Concentration (using the Airport Drainage Formula):

$$T_c = ((1.8) (1.1 - C) (D^{0.5})) / (S^{0.33})$$

where:

T_c = time of concentration in minutes

C = runoff coefficient (dimensionless, 0 < C < 1.0)

D = distance in feet from the crossing to the point in the watershed with the greatest time of flow

S = slope in percent

⁵⁷ For an expanded test of the flow prediction methods using the flow records for five Caspar Creek sub-watersheds, see Cafferata and Reid (in press).

$$T_c = ((1.8) (1.1 - 0.3) (2640^{0.5})) / (21^{0.33})$$

$T_c = 27$ minutes, or approximately 30 minutes
 0.749 inches/30 minutes x 60 minutes/hour = 1.50 inches/hour
 $I = 1.50$ inches/hour
 $C = 0.3$ (loam soil, Table 1)
 $Q_{10} = 0.3 \times 1.50$ inches/hour x 96 acres
 $Q_{10} = 43$ cfs

Time of Concentration (using the Estimated Travel Times Method—modified from Cafferata and Reid 2013):

$$T_c = (HL/V_1) + (CL/V_2)$$

where:

T_c = time of concentration in seconds
 HL= hillslope length
 CL= channel length
 V_1 = subsurface flow/saturation overland flow rate (ft/sec)
 V_2 = channel flow rate (ft/sec)

$$T_c = 400 \text{ ft} / 0.1 \text{ ft/sec} + 2400 \text{ ft} / 6 \text{ ft/sec}$$

$T_c = 4400$ seconds or 73 minutes (use $T_c = 1$ hr)
 $I = 1.06$ inches/hr (using the NOAA website data)
 $C = 0.3$ (loam soil, Table 1)
 $Q_{10} = 0.3 \times 1.06$ inches/hour x 96 acres
 $Q_{10} = 30.5$ or 31 cfs

Time of Concentration (using the BC Empirical Chart—BC Government 1991):

T_c = value determined from Figure 1, Chapter 7 in BC Government 1991 document (plot of T_c in hours vs square root of drainage area (km^2); use curve for steep slope (average basin slope $\geq 10\%$))

$$T_c = 0.72$$
 hours or 43 minutes (use $T_c = 1$ hr)
 $I = 1.06$ inches/hr (using the NOAA website data)
 $C = 0.3$ (loam soil, Table 1)
 $Q_{10} = 0.3 \times 1.06$ inches/hour x 96 acres
 $Q_{10} = 30.5$ or 31 cfs

Updated (10-yr RI Equation)

Known Information:

$$A = 0.15 \text{ miles}^2$$

$$P = 46.85 \text{ inches/year (Henry 1998)}$$

Calculate:

$$Q_{10} = 14.8 A^{0.88} P^{0.696}$$

$$Q_{10} = 14.8 (0.15)^{0.88} (46.85)^{0.696}$$

$$Q_{10} = 40.5 \text{ or } 41 \text{ cfs}$$

Flow Transference Method (Waananen and Crippen 1977)

Known Information:

$$A = 96 \text{ acres for HEN; } 1168 \text{ acres for the North Fork}$$

$$Q_{10g} \text{ (10-year RI discharge at NF Caspar Creek weir) } = 227.5 \text{ cfs (USGS PeakFQ Program)}$$

Calculate:

$$Q_{10u} = Q_{10g} (A_u/A_g)^b$$

$$Q_{10u} = 227.5 \text{ cfs (96 acres/1168 acres)}^{0.88}$$

$$Q_{10u} = 25.2 \text{ or } 25 \text{ cfs}$$

Direct Flow Transference Method (Skaugset and Pyles 1991)

Known Information:

A = 96 acres for HEN; 1168 acres for the North Fork

Q_{10g} (10-year RI discharge at NF Caspar Creek weir) = 227.5 cfs (USGS PeakFQ Program)

Watershed HEN is approximately one order of magnitude smaller than watershed NF Caspar

Calculate:

$$Q_{10u} = Q_{10g} (A_u/A_g)$$

$$Q_{10u} = 227.5 \text{ cfs (96 acres/1168 acres)}$$

$$Q_{10u} = 18.7 \text{ or } 19 \text{ cfs}$$

Table A-4. Summary of the results comparing predicted 10-year discharges at HEN.

Method	Predicted 10-yr RI Discharge (cfs)
Rational—CA Culvert Practice (modified Kirpich)	78
Rational—Airport Drainage	43
Rational—Estimated Travel Times	31
Rational—BC Empirical Chart	31
USGS Magnitude and Frequency – 10-yr RI equation	41
Flow Transference	25
Direct Flow Transference	19
Flow Frequency Analysis – 10 yr RI (see Table A-2, Part A)	16

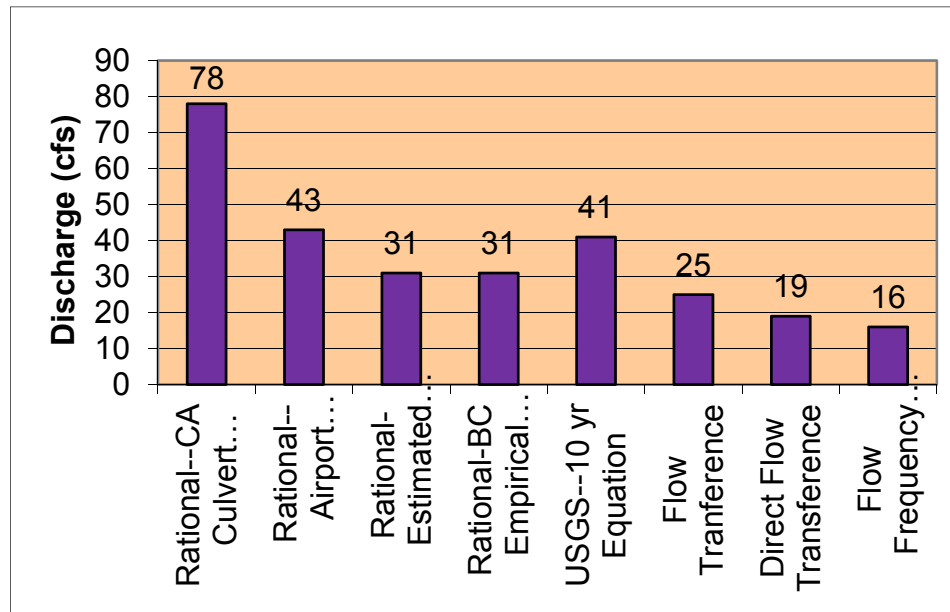


Figure A-6. Predicted 10-year recurrence interval discharges and the 10-year RI event determined using flow frequency analysis for subwatershed HEN (see Tables A-1 and A2, Appendix A—Part A).

Based on this limited comparison of the various estimated 10-year RI discharges (Q_{10}) to actual flow data from the Caspar Creek watershed, we can conclude the following:

- The Q_{10} estimate obtained for subwatershed HEN using flow frequency analysis is itself only an estimate of the actual 10-yr recurrence interval (RI) discharge and will change over time as the flow record expands. It is, however, assumed to be the best current estimate of the 10-yr RI discharge available and therefore is used as a standard against which the other discharge-estimating methods are compared.
- The direct flow transference method comes the closest to predicting the 10-year RI flow event for watershed HEN at Caspar Creek compared to the results of the flow frequency analysis obtained using the USGS PeakFQ program (Table A-4, Figure A-6).
- The direct flow transference method is preferred for predicting a peak discharge of a given RI if the gaged and ungaged watersheds are in close proximity, are hydrologically similar, and are approximately the same size (within roughly one order of magnitude)—as was the case for subwatershed HEN. Use of this method requires a nearby gaging station record of sufficient length (approximately 20 years or more). At the North Fork of Caspar Creek, this period of record is 53 years.
- Based on these results, it is concluded that the direct flow transference method likely provides the best estimate of the 100-year RI flood flow for subwatershed HEN.
- If the difference in gaged and ungaged watershed areas are larger than approximately one order of magnitude and/or the watershed is large (i.e., >2,500 acres), the flow transference method suggested by Waananen and Crippen (1977) is preferred.
- Most sites where crossings are proposed will not have the luxury of high quality, long-term downstream or hydrologically similar nearby gaging station data. If this type of data exists, it should be used. Where it does not, the Rational or updated USGS Magnitude and Frequency methods will be required, subject to the acreage limitations previously specified.

Part C-- Predicting the 100-Year Recurrence Interval Flood Flow for Teakettle Creek No. 2A, located in the North Fork Kings River Watershed (Figure A-7 and A-8)

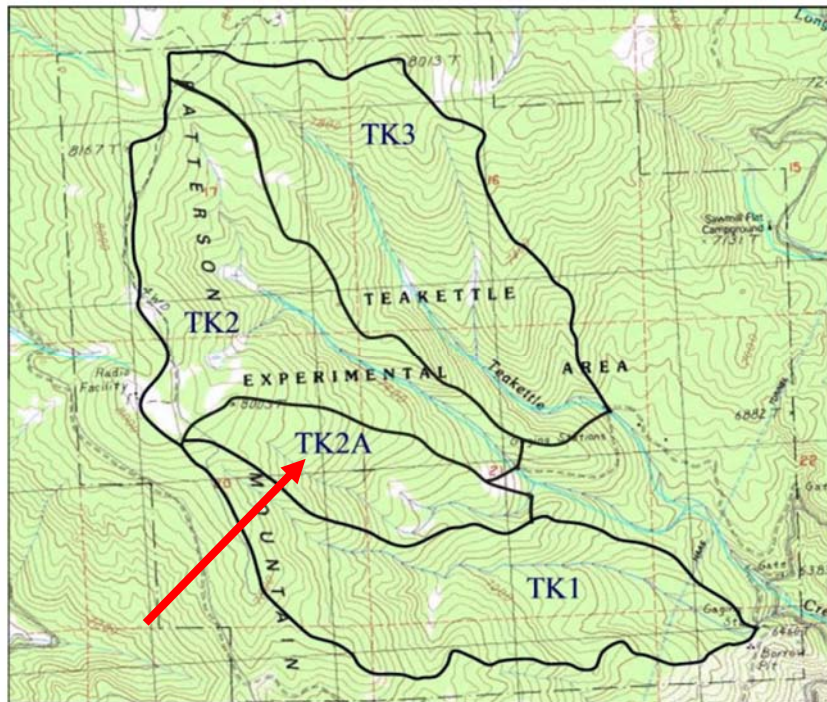


Figure A-7. The Teakettle Experimental Forest, USFS PSW, and location of watershed 2A, red arrow (from McGurk 2001).

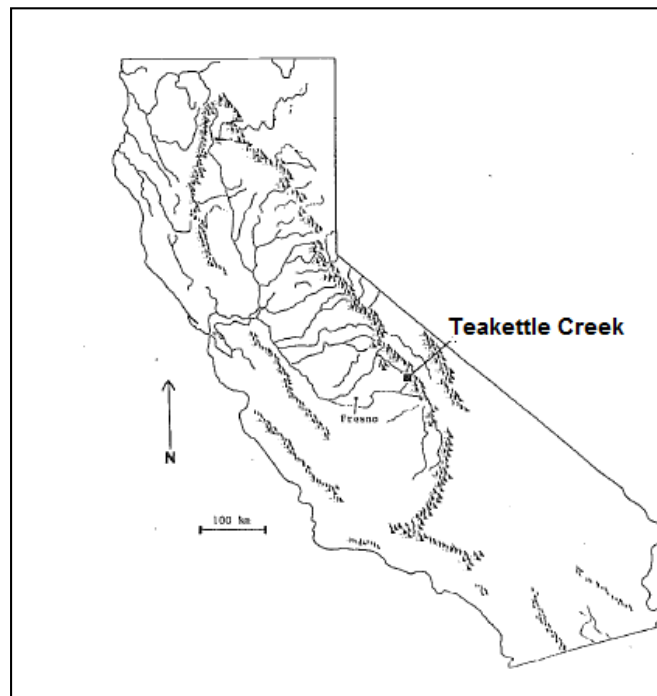


Figure A-8. Location of the Teakettle Creek watershed in North Fork Kings River watershed, southern Sierra Nevada Mountains (modified from McGurk 1989).

Rational Method

Known Information:

Drainage area (A) = 173 acres for Teakettle 2A
100-yr 15 minute rainfall = 1.23 inches (NOAA website)
100-yr 60 minute rainfall = 2.05 inches (NOAA website)
Hillslope length = 565 feet
Channel length = 1.4 miles from the ridge to the gaging station
Average basin slope = 13.5%
Difference in elevation = 995 feet from the ridge to the gaging station
Maximum elevation = 8,000 feet
Soil type = coarse sandy loam (North et al. 2002)
Subsurface flow/saturation overland flow rate = 0.1 ft/sec (Dunne and Leopold 1978)
Channel flow rate = 6 ft/sec (Dunne and Leopold 1978)

Calculate:

$$Q_{100} = CIA$$

Time of Concentration (using the California Culvert Practice or modified Kirpich equation):

$$T_c = \left(\frac{11.9(L)^3}{H} \right)^{0.385}$$

where:

T_c = time of concentration (hours)

L = length of the channel in miles from the head of the watershed to the crossing point

H = elevation difference between the highest point in the watershed and the crossing point (feet)

$$T_c = [(11.9 (1.4 \text{ miles})^3 / 995 \text{ feet})]^{0.385}$$

$$T_c = 0.26 \text{ hours or 16 minutes}$$

T_c = 16 minutes. 15-minute rainfall-depth-duration-frequency data

1.23 inches/15 minutes x 60 minutes/hour = 4.92 inches/hour

I = 4.92 inches/hour

C = 0.3 (loam soil, Table 1)

$Q_{100} = 0.3 \times 4.92 \text{ inches/hour} \times 173 \text{ acres}$

$Q_{100} = 255.3$ or 255 cfs

Pipe diameter = 82 inches (assumes HW/D = 1.0 and projecting pipe)

Pipe diameter = 102 inches (assumes HW/D = 0.67 and mitered pipe)

Time of Concentration (using the Airport Drainage Formula):

$$T_c = ((1.8) (1.1 - C) (D^{0.5})) / (S^{0.33})$$

where:

T_c = time of concentration in minutes

C = runoff coefficient (dimensionless, $0 < C < 1.0$)

D = distance in feet from the point of interest to the point in the watershed from which the time of flow is the greatest

S = slope in percent

$$T_c = ((1.8) (1.1 - 0.3) (7392^{0.5})) / (13.5^{0.33})$$

$T_c = 53$ minutes, or approximately 60 minutes or 1 hour

I = 2.05 inches/hour

C = 0.3 (coarse sandy loam soil, Table 1)

$Q_{100} = 0.3 \times 2.05 \text{ inches/hour} \times 173 \text{ acres}$

$Q_{100} = 106.4$ or 106 cfs

Pipe diameter = 59 inches (assumes HW/D = 1.0 and projecting pipe)

Pipe diameter = 73 inches (assumes HW/D = 0.67 and mitered pipe)

Time of Concentration (using the Estimated Travel Times Method)

$$T_c = (HL/V_1) + (CL/V_2)$$

where:

T_c = time of concentration in seconds

HL= hillslope length

CL= channel length

V_1 = subsurface flow/saturation overland flow rate (ft/sec)

V_2 = channel flow rate (ft/sec)

$$T_c = 565 \text{ ft} / 0.1 \text{ ft/sec} + 5840 \text{ ft} / 6 \text{ ft/sec}$$

$$T_c = 6623 \text{ seconds or } 110 \text{ minutes (use } T_c = 2 \text{ hr)}$$

$$2.66 \text{ inches}/2 \text{ hours} = 1.33 \text{ inches/hr (using the NOAA website data)}$$

$$I = 1.33 \text{ inches/hr}$$

$$C = 0.3 \text{ (loam soil, Table 1)}$$

$$Q_{100} = 0.3 \times 1.33 \text{ inches/hour} \times 173 \text{ acres}$$

$$Q_{100} = 69 \text{ cfs}$$

$$\text{Pipe diameter} = \underline{53 \text{ inches}} \text{ (assumes HW/D} = 1.0 \text{ and projecting pipe)}$$

$$\text{Pipe diameter} = \underline{62 \text{ inches}} \text{ (assumes HW/D} = 0.67 \text{ and mitered pipe)}$$

Time of Concentration (using the BC Empirical Chart—BC Government 1991):

T_c = value determined from Figure 1, Chapter 7 in BC Government 1991 document (plot of T_c in hours vs square root of drainage area (km^2); use the curve for steep slope since average basin slope $\geq 10\%$)

$$T_c = 0.95 \text{ hours (use } T_c = 1 \text{ hr)}$$

$$I = 2.05 \text{ inches/hr (using the NOAA website data)}$$

$$C = 0.3 \text{ (loam soil, Table 1)}$$

$$Q_{100} = 0.3 \times 2.05 \text{ inches/hour} \times 173 \text{ acres}$$

$$Q_{100} = 106.4 \text{ or } 106 \text{ cfs}$$

$$\text{Pipe diameter} = \underline{59 \text{ inches}} \text{ (assumes HW/D} = 1.0 \text{ and projecting pipe)}$$

$$\text{Pipe diameter} = \underline{73 \text{ inches}} \text{ (assumes HW/D} = 0.67 \text{ and mitered pipe)}$$

Updated USGS Magnitude and Frequency Method

Known Information:

$$A = 0.27 \text{ miles}^2$$

$$P = 49.2 \text{ inches/year (North et al. 2002)}$$

Calculate:

$$Q_{100} = 20.6 A^{0.874} P^{1.24} H^{-0.250}$$

$$Q_{100} = 20.6 (0.27)^{0.874} (49.2)^{1.24} (7417.5)^{-0.250}$$

$$Q_{100} = 88.7 \text{ or } 89 \text{ cfs}$$

$$\text{Pipe diameter} = \underline{56 \text{ inches}} \text{ (assumes HW/D} = 1.0 \text{ and projecting pipe)}$$

$$\text{Pipe diameter} = \underline{67 \text{ inches}} \text{ (assumes HW/D} = 0.67 \text{ and mitered pipe)}$$

Flow Transference Method (Waananen and Crippen 1977)

Known Information:

$$A = 0.27 \text{ mi}^2$$

$$Q_{100g} \text{ (100-year RI discharge at Kings River above North Fork near Trimmer, CA (USGS 11213500) = } 61410 \text{ cfs (USGS PeakFQ Program); drainage area} = 952 \text{ mi}^2$$

Calculate:

$$Q_{100u} = Q_{100g} (A_u/A_g)^b$$

$$Q_{100u} = 61410 \text{ cfs} (0.27 \text{ mi}^2/952 \text{ mi}^2)^{0.77}$$

$$Q_{100u} = \mathbf{119 \text{ cfs}}$$

Pipe diameter = 65 inches (assumes HW/D = 1.0 and projecting pipe)

Pipe diameter = 78 inches (assumes HW/D = 0.67 and mitered pipe)

3 X Bankfull Area Method

Not applicable for the Sierra Nevada Mountains.

Flow Frequency Analysis Method

Known Information:

Table A-5. Annual peak discharges for Teakettle 2A for water years 1958 through 1981.

Year	Peak Q (cfs)	Rank
1958	6.0	7
1959	2.0	13
1960	1.0	16
1961	2.0	13
1962	2.7	12
1963	35.0	2
1964	4.7	8
1965	6.7	6
1966	3.0	10
1967	60.0	1
1968	1.5	14
1969	17.8	3
1977	1.3	15
1978	7.8	5
1979	3.6	9
1980	16.0	4
1981	2.9	11

Calculate:

Table A-6. Estimated discharges for various recurrence intervals (RIs), including the 100-year RI discharge (discharges estimated by the USGS PeakFQ program).

RI (yr)	Q (cfs)	95% Confidence Limits	
		Lower	Upper
2	4.5	2.7	7.3
5	12.7	7.8	24.6
10	23.0	13.3	52.9
25	45.0	23.5	129.9
50	71.0	34.1	241.7
100	108.6	48.2	433.2

$$Q_{100} = 109 \text{ cfs}$$

Pipe diameter = 60 inches (assumes HW/D = 1.0 and projecting pipe)

Pipe diameter = 73 inches (assumes HW/D = 0.67 and mitered pipe)

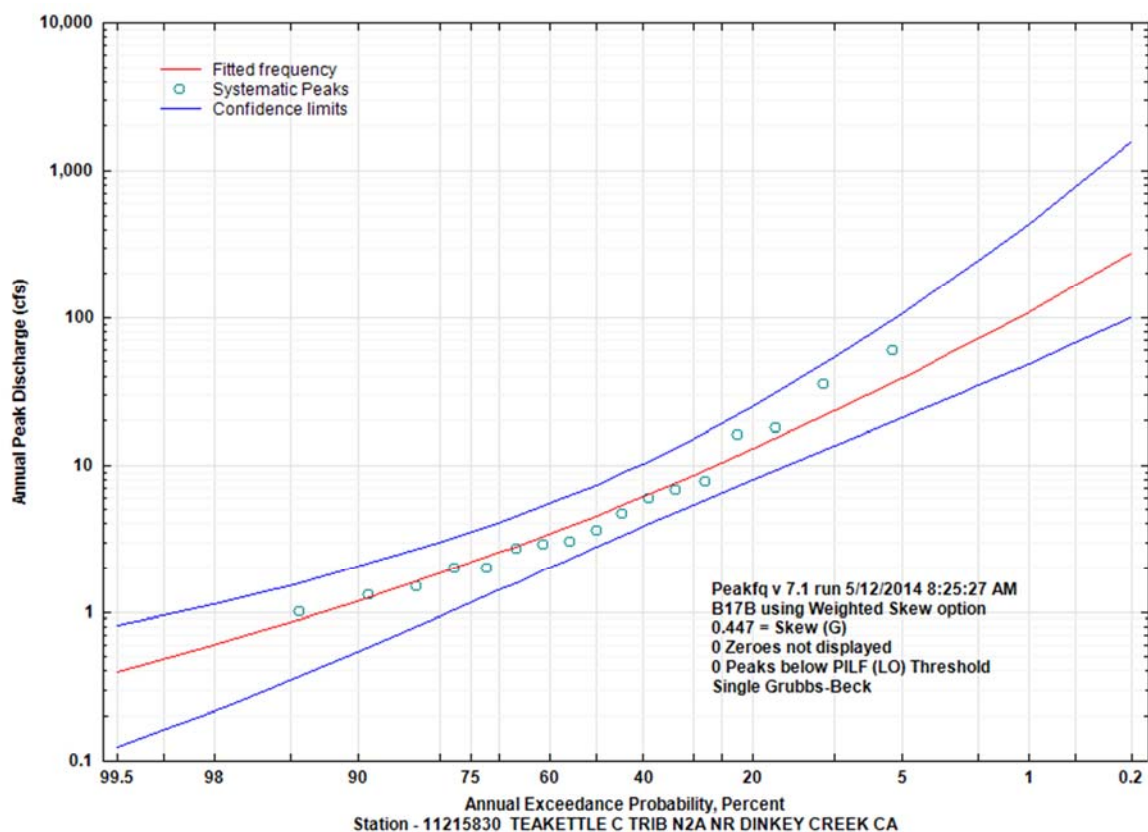


Figure A-9. USGS PeakFQ plot of annual exceedance probability vs. annual peak discharge for the Teakettle 2A watershed.

The Rational Method is recommended for basins with drainage areas less than 100 acres, and should never be used for basins greater than 200 acres. The updated USGS Magnitude and Frequency method is preferred over the Rational Method for drainage areas larger than 25-100 acres, depending on the region being considered (45 acres for the Sierra Nevada region). Both methods are utilized for the Teakettle 2A watershed for illustrative purposes. The flow transference method is preferred over both of these methods, but unregulated stream gaging stations have considerably larger drainage areas than Teakettle 2A and their period of record does not include the January 1997 flood event. We recommend using the average of the flow transference and updated USGS regional regression equation results, for an office-based estimate of discharge of 104 cfs (Table A-7). The flow frequency analysis confirms that this is a reasonable estimate for this small watershed (Tables A-5 and A-6, Figure 9). Based on the results from these office methods, a **72 inch CMP is appropriate with a HW/D of 0.67**. If a channel survey reveals that mobile wood is not considerable, a HW/D ratio of 0.8 could be considered, yielding a pipe diameter of 60 to 66 inches (Table A-7). Actual channel measurements revealed an active channel width of approximately three feet, and a bankfull channel width of 8.6 feet (Figure A-10).⁵⁸ Based on these channel measurements, a pipe diameter of 60 inches may be appropriate for this crossing site. If a round pipe of this size will not fit in the channel, other options can be considered, such installation of a pipe-arch culvert.

⁵⁸ Active channel width may not be appropriate for this site due to its high elevation (snow-dominated runoff area located above the rain-on-snow elevation band). High elevation channels are often smaller than what would be expected to accommodate the calculated 100-year flood flow.

Table A-7. Summary of the results using all the crossing sizing methods for determining the 100-year recurrence interval discharge and pipe diameters for the Teakettle 2A watershed.

Method	Predicted 100-Year Recurrence Interval Discharge (cfs)	Pipe Diameter—assuming HW/D Ratio = 0.67 for office-based methods, mitered inlet (in)	Pipe Diameter—assuming HW/D Ratio = 0.8 for office-based methods, mitered inlet (in)
Rational— CA Culvert Practice (modified Kirpich)	255	102	84
Rational—Airport Drainage	106	73	64
Rational—Estimated Travel Times	69	62	54
Rational—BC Empirical Chart	106	73	64
Updated USGS Magnitude and Frequency	89	67	60
Flow Transference	119	78	66
Direct Flow Transference	NA	NA	
3 X Bankfull Area	NA	NA	
Active Channel Width	--	34	
Flow Frequency Analysis	109	73	64



Figure A-10. Teakettle 2A watershed stream monitoring station (historic photo provided by Dr. Bruce McGurk, McGurk Hydrologic).

Appendix B—Watercourse Crossing Definitions and Diagrams

Bridge crossing: A structure spanning and providing passage over a watercourse or other opening.

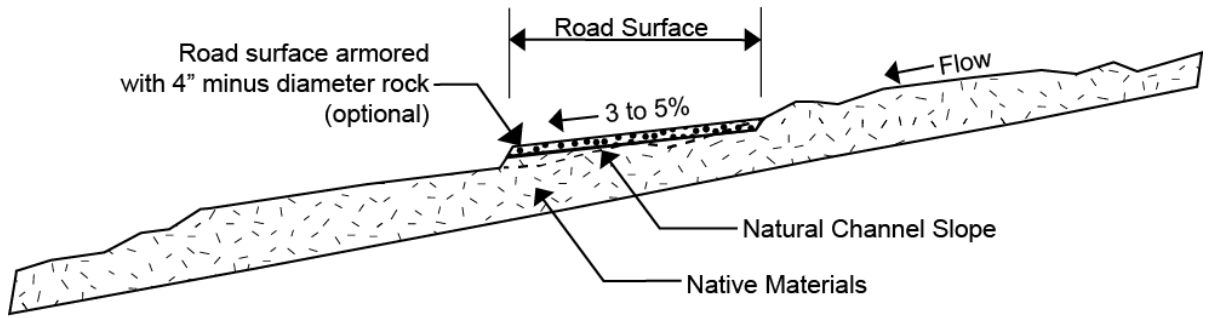
Ford crossing: A watercourse crossing where the road surface crosses at the natural grade of the channel. Thus, in ford crossings, no fill is placed within the channel to elevate the road grade and to make the crossing passible by vehicle traffic. If water is present at the time of use, the crossing is a “wet ford” and if water is not present at the time of use, the crossing is a “dry ford” (Figure B-1). In some cases a small amount of rock may be placed in the ford crossing to provide additional stability and a more suitable running surface for vehicle traffic or to ease the transition from the channel banks to the natural grade of the channel.

Rock-fill crossing: A watercourse crossing where rock that is free of fines is placed as fill in the channel to establish a usable road grade through the crossing to accommodate traffic (Figure B-2). Often a thin layer of sacrificial small-diameter rock is placed on top of the rock fill to provide a running surface that can accommodate truck traffic. Streamflow will typically pass through the rock fill during periods of low flow, but will pass over the rock fill during periods of high flow.

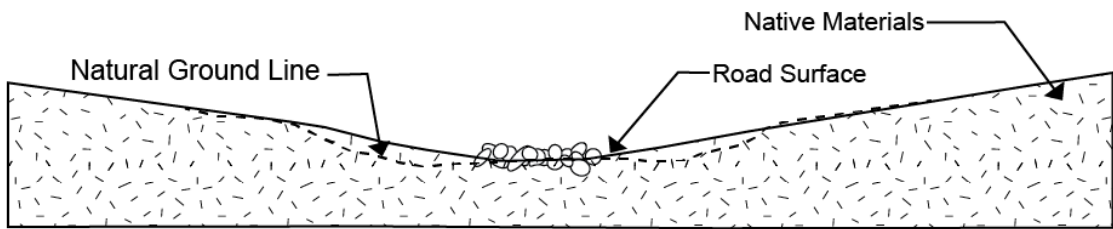
Rock-armored crossing: A watercourse crossing where fill, often composed of native earth material, is placed in the channel to establish a usable road grade through the crossing to accommodate traffic. The outfall of the crossing and road surface are protected against scour by revetment composed of rock (Figure B-3). Streamflow will typically pass over, rather than through, the crossing fill.

Vented crossing: A watercourse crossing structure designed to allow low water flow in the stream channel to pass through the structure (e.g., culverts) below a hardened (usually rock or concrete) roadway (Figure B-4). During periods of high water or flooding, streamflow passes over the roadway.

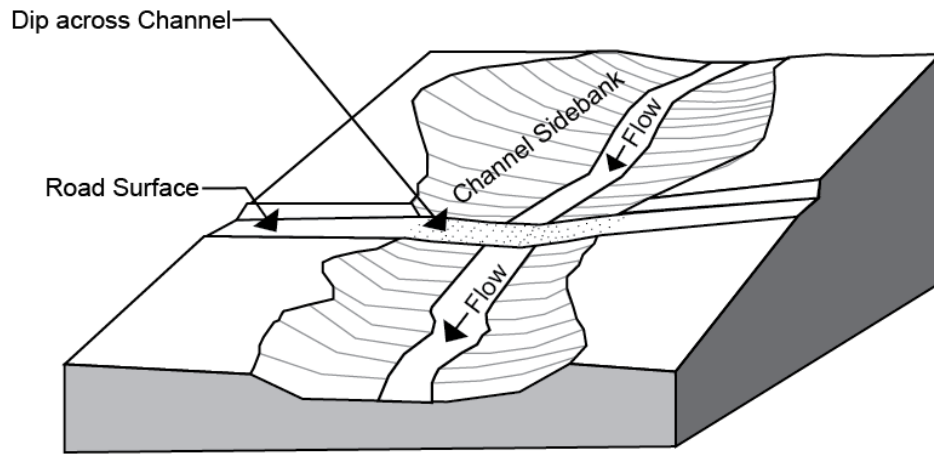
FORD CROSSING DIAGRAMS



CHANNEL PROFILE



CHANNEL CROSS-SECTION



ISOMETRIC VIEW

Figure B-1. Ford crossing diagrams.

ROCK-FILL CROSSING DIAGRAMS

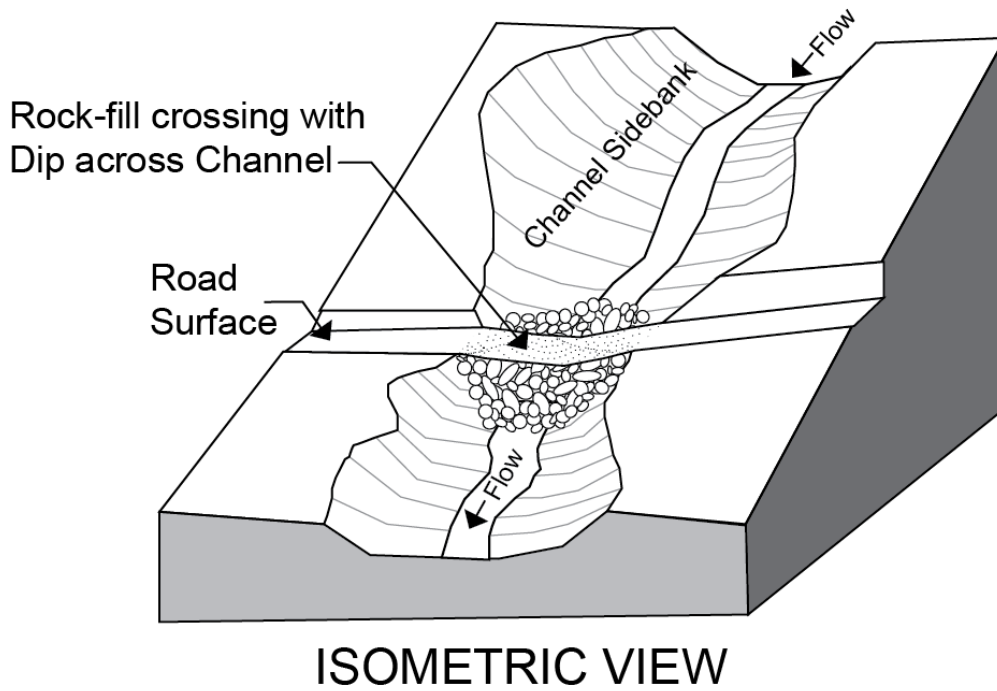
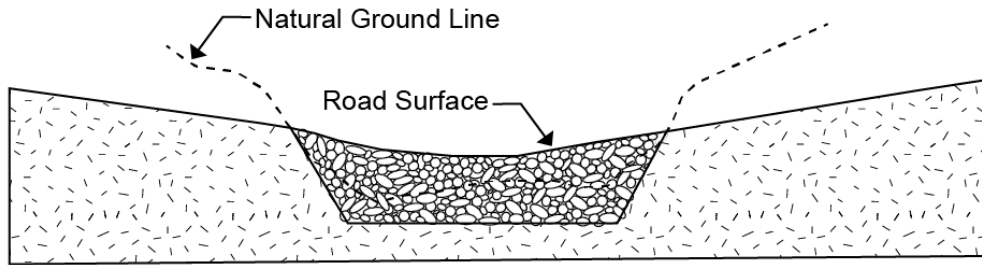
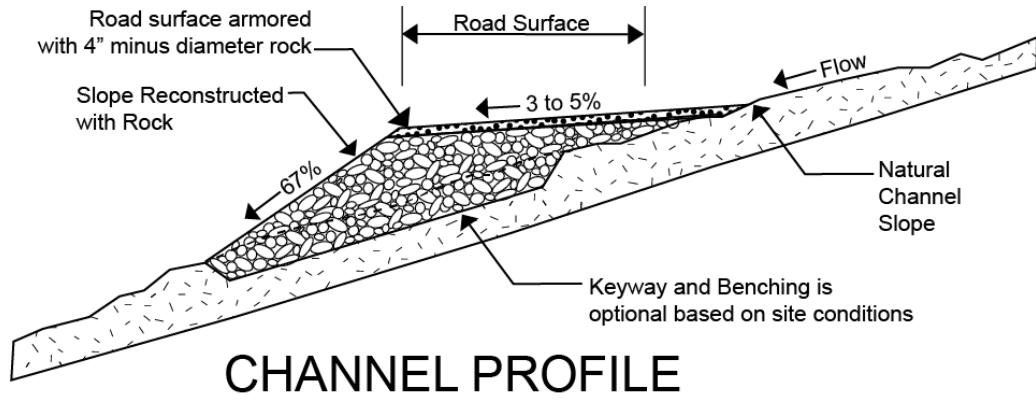
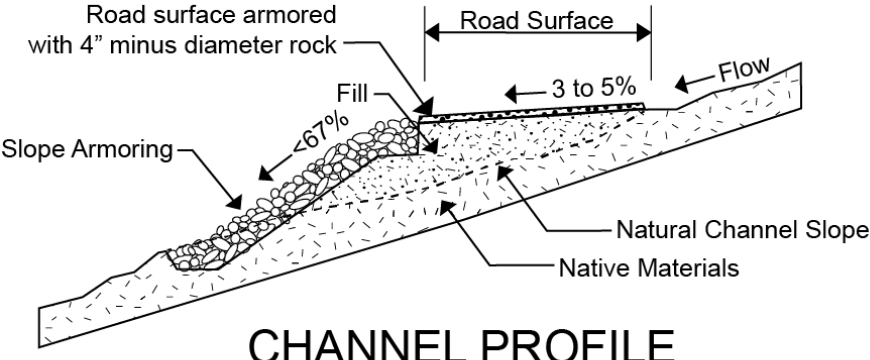
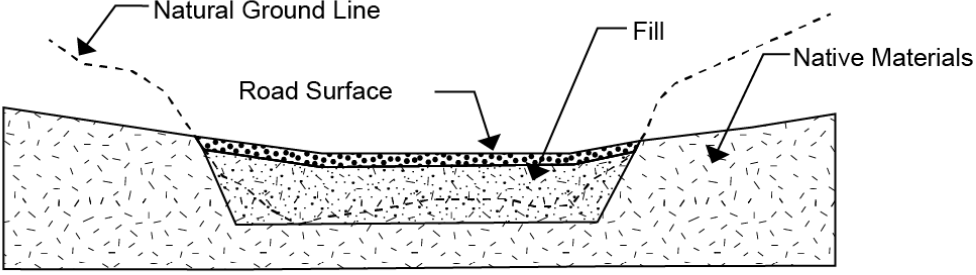


Figure B-2. Rock-fill crossing diagrams.

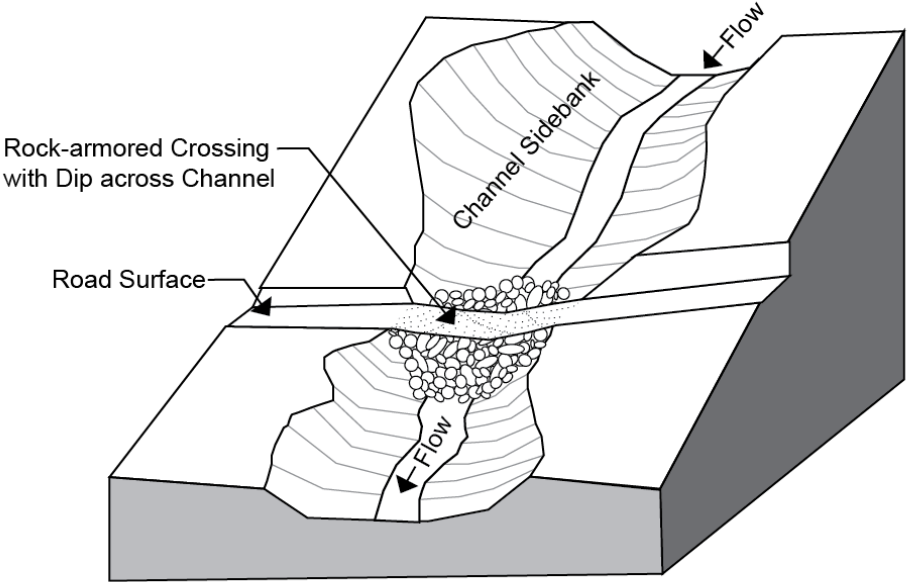
ARMORED CROSSING DIAGRAMS



CHANNEL PROFILE



CHANNEL CROSS-SECTION



ISOMETRIC VIEW

Figure B-3. Rock-armored crossing diagrams.

VENTED FORD CROSSING DIAGRAMS

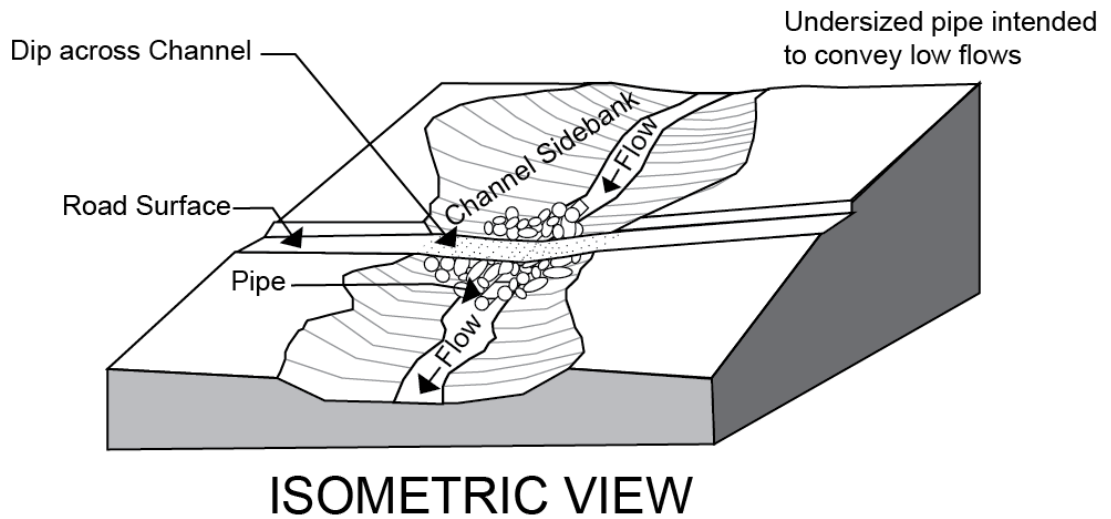
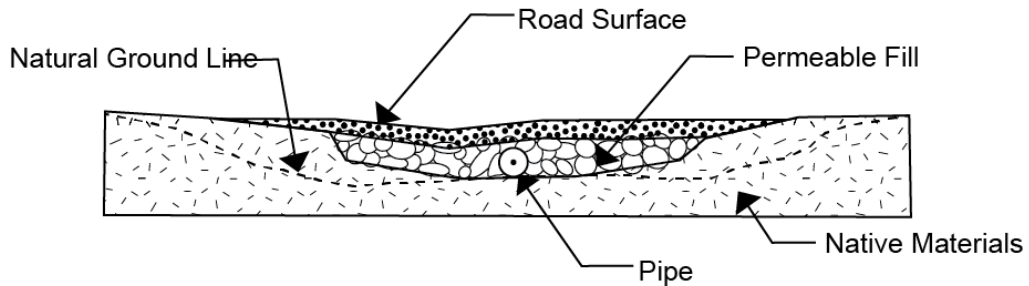
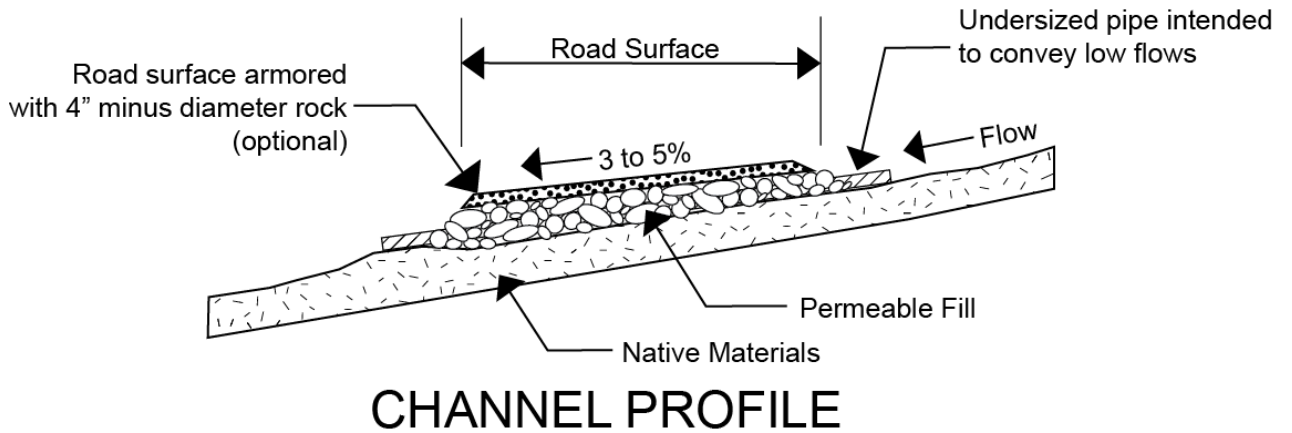


Figure B-4. Vented ford crossing diagrams.

Appendix C—Tables of Crossing Fill Heights and Fill Volumes

Assumes: Road Width of 16 feet; Fixed channel width of 2 ft.; Fillslope inclination of 67% (1.5h:1v); and 67% side slopes.

		Height of road surface (fill) above culvert inlet, ft.												
		2	4	6	8	10	12	14	16	18	20	22	24	26
Channel slope, percent.	0	2.0	4.0	6.0	8.0	10.0	12.0	14.0	16.0	18.0	20.0	22.0	24.0	26.0
		12	35	74	131	209	311	439	596	786	1012	1275	1580	1928
	5	3.1	5.3	7.6	9.8	12.0	14.3	16.5	18.7	21.0	23.2	25.4	27.7	29.9
		18	49	97	168	264	388	545	737	969	1243	1564	1934	2358
	10	4.4	6.9	9.4	11.9	14.4	16.9	19.4	22.0	24.5	27.0	29.5	32.0	34.5
		28	67	129	218	338	494	689	928	1216	1556	1954	2413	2937
	15	5.9	8.8	11.6	14.4	17.3	20.1	23.0	25.8	28.6	31.5	34.3	37.2	40.0
		41	93	174	288	442	640	888	1192	1557	1987	2490	3070	3732
	20	7.8	11.0	14.3	17.5	20.7	24.0	27.2	30.5	33.7	36.9	40.2	43.4	46.6
		62	132	238	388	589	847	1170	1564	2036	2594	3244	3993	4848
	25	10.1	13.9	17.6	21.3	25.1	28.8	32.5	36.3	40.0	43.7	47.5	51.2	54.9
		92	189	334	537	808	1154	1579	2103	2730	3470	4331	5323	6454
	30	13.1	17.5	21.8	26.2	30.5	34.9	39.3	43.6	48.0	52.4	56.7	61.1	65.4
		140	277	481	764	1139	1618	2214	2939	3806	4827	6015	7382	8941

= Maximum height of fill above culvert outlet, ft.
 = Volume of fill, cubic yards

Assumes: Road Width of 16 feet; Fixed channel width of 2 ft.; Fill Face inclination of 50% (2h:1v); and 67% side slopes.

		Height of road surface (fill) above inlet, ft.												
		2	4	6	8	10	12	14	16	18	20	22	24	26
Channel slope, percent.	0	2.0	4.0	6.0	8.0	10.0	12.0	14.0	16.0	18.0	20.0	22.0	24.0	26.0
		12	37	79	141	226	338	479	654	865	1116	1410	1751	2142
	5	3.7	5.4	7.7	10.0	12.3	14.6	16.8	19.1	21.4	23.7	25.9	28.2	30.5
		19	53	107	186	295	437	617	839	1107	1426	1799	2231	2726
	10	4.6	7.2	9.9	12.5	15.1	17.7	20.4	23.0	25.6	28.2	30.8	33.5	36.1
		31	76	149	254	397	584	820	1110	1461	1877	2365	2929	3576
	15	6.5	9.6	12.6	15.7	18.8	21.8	24.9	28.0	31.1	34.1	37.2	40.3	43.3
		49	113	214	359	555	810	1132	1528	2006	2572	3235	4001	4879
	20	9.0	12.7	16.3	20.0	23.6	27.3	31.0	34.6	38.3	42.0	45.6	49.3	53.0
		80	174	321	530	812	1178	1638	2203	2883	3689	4632	5721	6969
	25	12.5	17.0	21.5	26.0	30.5	35.0	39.5	44.0	48.5	52.9	57.4	61.9	66.4
		135	283	509	830	1261	1819	2518	3376	4407	5628	7055	8703	10588
	30	17.7	23.5	29.2	34.9	40.7	46.4	52.1	57.9	63.6	69.4	75.1	80.8	86.6
		244	495	876	1412	2130	3054	4212	5679	7331	9344	11695	14408	17510

= Maximum height of fill above culvert outlet, ft.
 = Volume of fill, cubic yards

Appendix D—Rock-Armor Crossing Design Information

Part A—Design of Rock Riprap for Overtopping Flow

For discussion purposes, the general steps to design rock riprap using the USBR/CSU design method for overtopping flow on slopes greater than 4h:1v (25%) involve:

Step 1: Given an estimate of the 100-year flood flow value, approximate the overtopping depth of flow and peak unit discharge down the crossing outfall.

The 100-year flood flow is determined as explained in Chapter 2. The overtopping depth down the outfall is a function of the crossing geometry and is approximated using the broad-crested weir equation (Eq. 1).

$$H = \left(\frac{Q_{100}}{C * L} \right)^{\frac{2}{3}} \quad \text{Eq. 1}$$

Where:

- H = Head or overtopping depth above the chute (ft)
- Q_{100} = Estimated 100-year flood flow (ft³/s)
- L = Minimum width of outfall or chute where the highest unit discharge is expected to occur (ft)
- C = Weir flow coefficient (ft^{0.5}/s), assumed to be 2.84.

This approach assumes that the flows down the fillslope are confined in a riprap-lined trapezoidal chute with a fixed width, depth, and side slope from the outside edge of the road to the toe of the fillslope (Figure D-1). Using the broad-crested weir formula to estimate the depth of water in the trapezoidal chute is conservative, particularly since it assumes all flow is constricted into the inlet of the chute before the flow goes through a hydraulic drop as it accelerates down the fillslope. Thus, as long as the rocked outfall has a cross-section (width, depth, and sideslope) equivalent or larger than a trapezoidal chute defined using the broad-crested weir formula, the crossing should be capable of handling flood flows without diversion out of the armored chute. However, typical rock-armored crossings on logging roads are seldom constructed with an outside berm and trapezoidal chute. Instead, armored crossings typically have a broad dip (sag) in the road bed over the channel that is pronounced enough to ensure water flows across the road and down the fillslope without being diverted, but not so pronounced that it impedes truck traffic (Figure D-1).

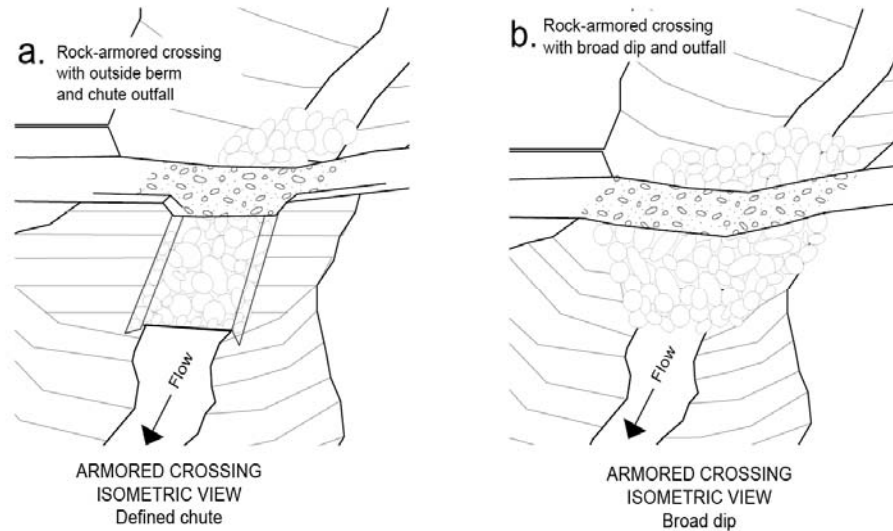


Figure D-1. Comparison of a rock riprap armored chute verses rolling dip with armored outfall.

Water flow across a typical rock-armored crossing goes from being laterally confined in the channel upstream to relatively unconfined where it fans out after intersecting the gentle-sloping road bed. As the water fans out on the road surface, the hydraulic energy and transport capacity of the water decreases allowing for an alluvial fan to deposit onto the road. This fan deposit can have a dramatic effect on concentrating flows and controlling the location where the flows discharge down the rock-armored outfall. For this reason, it is imperative that the broad dip of the crossing is pronounced enough to accommodate alluvial deposition without risk of the flows being diverted outside of the armored outfall. Moreover, regular maintenance may be required to address accumulated alluvial deposits that develop on the road surface to ensure the proper long-term function of the crossing.

As the flows cascade down the steep, concave fillslope (i.e., outfall), they become re-concentrated back into the native channel near the toe of the fill. For this reason, the highest unit discharge and the corresponding highest erosion potential for crossings constructed without a rock-armored trapezoidal chute would be the area near the toe of the fillslope. Therefore, the width of the outfall/chute near the fillslope toe should be used to define the width of the chute in Eq. 1 above and in determining the d50 rock size for the entire length of the armored outfall/chute.

For rock-armored crossings in mountainous terrain, experience shows that typical estimates of the outfall/chute width used to calculate the unit discharge should range from about 1.5 to 2.5 times the active channel width, depending on the lateral confinement of the downstream channel.

Step 2: Calculate the smallest possible median rock size (d50) that is stable given the material properties of the riprap, embankment slope, and unit discharge. The smallest possible median rock size (d50) can be calculated using Equation 2.

$$d50 = \frac{K_u q_f^{0.52}}{C_u^{0.25} S^{0.75}} \left(\frac{\sin \alpha}{(S_g \cos \alpha - 1)(\cos \alpha \tan \phi - \sin \alpha)} \right)^{1.11} \quad \text{Eq. 2}$$

Where:

- d50 = Median rock size (ft)
- K_u = Riprap sizing equation coefficient, equal to $0.525s^{0.52}/ft^{0.04}$
- q_f = Unit discharge at failure (ft³/s/ft)
- C_u = Coefficient of uniformity of the riprap (d60/d10)
- S = Slope of the embankment (ft/ft)
- S_g = Specific gravity of the riprap
- α = Slope of the embankment, degrees
- ϕ = Angle of repose of the riprap, degrees

Acceptable ranges of material properties common for riprap types are provided below, including the rock's: specific gravity, S_g (Table D-1); coefficient of uniformity, C_u , and porosity, n (Table D-2); and angle of repose, ϕ (Figure D-2).

Table D-1. Specific gravity for various rock types, adapted from http://geology.about.com/cs/rock_types/a/aarockspeggrav.htm.

Material	Specific Gravity
Andesite	2.5-2.8
Basalt	2.8-3.0
Diorite	2.8-3.0
Granite	2.6-2.7
Shale	2.4-2.8
Slate	2.7-2.8
Limestone	2.3-2.7
Earth (dry)	1.6-1.8

Table D-2. Coefficient of uniformity for riprap of various gradations, adapted from reported ranges in Lagasse et al. (2006), Abt et al. (1988), and Terzaghi et al. (1996).

Description	Coefficient of Uniformity	Porosity
Narrow or "single sized" (e.g., "uniform")	1.1 to 1.5	0.35 to 0.46
Wide (e.g., "well graded")	1.5 to 2.5	0.25 to 0.40
Very Wide (nominally, "quarry run")	Greater than 2.5	0.20 to 0.35

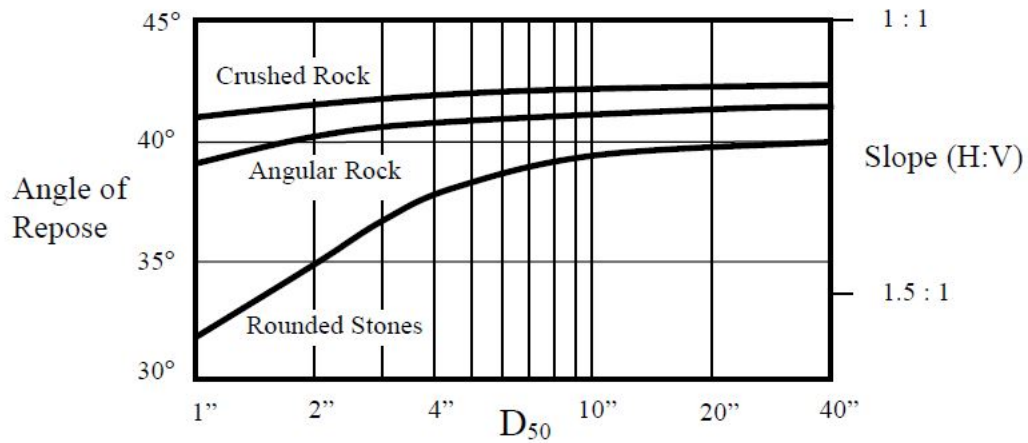


Figure D-2. Angle of repose for riprap based on average stone size, D_{50} , where 50% of particles are larger (Source: City of Knoxville Engineering Department).

Step 3: Select a riprap class or rock source that has a d_{50} diameter at or slightly larger than the d_{50} calculated. Table D-3 provides a general range of gradations for various riprap classes commonly used in Federal and State projects, and are generally available through commercial rock suppliers.

Table D-3. Minimum and maximum allowable particle size in inches (Lagasse et al. 2006).

Nominal Riprap Class by Median Particle Diameter		d_{15}		d_{50}		d_{85}		d_{100}
Class	Diameter (in.)	Min	Max	Min	Max	Min	Max	Max
I	6	3.7	5.2	5.7	6.9	7.8	9.2	12.0
II	9	5.5	7.8	8.5	10.5	11.5	14.0	18.0
III	12	7.3	10.5	11.5	14.0	15.5	18.5	24.0
IV	15	9.2	13.0	14.5	17.5	19.5	23.0	30.0
V	18	11.0	15.5	17.0	20.5	23.5	27.5	36.0
VI	21	13.0	18.5	20.0	24.0	27.5	32.5	42.0
VII	24	14.5	21.0	23.0	27.5	31.0	37.0	48.0
VIII	30	18.5	26.0	28.5	34.5	39.0	46.0	60.0
IX	36	22.0	31.5	34.0	41.5	47.0	55.5	72.0
X	42	25.5	36.5	40.0	48.5	54.5	64.5	84.0

However, in most cases, rock riprap used in the construction of logging roads and watercourse crossings is not “processed” into different size classes, but is generated as pit-run material from local rock sources where sorting of material is limited. When local pit-run rock is used, the RPF can visually inspect the rock for its quality and average rock size. Where necessary, simple aggregate point-count techniques, similar to that developed by Wolman (1954), can be applied to more accurately measure the rock size distribution of rock riprap. Provided the average rock diameter by volume is at least d50 size or larger, then the riprap should satisfy the requirements specified by the USBR/CSU method and be stable at the design hydraulic loading.

Step 4: Compute the interstitial velocity and the average velocity of the overtopping flow. The interstitial velocity (i.e., velocity of water flowing within the open area between rocks) and the average velocity of the overtopping flow is a function of the embankment slope, coefficient of uniformity, rock porosity, and d50 rock size. It is calculated using equations Eq. 3 and Eq. 4.

$$V_i = 2.48\sqrt{gd50} \left(\frac{S^{0.58}}{C_u^{2.22}} \right) \quad \text{Eq. 3}$$

Where:

- V_i = Interstitial velocity (ft/s)
- g = Acceleration due to gravity, 32.2 ft/s²
- d50 = Median rock size (ft)
- C_u = Coefficient of uniformity of the riprap (d60/d10)
- S = Slope of the embankment (ft/ft)

$$V_{ave} = \eta V_i \quad \text{Eq. 4}$$

Where:

- V_{ave} =Average flow velocity (ft/s)
- η =Porosity of the riprap
- V_i =Interstitial flow velocity (ft/s)

Step 5: Compute the average flow depth as if all the flow is contained within the thickness of the riprap layer. The thickness of the riprap layer is assumed to be two times the d50 rock size. For example, if the d50 rock size is 8 inches then the thickness of the riprap would be 2(d50) or 16 inches.

If the average flow depth is less than 2(d50), then the flow occurs entirely within the riprap layer and the design is complete. If the flow depth is greater than 2(d50), then flow overtops the riprap layer and the rock size determined in Step 2 must be increased and Steps 3 through 5 repeated until all the flow occurs within the riprap layer.

Rock-armored crossings designed and constructed in the forest setting are often required to pass not only clear water, but also debris and sediment that can infill the interstitial voids over time, causing the porosity and permeability of the riprap layer to decrease and flows to be forced to the surface. Theoretically, if the water is forced to

the surface due to infilling, then the water cascading down the fillslope would be increased and would be less aerated, resulting in a higher tractive force being exerted on the outside layer of riprap that could lead to crossing failure. This elevated risk of failure due to infilling over time should be considered prior to construction. If required, the designer may wish to apply a factor of safety to upsize the rock riprap in both size (d50) and thickness to mitigate against the effects of infilling. However, for most crossings in remote forest settings, it is likely that rock-armored crossings initially designed to convey the flows within the riprap layer without applying a factor of safety, as outlined above, would perform better under stressing storm events when compared to culvert crossings that could plug and be overtopped.

Part B—Example of a Rock-Armor Crossing Project

The following is an example problem showing the steps used to size riprap for steep (>25%), rock-armored crossings subjected to overtopping flows (particularly high risk crossings, as displayed in Table 5).

Problem:

Rock riprap is to be designed to protect a steep, 50%, fillslope against overtopping flows associated with a proposed rock-armored crossing in a Class II (non-fish bearing) watercourse in the North Coast Region, designated as Crossing #1 (Figures D-3 and D-4). The Class II watercourse has an approximately 15-foot wide, shallowly-incised active channel with flood terraces on either side. The estimated bankfull width is about 30 feet. The 100-year flow is estimated at the crossing to be 325 cfs (see attached calculations in Figure D-5) and the rock-armored chute will be designed to be 40 feet wide to encompass the natural bankfull width and to best fit the surrounding topography. The riprap will be sourced from a local pit and has the following estimated properties: Specific gravity (S_g) of 2.65, uniformity coefficient (C_u) of 2.0, porosity (n) of 0.45, and an angle of repose (ϕ) of 42 degrees.

Solution:

Following the steps outlined in the following worksheet (Figure D-6), the rock riprap will have the following parameters: minimum chute width of 40 feet, minimum chute depth of 2 feet, and a d50 rock size of 2.8 feet.

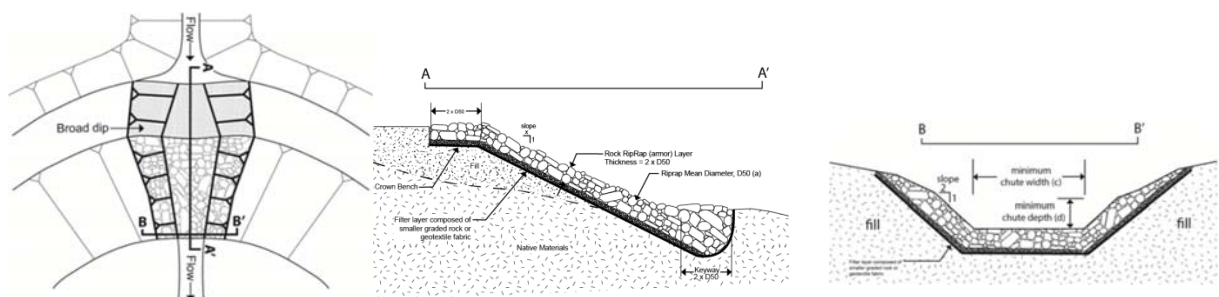
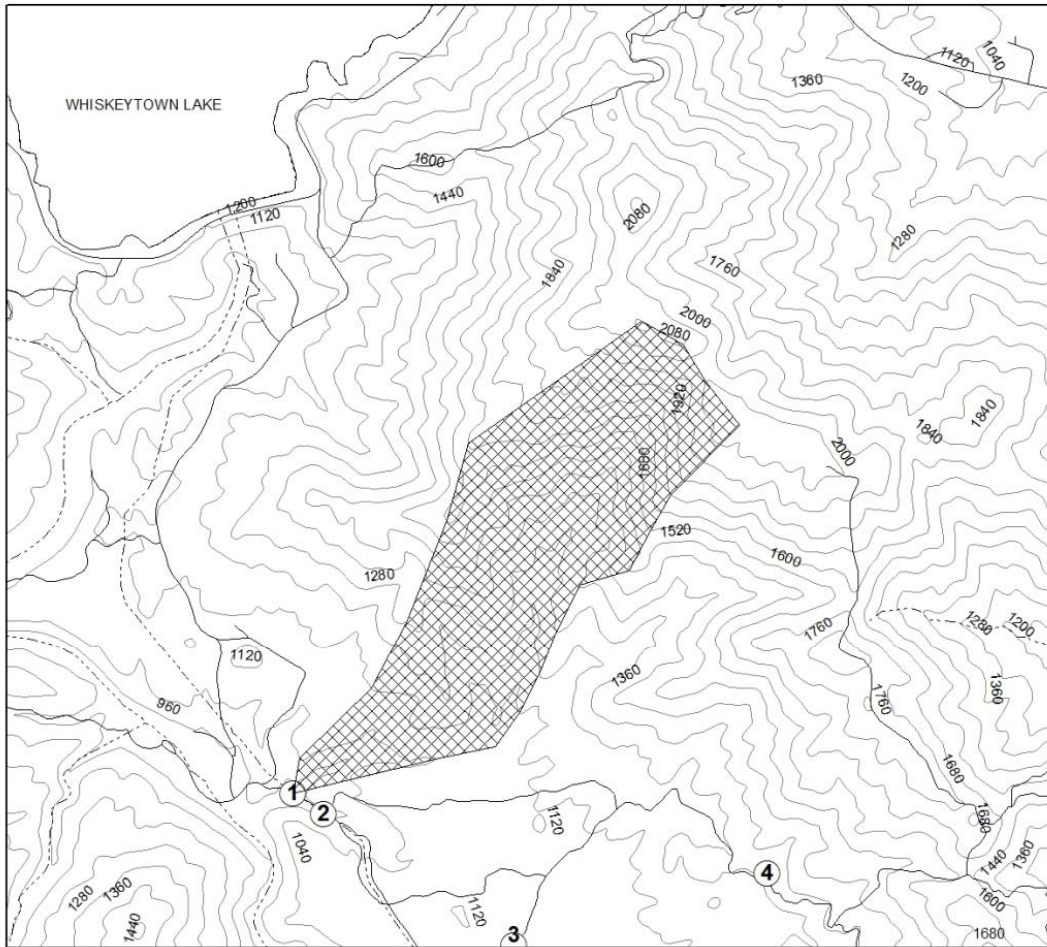




Figure D-3. Design components of the rock-armored crossing specified for Crossing #1.



Legend

-  Crossing Point Drainage Basin
-  Crossing Point

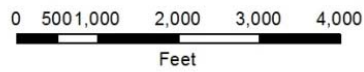


Figure D-4. Map of the drainage area associated with the rock-armored crossing example.

Location: Crossing 1 (sample problem)											
<i>(Enter data in fields with red-colored headings. Other data fields will be calculated automatically.)</i>											
Magnitude and Frequency Method for 100-year flood flow (A > 100 acres)								100-yr flood flow Q ₁₀₀ (cfs)			
No.	Crossing	Area (acres) A	Basin maximum elevation (ft)*	Crossing elevation (ft)*	Area (mi ²) A	Avg. Annual Precipitation (in/yr) P	Average Basin Elevation H	North Coast ⁽¹⁾ (NC)	Sierra ⁽²⁾ (S)	North-east ⁽³⁾ (NE)	Central Coast ⁽⁴⁾ (CC)
1	4	480	2012	930	0.750	48	1471	325.3	314.4	242.4	405.1

Magnitude & Frequency Q ₁₀₀ equations	
NC	(1) $Q_{100} = 48.5(A)^{0.866} (P)^{0.556}$
S	(2) $Q_{100} = 20.6(A)^{0.874} (P)^{1.24} (H)^{-0.250}$
NE	(3) $Q_{100} = 0.713(A)^{0.731} (P)^{1.56}$
CC	(4) $Q_{100} = 11.0(A)^{0.84} (P)^{0.994}$



Figure. D-5. Spreadsheet solutions for 100-year flood flow for the rock-armored crossing example (see Figure 34).

Design of riprap for overtopping flow, Design Guideline 5 - HEC 23 - FHWA (Lagasse et al., 2006)

$$H = \left(\frac{Q_{100}}{C \cdot L} \right)^{\frac{2}{3}} \quad \text{Eq. 3}$$

Where:

- H = Head or overtopping depth above the chute (ft)
- Q₁₀₀ = Estimated 100-year storm flow (ft³/s)
- L = Width of outfall or chute (ft)
- C = Weir flow coefficient (ft^{0.5}/s), assumed to be 2.84.

$$d50 = \frac{K_u q_f^{0.52}}{C_u^{0.22} S^{0.75}} \left(\frac{\sin \alpha}{(S_g \cos \alpha - 1)(\cos \alpha \tan \varphi - \sin \alpha)} \right)^{1.11} \quad \text{Eq. 4}$$

Where:

- d50 = Median rock size (ft)
- K_u = Riprap sizing equation coefficient, equal to 0.525S^{0.52}/ft^{0.04}
- q_f = Unit discharge at failure (ft³/s/ft)
- C_u = Coefficient of uniformity of the riprap (d60/d10)
- S = Slope of the embankment (ft/ft)
- S_g = Specific gravity of the riprap
- α = Slope of the embankment, degrees
- φ = Angle of repose of the riprap, degrees

$$V_i = 2.48 \sqrt{g d50} \left(\frac{S^{0.58}}{C_u^{2.22}} \right) \quad \text{Eq. 5}$$

Where:

- V_i = Interstitial velocity (ft/s)
- g = Acceleration due to gravity, 32.2 ft/s²
- d50 = Median rock size (ft)
- C_u = Coefficient of uniformity of the riprap (d60/d10)
- S = Slope of the embankment (ft/ft)

$$V_{ave} = \eta V_i \quad \text{Eq. 6}$$

Where:

- V_{ave} = Average flow velocity (ft/s)
- η = Porosity of the riprap
- V_i = Interstitial flow velocity (ft/s)

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Figure D-6. Rock-armored crossing riprap design details.

Rock-armored Crossing Riprap Design				
Input parameters*	Rock riprap Parameter:		Hydrologic/hydraulic Parameters:	
	Angle of repose, phi	42 degrees	Q100, cfs	325 cfs
	Porosity, n	0.45	Minimum width of rock chute, L.	40 ft.
	Uniformity Coef., Cu	2		
Other parameters	Embankment Slope, S (not to exceed 0.5 ft/ft)	0.5 ft/ft		
	Units constant, Ku	0.525	Unit Discharge**, qf	8.125 cfs/ft
	Specific gravity, Sg	2.65		
	Acceleration due to gravity, g	32.2 ft/s ²		
	Embankment Slope angle	26.6 degrees		
Wier flow coefficient, C	2.840			
Design steps with computations	Step 1: Determine the overtopping depth (Eq. 3):			
	Overtopping depth, H	2.02 ft		
	Step 2: Compute smallest possible median rock size (Eq. 4):			
	D50 rock size	1.99 ft	2*D50	3.98 ft.
	Step 3: Select riprap size class or source that has the correct d50 size.			
	Step 4: Compute the interstitial and average velocity (Eq. 5 and Eq. 6):			
	Interstitial velocity, Vi	2.9 ft/s		
	Average velocity, Vave	1.3 ft/s		
	Step 5: Compute the average flow depth (y) and test if flow is contained within the thickness, t, of the riprap (i.e. t<2d50):			
	Average flow depth, y (y=qf/Vave)	6.33 ft		
			Test:	FAIL If "PASS" record d50 and riprap thickness from Step 2 above; If "FAIL", continue below.
	If the test in Step 5 "FAILS", increase size of riprap and repeat Steps 2-5 until the design achieves a "PASS", see below:			
	Stone size increase*** (factor)	1.4	Increase factor above 1 in increments of 0.05 (eg. 1.05, 1.10, ...)	until the design receives a "PASS"
	Increased D50 rock size	2.8 ft	2*D50	5.57
	Interstitial velocity, Vi	3.4 ft/s		
Average velocity, Vave (Vave=nVi)	1.5 ft/s			
Average flow depth, y (y=qf/Vave)	5.35 ft			
		Test:	PASS If "PASS" record increased d50 and riprap thickness above.	

Notes:

* All parameters requiring user input are highlighted in "Yellow"

**Caution should be used in the event the unit flows exceed 10 cfs/ft.

***Must increase size of riprap to ensure the water runs interstitially within the layer of riprap.

Part C—Design of Rock Riprap for Overtopping Flow using the Simplified Nomograph

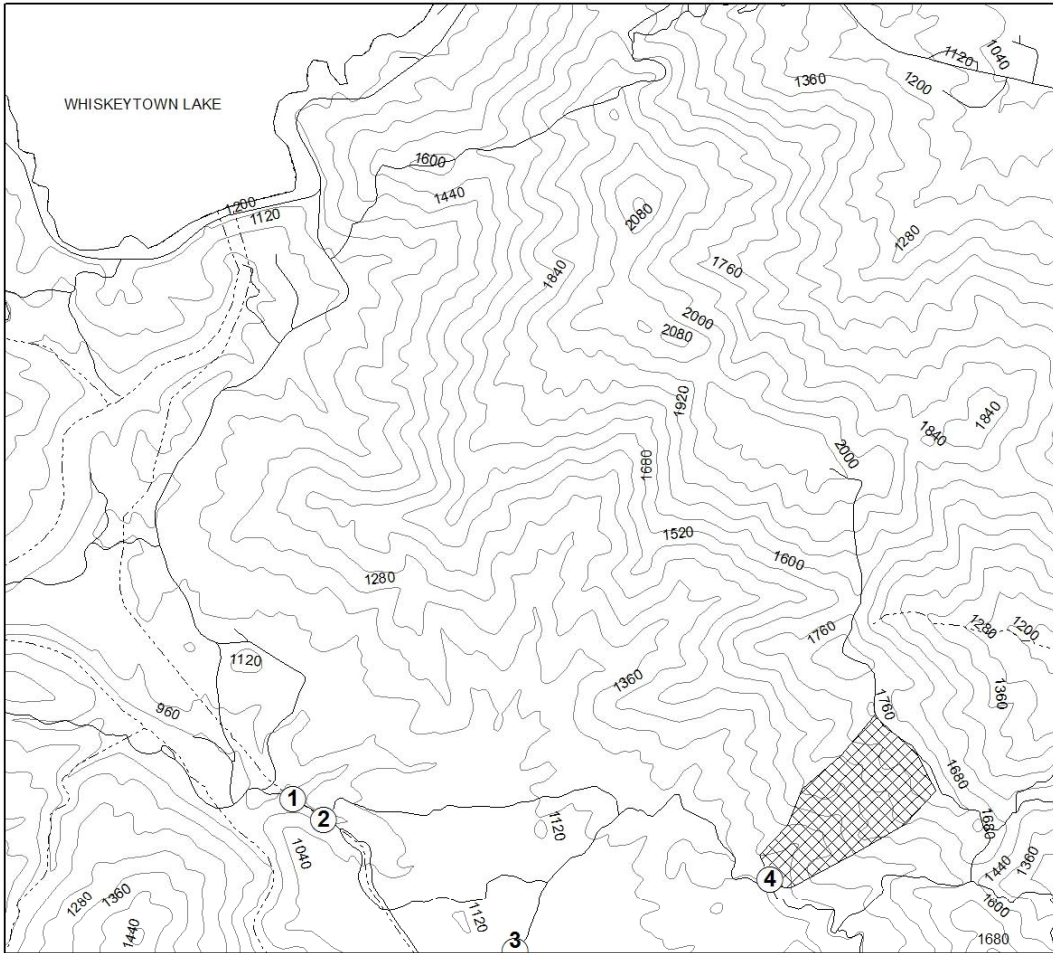
The following is an example problem showing the design of a rock-armored crossing subjected to overtopping flows.

Problem:

Rock riprap is to be designed to protect a steep, 50% fillslope against overtopping flows associated with a proposed rock-armored crossing in a Class III watercourse in the North Coast Region, designated as Crossing #4 (Figure D-7). The Class III watercourse has an approximately 3.5-foot wide active channel that is flanked by 50% side slopes. The 100-year flow is estimated at the crossing to be 48 cfs (see attached calculations in Figure D-8). The riprap will be sourced from a local pit and appears to comply with the material properties assumed in the simplified nomograph.

Solution:

Using the simplified nomograph (Figure D-9), the rock-armored crossing will be constructed with a minimum chute width of 6.6 feet, minimum chute depth of 1.8 feet, and a d50 rock size of 2.2 feet.



Legend

- Crossing Point
- ▣ Crossing Point Drainage Basin

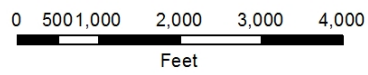


Figure D-7. Drainage area associated with the rock-armored crossing example using the simplified nomograph method.

Location: Crossing 4 (sample problem)											
<i>(Enter data in fields with red-colored headings. Other data fields will be calculated automatically.)</i>											
Magnitude and Frequency Method for 100-year flood flow (A > 100 acres)								100-yr flood flow Q ₁₀₀ (cfs)			
No.	Crossing	Area (acres) A	Basin maximum elevation (ft)*	Crossing elevation (ft)*	Area (mi ²) A	Avg. Annual Precipitation (in/yr) P	Average Basin Elevation H	North Coast ⁽¹⁾ (NC)	Sierra ⁽²⁾ (S)	North-east ⁽³⁾ (NE)	Central Coast ⁽⁴⁾ (CC)
1	4	53	1760	1206	0.083	48	1483	48.3	45.7	48.4	63.6



Magnitude & Frequency Q ₁₀₀ equations	
NC (1)	$Q_{100} = 48.5(A)^{0.866} (P)^{0.556}$
S (2)	$Q_{100} = 20.6 (A)^{0.874} (P)^{1.24} (H)^{-0.250}$
NE (3)	$Q_{100} = 0.713 (A)^{0.731} (P)^{1.56}$
CC (4)	$Q_{100} = 11.0 (A)^{0.84} (P)^{0.994}$

Rational Method for 100-year flood flow (A < 200 acres, best < 100 acres)								
		$T_c = 60((11.9 \times L^3)/H)^{0.385}$			$Q_{100} = CIA$			
	Crossing	Channel length (to top of basin) (mi) L	Elevation difference (ft) H	Concentration time (min) T _c	Runoff coefficient C	100-year Return-Period Precipitation (in/hr) I*	Area (acres) A	100-yr flood flow (cfs) Q ₁₀₀
No.		L	H	T _c	C	I*	A	Q ₁₀₀
1	0	0.5	554	30	0.3	2.88	53	45.8

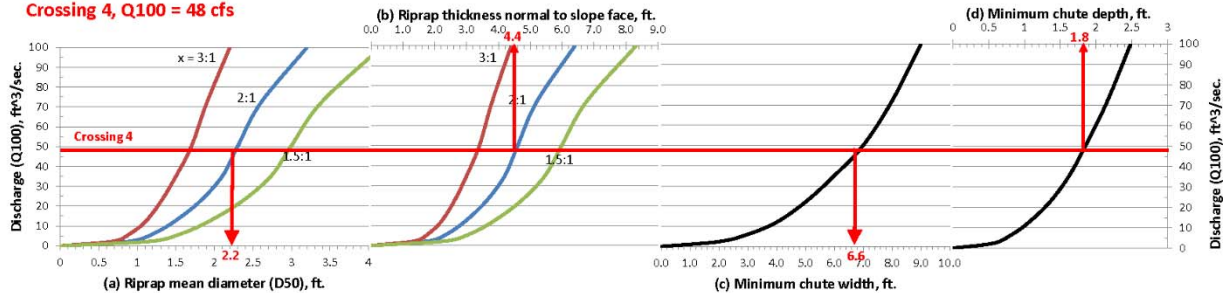
Figure D-8. Spreadsheet solutions for 100-year flood flow for the rock-armored crossing example using the simplified nomograph method (Figure 34).

Note: The USGS Magnitude and Frequency Method and Rational Method estimates for the 100-year flood flow are similar (time of concentration was estimated using the BC Government 1991 nomograph method).

Simplified design of rock-armored crossings

EXAMPLE

Crossing 4, $Q_{100} = 48$ cfs



Note: Slopes armored at a 1.5:1 slope inclination shall be less than 15 feet in height, and be constructed by placing rock in a running bond with at least 3 points of contact and with the long axis of the rock placed horizontal and facing into the slope.

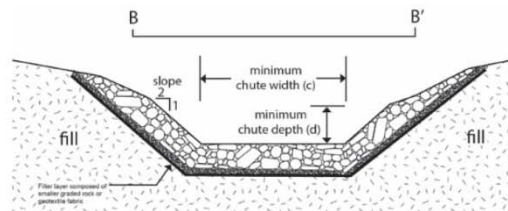
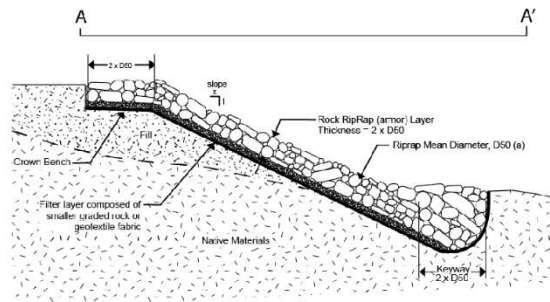
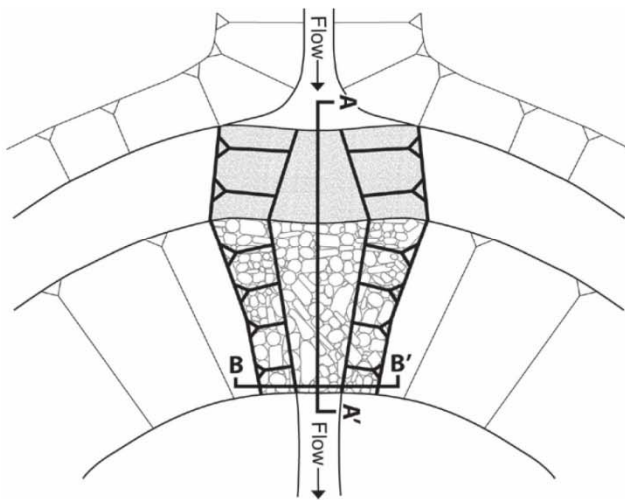


Figure D-9. Simplified nomograph example.

Appendix E—Bridge Design Information

Part A—Manning's n Values

Manning's n for Small Natural Stream Channels⁵⁹ Surface width at flood stage less than 30 m (100 ft)

1. Fairly regular section:
 - a. Some grass and weeds, little or no brush 0.030--0.035
 - b. Dense growth of weeds, depth of flow generally greater than weed height 0.035--0.05
 - c. Some weeds, light brush on banks..... 0.035--0.05
 - d. Some weeds, heavy brush on banks 0.05--0.07
 - e. Some weeds, dense willows on banks 0.06--0.08
 - f. For trees within channel, with branches submerged at high stage, increase all above values by: 0.01--0.02
2. Irregular sections, with pools, slight channel meander; increase values given above about: 0.01--0.02
3. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage:
 - a. Bottom of gravel, cobbles, and few boulders 0.04--0.05
 - b. Bottom of cobbles, with large boulders 0.05--0.07

⁵⁹ Adapted from: Schall et al. 2012, Hydraulic Design of Highway Culverts, Third Edition. U.S. Department of Transportation Report No. HIF-12-026, Hydraulic Design Series No. 5.

Part B—Hydraulic Structure Initial Site Examination Form

HYDRAULIC STRUCTURE INITIAL SITE EXAMINATION FORM (Adapted from Clarkin et al., 2006)			
(DATA SHEET FOR FORDS, BRIDGES, AND CULVERTS) (INCLUDE SITE SURVEY, LONGITUDINAL PROFILE, AND CROSS-SECTIONS)			
TIMBER HARVEST PLAN		ROAD NAME	
STRUCTURE NAME		STREAM NAME	
STRUCTURE NUMBER	LOCATION		
	SECTION	TOWNSHIP	RANGE
A. HYDROLOGIC & HYDRAULIC DATA			
1. SHOW ON A 15 MINUTE TOPOGRAPHIC MAP		2. NAME OF CLOSEST GAGING STATION	
DRAINAGE AREA		DISTANCE	MILES
3A. MANNING'S ROUGHNESS COEFFICIENT (N):	500' UPSTREAM:	3B. AVERAGE STREAMBED SLOPE	500' DOWNSTREAM:
4. DESCRIBE CHARACTER OF STREAM BED MATERIAL AND STREAM BANKS WITHIN THE 1000-FOOT AREA:			
5A. AMOUNT OF DEBRIS IN CHANNEL		5B. TYPE OF DEBRIS	
6. WATER ELEVATIONS			
6A. DATE AND FLOW DEPTH AT TIME OF SURVEY:	6B. EST. BASE FLOW DEPTH	OCCURS MONTH	6C. EST. EXTREME HIGH WATER DEPTH (HOW DETERMINED ?)
6D. CAUSE AND SEASON OF FLOODS:			
B. OTHER CHANNEL CHARACTERISTICS			
1. NOTE EVIDENCE OF INSTABILITY OF BANKS OR SCOUR			
2A. STRAIGHT CHANNEL, OR NOTE DEGREE OF SINUOUSITY		2B. HIGH FLOW ANGLE OF APPROACH (PARALLEL OR IMPINGING?)	
3. CHANNEL STABILITY (AGGRADATION, DOWNCUTTING, LATERAL CHANNEL MIGRATION, ETC)			
4. CHANNEL CLASSIFICATION (ROSGEN OR OTHER)			
5. CHANNEL ENTRENCHMENT (RATIO = FLOOD-PRONE / BANKFULL WIDTH)			
6. UPSTREAM / DOWNSTREAM STRUCTURES AFFECTING SITE (DAMS, BRIDGES, ETC.)			
7. OTHER SITE ASSESSMENT FACTORS			
C. FOUNDATION CONDITIONS			
1. CHARACTER OF SURFACE OR LOCAL MATERIALS:			
2. ESTIMATED DEPTH TO BEDROCK FEET	2A. BEDROCK TYPE & CONDITION		
3. ANY SPECIAL FOUNDATION CONDITIONS? INVESTIGATION NEEDED? EXPLAIN:			
D. EXISTING STRUCTURE			
1. TYPE OF EXISTING STRUCTURE	1A. NO & LENGTH OF SPANS	1B. TYPE OF CULVERT	1C. SIZE
2. WATERWAY OPENING	FEET WIDE OR SQUARE	2A. WATERWAY ADEQUATE? YES NO	
3. STRUCTURE AFFECTED BY: DEBRIS ICE DAMAGE SCOUR		4. DOES STRUCTURE CONSTRICT THE NATURAL CHANNEL: YES NO	
5. CONDITION OF EXISTING STRUCTURE:			
E. PROPOSED STRUCTURE			
1. BRIDGE OR LOW-WATER CROSSING TYPE			1A. LOADING (JUSTIFY IF OTHER THEN HS 20)

1B. WIDTH	1C. SUBSTRUCTURE OR SPECIAL NEEDS		
2. TYPE OF CULVERT			2A. SIZE
2B. CULVERT DESIGN ISSUES?			
2C. CORROSION OR ABRASION CONCERNS?		2D. TYPE OF FILL MATERIAL TO BE USED	
F. MISCELLANEOUS DATA			
1. TIME AND DURATION OF CONSTRUCTION SEASON		2. RIPRAP IS AVAILABLE YES NO	2A. DISTANCE FROM SITE AT MILES
2B. DESCRIPTION OF RIPRAP MATERIAL			
3. TRAFFIC CONTROL AND SAFETY NEEDS			
4. ROADWAY ALIGNMENT AND GRADE (ADEQUATE?)			
5. CHANNEL OR STRUCTURE ALIGNMENT CHANGES RECOMMENDED (SHOW ON COPY OF SITE PLAN)			
6. ARE DIKES OR BANK PROTECTION REQUIRED TO CONTROL FLOW (SHOW ON COPY OF SITE PLAN)			
7. DESCRIPTION OF ON-SITE CONSTRUCTION MATERIAL TO BE USED			
8. STORAGE AND/OR WASTE AREAS AVAILABLE FOR CONSTRUCTION (LOCATION, SIZE, AND DESCRIPTION)			
9. WHAT IS THE MAXIMUM LENGTH OF GIRDERS THAT CAN BE HAULED TO THE SITE? FEET			
10. METHOD OF CONSTRUCTION CONTRACT	FORCE ACCOUNT		TIMBER PURCHASER
11. OTHER REMARKS AND SPECIAL RECOMMENDATIONS			
G. FISH AND OTHER WILDLIFE PASSAGE CONSIDERATIONS			
1A. IS FISH PASSAGE REQUIRED? YES NO	1B. IF YES, WHAT SPECIES AND LIFE STAGES?	2. IS PASSAGE FOR OTHER SPECIES REQUIRED? (TERRESTRIAL, CRAWLING, SWIMMING) YES NO WHICH?	
3. SPECIAL/IMPORTANT CONSIDERATIONS FOR HABITAT PROTECTION?			
4. FOREST BIOLOGIST RECOMMENDATIONS			
PREPARED BY:		DATE	FOREST ENGINEER REVIEW: DATE
FIELD SITE SKETCH, LONGITUDINAL PROFILE, AND CROSS-SECTIONS			
<small>Adapted From: Form R5-7700-71</small>			

Part C—Example Bridge Project

The following is an example problem demonstrating the multi-step process to size a bridge crossing to pass the 100-year flood flow, including debris and sediment loads.

Problem

A permanent 40-foot long by 14-foot wide, engineered steel girder bridge is proposed to span a Class I watercourse (see Figure E-3). The proposed bridge would be placed on 2-foot wide by 2-foot tall by 16-foot long, engineered pre-cast concrete footings stacked two high and founded a minimum of 1-foot into firm native soil setback from the channel banks. Approximately 3 feet of compacted fill composed of locally sourced silty sand with gravel will be placed to construct the northern approach. No alteration to the existing channel bed or banks is anticipated. The bridge has been sized to adequately pass the anticipated 100-year flood flow and associated sediment and debris. See the supporting documentation on bridge sizing and hydraulic calculations below.

Step 1: Perform a longitudinal profile of the channel extending upstream and downstream of the crossing (Figure E-1).

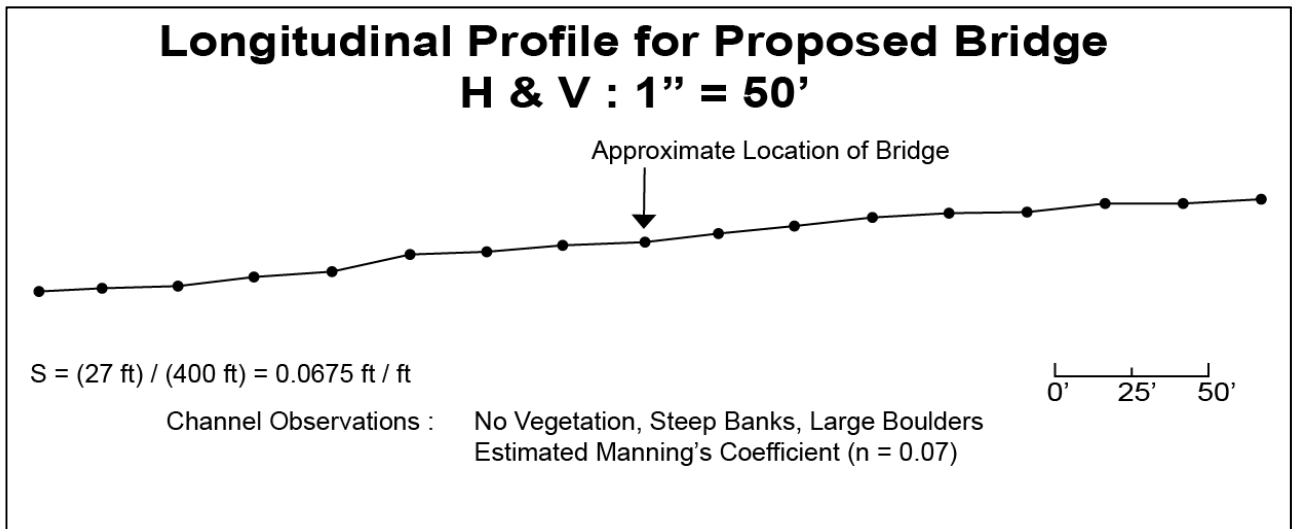


Figure E-1. Longitudinal profile for the bridge example. Note that hand drawn diagrams are acceptable for plan submission and review.

Average channel slope $S = 0.0675$ feet/foot
Estimated Manning's Coefficient $n = 0.07$

Step 2: Survey and draw a scaled cross-section perpendicular to the channel through the centerline of the proposed bridge crossing (Figure E-2).

Proposed Bridge Cross-Section H & V : 1" = 5'

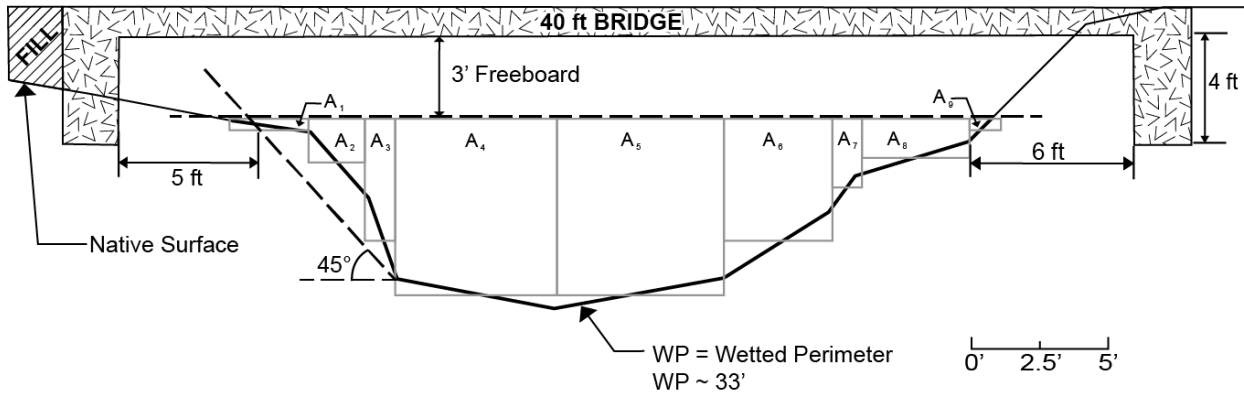


Figure E-2. Cross-section profile for the bridge example. Note that hand drawn diagrams are acceptable for plan submission and review.

Step 3: On the scaled cross-section drawing, project a horizontal line 3 feet below the bridge's lower surface.

Measure the "Wetted Perimeter":

$$WP = 33 \text{ feet}$$

Calculate the Wetted Cross-Section Area (A_{total})

$$A_{total} = A_1 + A_2 + A_3 + A_4 + A_5 + A_6 + A_7 + A_8 + A_9$$

$$A_{total} = (3 \text{ ft} \times 0.5 \text{ ft}) + (2 \text{ ft} \times 2 \text{ ft}) + (4 \text{ ft} \times 4.5 \text{ ft}) + (6 \text{ ft} \times 6.5 \text{ ft}) + (6 \text{ ft} \times 6.5 \text{ ft}) + (4 \text{ ft} \times 4.5 \text{ ft}) + (1 \text{ ft} \times 2.5 \text{ ft}) + (4 \text{ ft} \times 1.5 \text{ ft}) + (1.5 \text{ ft} \times 0.5 \text{ ft})$$

$$A_{total} = 1.5 \text{ ft}^2 + 4 \text{ ft}^2 + 4.5 \text{ ft}^2 + 39 \text{ ft}^2 + 39 \text{ ft}^2 + 18 \text{ ft}^2 + 2.5 \text{ ft}^2 + 6 \text{ ft}^2 + 0.5 \text{ ft}^2$$

$$A_{total} = 115 \text{ ft}^2$$

Step 4: Calculate the average water velocity of the channel below the bridge.

$$v = \left(\frac{1.49}{n} \right) \left(\frac{A}{WP} \right)^{2/3} S^{1/2} \quad \text{Eq. 1}$$

Where:

v = average flow velocity (ft/s)

n = Manning's roughness coefficient = **0.07**

S = channel slope (e.g., slope of energy grade line) (ft/ft) = **0.0675**

A = wetted cross-section area (ft²) = **115 ft²**

WP = wetted hydraulic perimeter (ft) = **33 ft**

$$v = \left(\frac{1.49}{n}\right) \left(\frac{A}{WP}\right)^{2/3} S^{1/2}$$

$$v = \left(\frac{1.49}{0.07}\right) \left(\frac{115}{33}\right)^{2/3} 0.0675^{1/2}$$

$$v = (21.3)(3.48)^{2/3} 0.0675^{1/2}$$

$$v = (21.3)(2.29)(0.26)$$

$$v = \mathbf{12.71 \text{ ft/s}}$$

Step 5: Calculate the hydraulic capacity of the bridge.

$$Q = vA \qquad \text{Eq. 2}$$

Where:

- Q = flow (cubic feet per second)
- v = average flow velocity (feet/second) = **12.71 ft/s**
- A = wetted cross-section area (square feet) = **115 ft²**

$$Q = vA = 12.71 \text{ ft/s} \times 115 \text{ ft}^2$$

$$Q = \mathbf{1,462 \text{ ft}^3/\text{s}}$$

Step 6: Compare the hydraulic capacity of the bridge to the estimated 100-year flood flow (Figure E-4).

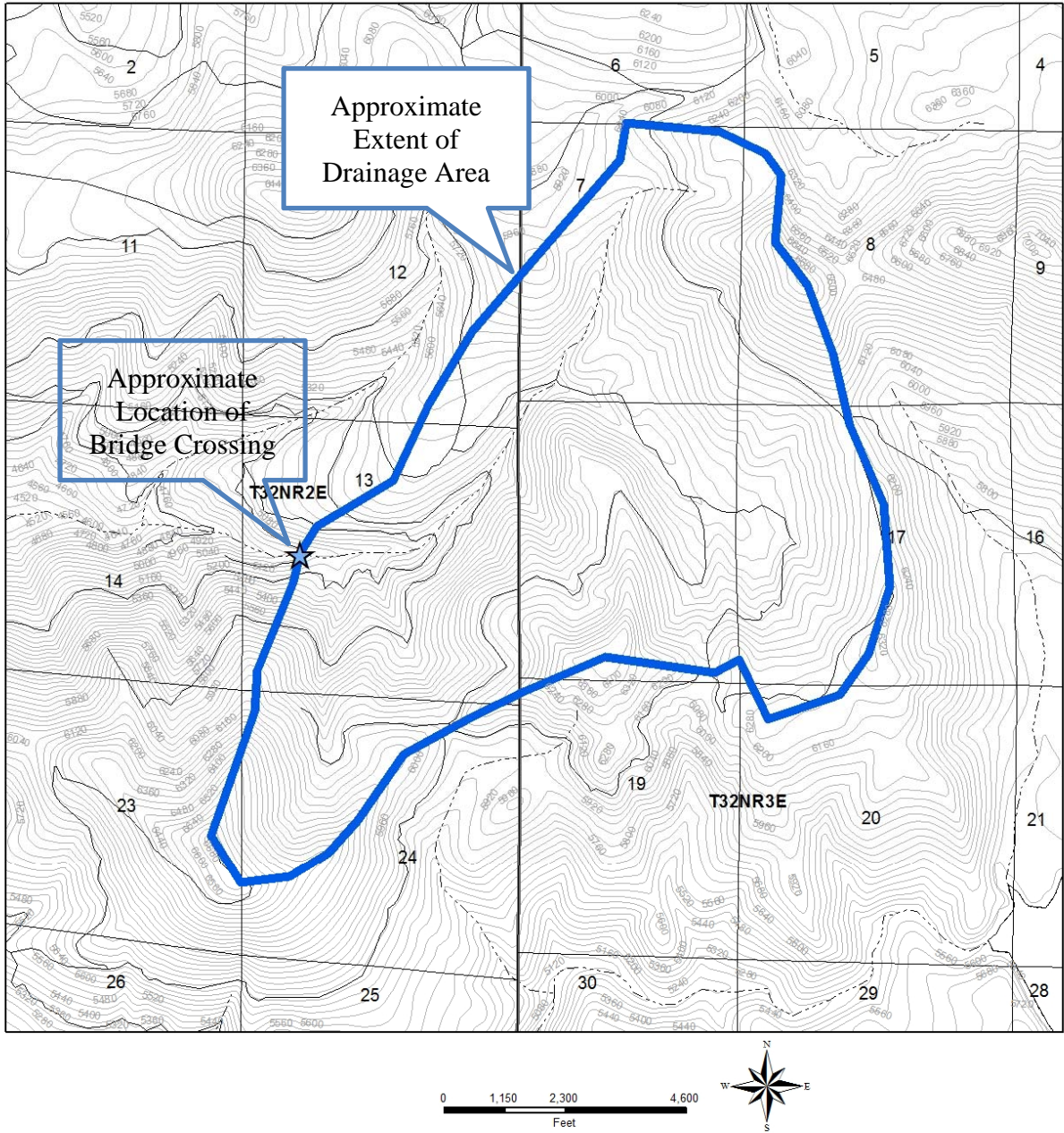


Figure E-3. Drainage area for the bridge example.

Location: Bridge (sample problem)											
<i>(Enter data in fields with red-colored headings. Other data fields will be calculated automatically.)</i>											
Magnitude and Frequency Method for 100-year flood flow (A > 100 acres)								100-yr flood flow Q ₁₀₀ (cfs)			
No.	Crossing	Area (acres) A	Basin maximum elevation (ft)*	Crossing elevation (ft)*	Area (mi ²) A	Avg. Annual Precipitation (in/yr) P	Average Basin Elevation H	North Coast ⁽¹⁾ (NC)	Sierra ⁽²⁾ (S)	North-east ⁽³⁾ (NE)	Central Coast ⁽⁴⁾ (CC)
1	Bridge	2130	6720	5000	3.328	50	5860	1209.5	861.0	767.7	1475.1



Magnitude & Frequency Q ₁₀₀ equations	
NC	(1) $Q_{100} = 48.5(A)^{0.866} (P)^{0.556}$
S	(2) $Q_{100} = 20.6 (A)^{0.874} (P)^{1.24} (H)^{-0.250}$
NE	(3) $Q_{100} = 0.713 (A)^{0.731} (P)^{1.56}$
CC	(4) $Q_{100} = 11.0 (A)^{0.84} (P)^{0.994}$

Figure E-4. Spreadsheet solutions for 100-year flood flow for the bridge crossing example.

The proposed bridge crossing is adequately sized, since the estimated 100-year flood flow is smaller than the discharge that will fit under the bridge with 3 feet of freeboard:

$$Q_{100} < Q_{\text{bridge}}$$

$$861 \text{ cfs} < 1462 \text{ cfs}$$

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