State of California
The Resources Agency
Department of Forestry & Fire Protection

Designing Watercourse Crossings for Passage of 100-year Flood Flows, Wood, and Sediment

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Michael Wopat, Greg Bundros,
and Sam Flanagan

California Forestry
Report No. 1

February 2004

Summary

Watercourse crossings associated with timber harvesting can produce substantial amounts of stream sediment. To reduce the potential for crossing failures and resulting impacts, the California Forest Practice Rules specify that all constructed or reconstructed permanent watercourse crossings must accommodate the estimated 100-year flow, including debris and sediment.

Three methods for making office-based estimates of 100-year recurrence-interval streamflows for ungaged basins are presented: (1) an analytical relationship between storm precipitation, watershed characteristics, and runoff, (2) regional regression equations based on long-term flow records, and (3) flow transference methods that adjust nearby measured discharges for differences in drainage basin size. Watershed area limitations for each method are identified. In general, flow transference methods are preferred for determining 100-year peak discharges 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 regional regression equations or analytical relationships. The estimated 100-year peak flows are then used to determine a culvert diameter large enough to handle the estimated peak flow and accommodate flood-associated wood and sediment.

Research conducted in northwestern California and the Pacific Northwest shows 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 culvert plugging. Other issues related to fish passage are covered elsewhere in the literature and also need to be considered in crossing design for fish-bearing streams.

Culvert diameters determined from estimated peak flows need to be checked in the field by making direct channel cross section measurements. The 3 times (3 X) bankfull stage method is suggested as one approach for field verification, but has only been validated for the rain-dominated North Coast region. Annual high flow line or active channel width measurements are alternatives for smaller or more entrenched channels where bankfull characteristics may be poorly developed.

Examples displayed in the appendix apply the watercourse crossing sizing techniques to a small tributary basin located in the Caspar Creek watershed near Fort Bragg, California. One-hundred year recurrence interval peak discharges are estimated, and wood passage concerns are addressed by sizing the culvert to fit the active channel width. Additionally, the various discharge-estimating techniques for ungaged basins are used to estimate a 10-year peak flow, and these results are compared to actual gaging station data. In this example, the direct flow transference method was found to provide the best estimate of the 10-year recurrence interval event. It is assumed that the techniques giving the best estimates for a 10-year event for this basin would also provide the best estimates of the basin's 100-year peak flow.

February 10, 2004

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Contents

Introduction

Timberland-owners and foresters are required by the Forest Practice Rules, as amended by the California State Board of Forestry and Fire Protection in July 2000, to design all new and reconstructed permanent watercourse crossings to accommodate an estimated 100-year flood flow, including wood and sediment loads (CDF 2003).[1] Recent hillslope monitoring work conducted throughout California has shown that problems most frequently occurred at watercourse crossings; inadequate design was cited as one of the reasons for these results (Cafferata and Munn 2002). While culverts are commonly sized to accommodate flood flows, studies in northwestern California show that flood discharge alone is usually not the primary cause of crossing failures (Furniss and others 1998; Flanagan, unpublished data, see 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 and others (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."

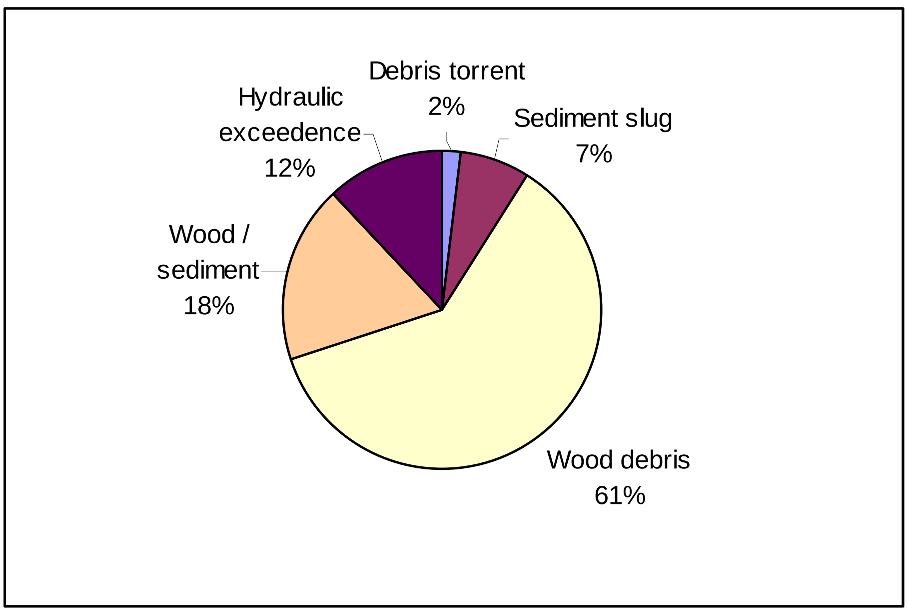

Figure 1. Failure mechanisms for culverts occurring along forest roads in northwestern California associated with storm events with recurrence intervals less than approximately 12 years (S. Flanagan, NOAA Fisheries, Arcata, CA, unpublished information; n = 57). Note that the specific distribution of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

[1] Currently the 100-year flood flow requirement is part of the Threatened and Impaired Watersheds Rule Package, which has an expiration date of December 31, 2006.

This paper presents three traditional office techniques for estimating 100-year recurrence interval water discharge: (1) the **rational method** (Chow 1964, Dunne and Leopold 1978, CDF 1983, Weaver and Hagans 1994), (2) the **USGS Magnitude and Frequency Method** (Waananen and Crippen 1977), and (3) **flow transference methods** (Waananen and Crippen 1977, Skaugset and Pyles 1991). A discussion of field techniques for evaluating proposed culvert diameters is also presented. A section on wood and sediment passage at crossings provides important information for preventing catastrophic crossing failure. Sections suggesting additional design considerations and approaches for evaluating the risk of existing crossings are also included. This report generally applies to non-fish bearing streams, since culvert sizing issues addressing fish passage are not included here. For detailed discussions of design criteria for fish passage, refer to USFS (2000), NMFS (2001), ODF (2002a), Flosi and others (2003), and WDFW (2003).

The California Forest Practice Rules specify that flood flows are to be estimated by empirical relationships between precipitation and watershed characteristics and runoff, and can be modified based on direct channel cross section measurements and local experience. The rational, USGS Magnitude and Frequency, and flow transference methods 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 stage, or where bankfull stage characteristics are difficult to determine, using the annual high flow line or active channel width.

While proper crossing design is critical for passage of water, sediment, and wood, the most important method for reducing environmental impacts is to locate roads to avoid or minimize crossings. Proper location of roads, and hence crossings, reduces chronic sediment impacts as well as 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 (M. Furniss, USFS-PNW, Corvallis, written communication, Furniss and others 2000).

Office Techniques for Determining Discharge

Rational Method

The rational method is an analytical approach for predicting peak runoff rates that has been used for engineering calculations for more than 100 years (Chow 1964, Portland Cement Association 1964, Dunne and Leopold 1978, Rossmiller 1980). Its development preceded the availability of long-term flow records that have become increasingly accessible in recent decades. 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 (cubic feet per second or cfs)
C	=	runoff coefficient[2]
I	=	rainfall intensity (inches per hour) for the 100-year storm
A	=	basin drainage area (acres)

To determine the rainfall intensity, one must: (1) determine the time of concentration for the drainage basin upstream of the watercourse crossing, (2) choose a 100-year return-period rainfall duration (e.g., 15 minutes, 30 minutes, etc.) from depth-duration-frequency (DDF) rainfall data that is similar in duration to the time of concentration, and (3) convert the 100-year return period DDF data to inches per hour for use as the rainfall intensity variable in the rational method discharge calculation. The time of concentration may be calculated using either the Kirpich Formula (Kirpich 1940, Weaver and Hagans 1994) or the Airport Drainage Formula (see Figure 7 in FAA 1970).[3] Both formulas and examples using the equations are presented in the Appendix. Based on past experience, 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 5 minutes are often unavailable (Yee 1994). With the Kirpich equation, the time of concentration is calculated from the channel length and elevation change from the top of the basin to the crossing, both of which can be obtained from topographic maps. The Airport Drainage equation incorporates the runoff coefficient (C), in addition to upstream watershed gradient and runoff distance, and generally produces longer estimates for the time of concentration.[4]

Short-duration rainfall-depth-duration-frequency data for 100-yr recurrence interval events (to determine rainfall intensity) are required once the time of concentration is known. Rainfall data are available for selected stations in California on microfiche cards or in tables or graphs from out-of-print California Department of Water Resources (CDWR) publications (CDWR 1976, 1981) and from the data set compiled more recently by Goodridge (2000).[5] According to

[2] The runoff coefficient is dimensionless because it represents the estimated proportion of rainfall that runs off. 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.
[3] An improved method for determining the time of concentration has been developed by Papadakis and Kazan (1987). This approach is a kinematic wave empirical equation specifically designed to determine the time of concentration for small rural watersheds and has been adapted by several recent hydrology manuals. While it is an improved method, the equation must be iteratively solved. Dr. Wopat will attempt to develop a spreadsheet to solve this equation, so it may be more easily used by field personnel.
[4] Yee (1994) recommends the use of the Airport Drainage equation to calculate the time of concentration.
[5] Copies of the Goodridge (2000) California weather CD ROM are available from Mr. Cafferata at CDF in Sacramento or Dr. Wopat at CGS in Redding.

CDWR staff, these data will be made accessible via the Internet at CDWR's Water Data Library (http://wdl.water.ca.gov/).

Determining the appropriate runoff coefficient (C) for the crossing site is very important when using the rational method. For 100-year flood flows on California's North Coast, Buxton and others (1996) suggest that the runoff coefficient should be 0.40.[6] Experience in the Redwood Creek watershed has led to the use of runoff coefficients ranging from 0.35 to 0.45 for 100-year discharge estimates, depending on terrain type (G. Bundros, RNSP, 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 ranging from 0.30 to 0.45, depending on the specific location of the crossing.**[7]

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) overland flow occurs, (3) the runoff coefficient is constant 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. These 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 frequently has been used for basins up to 640 acres. **We recommend that the rational method be limited to watersheds less than 200 acres.**

This method is easy to use, generally understood, and accounts for local conditions. Disadvantages include difficulty in obtaining rainfall intensity data for remote field sites, the assumptions listed above that are usually not met, little field validation to determine appropriate runoff coefficients for different parts of the state, and the inability of the method to account for rain-on-snow events. Detailed examples for use of the rational method (and other methods) are provided in Wopat (2003), CDF (1983), and the Appendix of this document.

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[6] This report provided the results of field tests on the rational method and other techniques made in southern Humboldt County during a large runoff event.

[7] Rossmiller (1980) lists the variables that have been used by one or more investigators to estimate the runoff coefficient (C). Table 1 in the current report only takes into account one factor—soil type. Caltrans' (2001) 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 yr 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.

Table 1. Values for rational 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

USGS Magnitude and Frequency Method

The USGS Magnitude and Frequency Method is based on a set of empirical equations derived from precipitation and runoff data collected at more than 700 stream gaging stations in California (Waananen and Crippen 1977). These records were analyzed to derive equations which were developed for 2, 5, 10, 25, 50, and 100 year recurrence intervals.[8] The equations for 100-year discharges for the six regions of California are as follows (see Figure 2 for the regional boundaries):

North Coast	$Q_{100} = 9.23\ A^{0.87}\ P^{0.97}$
Sierra	$Q_{100} = 15.7\ A^{0.77}\ P^{1.02}\ H^{-0.43}$
Northeast	$Q_{100} = 125\ A^{0.59}$
Central Coast	$Q_{100} = 19.7\ A^{0.88}\ P^{0.84}\ H^{-0.33}$
South Coast	$Q_{100} = 1.95\ A^{0.83}\ P^{1.87}$
South Lahontan-Colorado Desert	$Q_{100} = 1080\ A^{0.71}$

where:
- Q_{100} = predicted 100-year peak runoff event in cfs
- A = drainage area above the crossing in square miles
- P = mean annual precipitation in inches per year
- H = altitude index (average channel altitude) in thousands of feet (e.g., 2000 feet is 2.0)[9]

Drainage area and altitude index are reasonably easy to determine from topographic maps or newer GIS computer software tools. Mean annual precipitation is available from several sources, including: (1) isohyetal maps (such as Rantz 1972), (2) CD ROM (Goodridge 2000), (3) internet sites (Calwater planning watershed average annual precipitation is available from the

[8] Note that each equation encompasses a large, diverse geographic area (see Figure 2), and therefore is likely to overestimate discharge in some places and underestimate it in others.

[9] The altitude index should be computed as the average of the altitudes at the 10 and 85 percent points along the main channel from the crossing to its hydrologic divide, or alternately as the average altitude between the highest point in the basin and the crossing ($[H_{max} + H_{min}]/2$). However, Magnitude and Frequency discharge estimates for bridged crossings should use only the altitude index determined from the 10 and 85 percent points along the main channel because the alternative method of determining the altitude index ($[H_{max} + H_{min}]/2$) increasingly underestimates Q_{100} as the watershed area increases.

CDF FRAP site at: http://frap.cdf.ca.gov/projects/esu/esulookup.asp; additionally tabular precipitation data is available for numerous northern California stations at: http://www.wrcc.dri.edu/summary/climsmnca.html from the Western Regional Climate Center), and (4) local records.[10]

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. The primary disadvantages are that it generalizes vast regions of the state, resulting in overestimation in some areas and underestimation in other areas, and that the equations have not been updated since 1977. **The USGS Magnitude and Frequency Method is preferred over the rational method for drainage areas larger than 100 acres.** 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. The minimum drainage areas used to generate the regression equations, along with other information for the 100-year discharge regression equations, are displayed in Table 2.

The USGS Magnitude and Frequency Method regression equations are used for the National Flood Frequency (NFF) program in California. NFF 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 and Crouse 2002; see the following websites for more information on NFF and the computer software that can be downloaded: http://water.usgs.gov/software/nff.html; http://www.fema.gov/fhm/ot_main.shtm). Use of the program allows 100-year peak flow estimates to be generated along with standard error estimates.

Table 2. USGS Magnitude and Frequency Method 100-year regression equation information (Waananen and Crippen 1977).

Region	**Minimum Drainage Area (ac)**	Maximum Drainage Area (ac)	# of Stations used in the Analysis	Std Error of Estimate ($\log_{10}$ units)[11]
North Coast	**83**	1,992,320	125	0.26
Sierra	**90**	5,772,800	212	0.37
Northeast	**38**	15,872	20	0.45
Central Coast	**109**	2,659,840	91	0.41
South Coast	**96**	412,160	137	0.39
South Lahontan-Colorado Desert	**6**	16,000	35	0.36

[10] Mean annual precipitation and rainfall-depth-duration-frequency tables for the Redwood Creek basin and a culvert sizing program are available from Redwood National and State Parks (RNSP). Contact Mr. Bundros at RNSP, Arcata, CA, for further information.
[11] To use the standard error estimate (SEE), obtain the Q_{100} discharge estimate, convert it to $\log_{10}$ 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.

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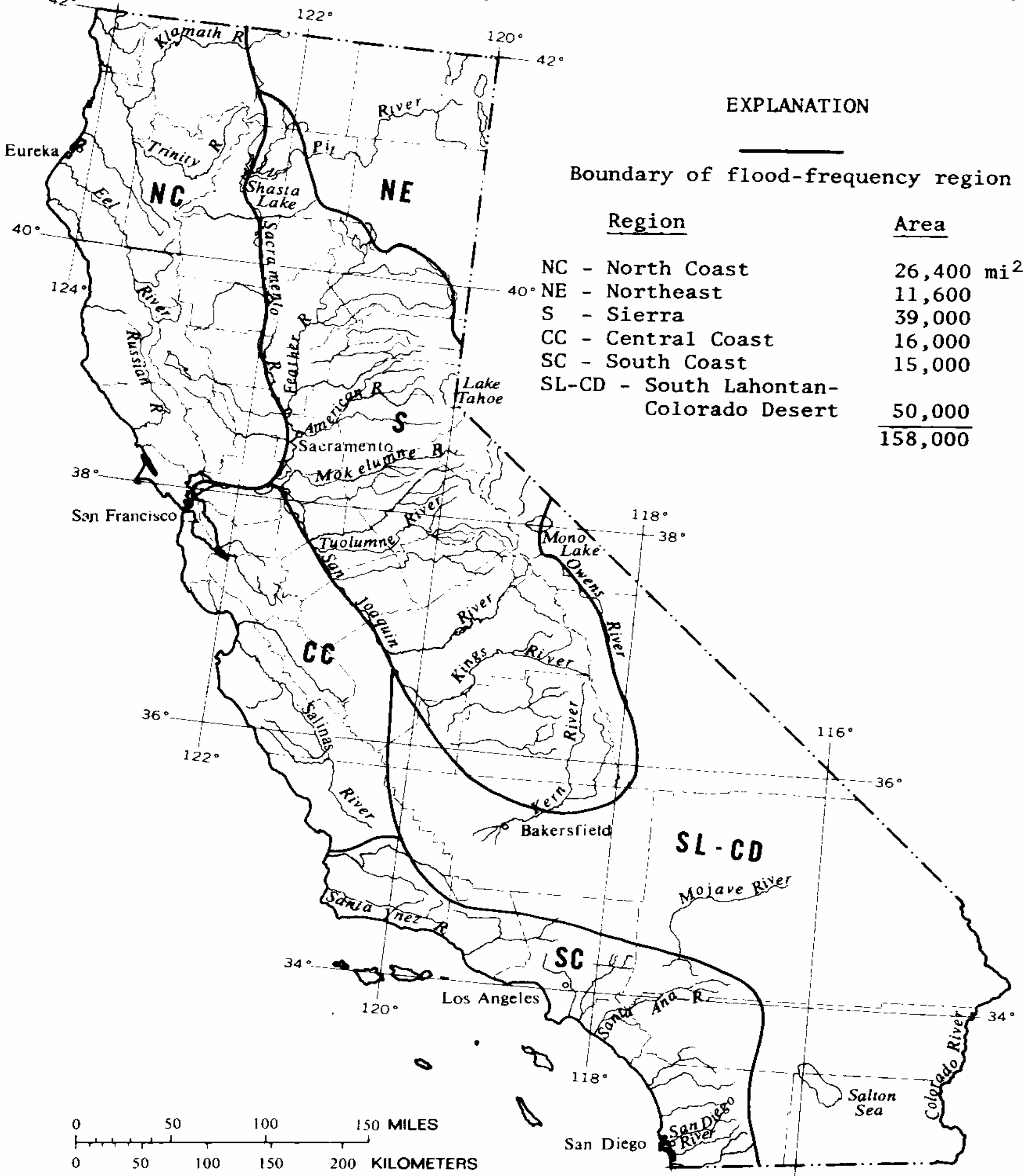

FIGURE 3.--Flood-frequency regions used in this study.

Figure 2. USGS Magnitude and Frequency Method regression equation regions (from Waananen and Crippen 1977).

Flow Transference Methods

If a stream gaging station is located on the same stream as the proposed crossing site or on a hydrologically similar nearby stream, it is possible to adjust the 100-year discharge 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 discharge at ungaged site (cfs)
Q_{100g}	= 100-year discharge at gaged site (cfs)
A_u	= drainage area of ungaged site (mi^2)
A_g	= drainage area of gaged site (mi^2)
b	= exponent for drainage area from the appropriate USGS Magnitude and Frequency equation (e.g., 0.77 for the 100-yr equation for the Sierra Region—see the equations above for the exponents for the other regions)

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 discharge estimate for the gaged station must be known. This can be determined relatively easily for USGS gaging stations through the use of PEAKFQ, a computer software program available online that performs a flood-frequency analysis based on Bulletin 17B, which is the accepted methodology published by the Interagency Advisory Committee on Water Data (IACWD 1982), and is available from the USGS website at: http://water.usgs.gov/software/peakfq.html. Q_{100} can also be determined from Waananen and Crippen (1977) for the stations they used in their regression analysis (see Table 5 in their report), or by manually calculating Q_{100} with a flow 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 when the criteria listed above are met (i.e., the stream gaging station is nearby and the available stream gaging annual peak discharge records are adequate). **The flow transference method is preferable to the USGS regional regression equations because local data are more likely to represent the drainage-basin characteristics in terms of slopes, geology, soils, and climate when compared to the more general regional equations.**

An alternate approach to the Waananen and Crippen (1977) flow transference approach can be used if the gaged and ungaged watersheds 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 differences in watershed area:

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

Field Techniques for Evaluating Proposed Culvert Diameters

Following the office calculation of 100-year flood discharges, the determination of the required pipe diameter is often made through the use of a culvert sizing nomograph (see Figure 12 or Normann and others 1985, and assume inlet control when using these nomographs).[12] It is critical to specify an appropriate headwater depth to pipe diameter ratio (HW/D) when making this calculation. The HW/D ratio used should be no more than 0.67 (previous crossing design documents usually specified a HW/D ratio of 1.0) (M. Furniss, USFS-PNW, Corvallis, written communication). A reduced HW/D ratio lowers the potential for plugging associated with pieces of wood. The proposed pipe diameter can then be field checked using either: (1) bankfull stage, (2) the annual high flow line, or (3) the width of the active stream channel in the vicinity of the crossing.

The **3 X bankfull stage method** is a potential field check (BC MOF 1995, 2002) that appears to be valid for the coastal portions of northwestern California, but may underestimate Q_{100} culvert sizes for inland areas away from the rain-dominated portion of the Coast Range. This procedure assumes that: (1) the bankfull scenario of any stream represents the mean annual flood cross-sectional flow area for the stream (Q_2)[13]; (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. These assumptions are not truly representative of all situations, but within the accuracy expected for establishing design discharge, this method should be acceptable for verifying proposed stream-culvert sizes smaller than 78 inches on forest roads in counties along the coast of northern California (BC MOF 1995, 2002).

To utilize the 3 X bankfull stage technique, measure the bankfull cross section allowing for scour in a representative stream reach that is not influenced by a road. In unconfined stream channels, bankfull stage 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). 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 3). Measure the depth of the stream at several spots across the opening to obtain the average depth (D) in feet.

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[12] A culvert that has a slope greater than 1.5% to 2% will normally exhibit inlet control (Beschta 1984, Piehl and others 1988). Normann and others (1985) provide nomographs for determining flow capacity for both round pipe culverts and other types of stream crossing structures.
[13] Q_2 is actually the median annual flood; mean annual flood is more often approximately a $Q_{2.5}$ recurrence interval event (R. Beschta, Oregon State Univ., Corvallis, written communication).

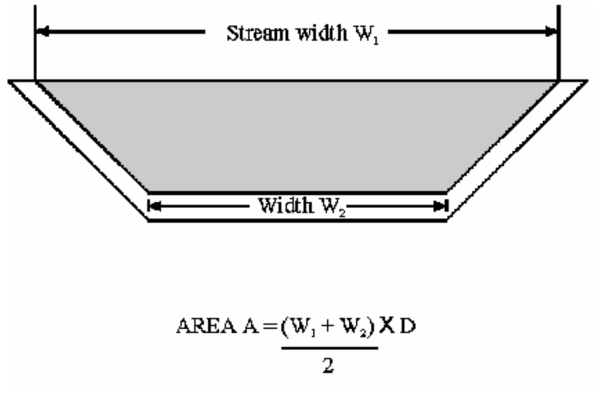

Figure 3. Diagram illustrating how to determine bankfull channel area (from BC MOF 1995).

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:

$$\pi\ r^2 = 3\ A_{bf}$$
$$r^2 \approx A_{bf}$$ (note that this is approximate)
$$r \approx (A_{bf})^{1/2}$$
$$\mathbf{d \approx 2[(A_{bf})^{1/2}]}$$

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

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

Any evidence from major storms must be accommodated with this method. 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.[14] In addition to the need to accommodate streamflow, wood and sediment passage must also be considered. **The 3 X bankfull stage method works best for pipe sizes up to 48 inches (G. Bundros, RNSP, Arcata, unpublished information), and it is not applicable to culverts greater than 78 inches in diameter (BC MOF 1995, 2002).**

[14] Major storm events have a recurrence interval of 10 to greater than 100 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.

The 3 X bankfull stage method uses on-site field conditions, is easy to use, provides the culvert diameter directly, and offers an easy field check of office calculations for 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.[15] 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 is acceptable to approximate bankfull stage with the annual high flow line (M. Furniss, USFS-PNW, Corvallis, written communication). Another approach for these types of small channels is to simply make the culvert diameter equal to the active channel width (W_2) at the crossing location.[16]**

Other limitations of the 3 X bankfull stage method include: (1) cross-sections measured should be representative of the channel in the general crossing area and not be affected by roads, (2) the identification of bankfull stages in severely impacted channels is difficult, especially when accumulations of large wood and sediment are present in the channel, and (3) while some field verification of this method has occurred in northwestern California[17] (Figure 4), virtually none has taken place in interior areas of California, so it may not be valid in inland areas away from the coast.

To illustrate the last point, Beckers and others (2002) in reviewing the 3 X bankfull stage method proposed by the BC MOF (1995, 2002) found that the ratios of 100-year stream discharge to 2-year discharge (Q_{100}/Q_2) vary substantially with basin area and climate.[18] For flood peaks generated by rainfall and rain-on-snow 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

[15] 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 stages.

[16] 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.

[17] 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 harder geologic units where bankfull stages may be hard to discern. More testing of this approach is needed.

[18] 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.

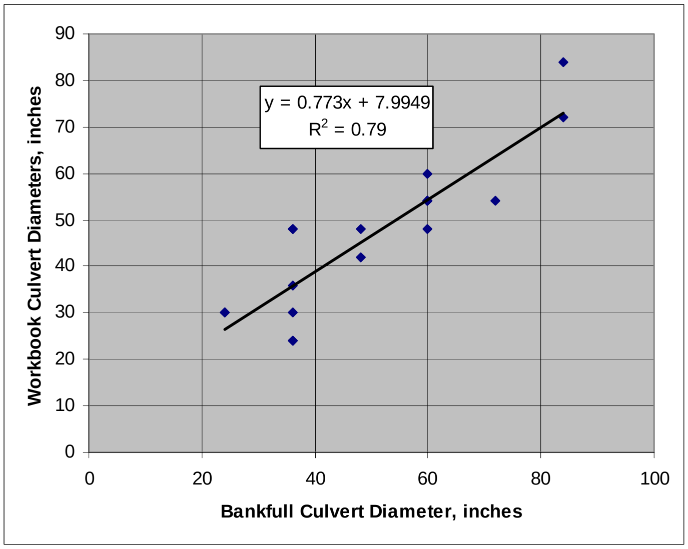

Figure 4. Plot of 3 X bankfull stage determined culvert diameters for drainage areas less than 200 acres (x axis) vs. culvert diameters determined by a workbook spreadsheet (y axis) using either the rational method (for drainage areas less than 80 acres and a runoff coefficient of 0.40) or USGS Magnitude and Frequency Method (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 workbook-estimated 100-year return interval flood flows using a culvert sizing nomograph (for example, see Figure 12), and assuming a projecting pipe entrance and HW/D = 1.0 (unpublished data collected by Greg Bundros, RNSP, Arcata, CA).

mean annual flood. 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 six large floods over 60 years in the Sierra Nevada 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 = 3.65, n = 6), through the Sierra FFR (avg. Q_{100}/Q_2 = 5.39, n = 6).[19] The increase in the 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 stage method to

[19] 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 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.

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 stage 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 stage method and may be larger.

Wood and Sediment Passage at 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. Wood and sediment passage are often of equal or greater concern than hydraulic capacity for preventing culvert failure (Flanagan 1996, see Figure 1). Furniss and others (1998) provide advice on crossing design to accommodate wood and sediment passage. Unfortunately, it remains difficult to directly predict the loading of sediment and wood at a given crossing, but we can design crossings to better accommodate these watershed products and reduce the risk of failure.

How watershed products such as wood and sediment are processed at the pipe inlet is what determines plugging potential, and thus the actual culvert capacity. **For example, flared metal end sections are relatively inexpensive, easy to retrofit, and yield large gains in capacity for all watershed products.** Additionally, they appear to prevent the lodging of rocks and woody debris at the inlet lip (M. Furniss, USFS, Pacific Northwest Research Station, Corvallis, OR, written communication; AISI 1971).

Furniss and others (1998) describe several additional techniques for increasing the capacity of culverts for 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)[20], (2) utilizing culverts that are as wide, or nearly as wide, as the stream's active channel width (i.e., the zone of active, annual streambed scour and deposition)—particularly for small streams, (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 5). Additionally, a single large culvert at a crossing is better for wood passage than several small ones (Furniss and others 1991). Beschta (1984) reported that: (1) various types of structures, commonly denoted as "trash racks", can be constructed upstream of the inlet to help prevent plugging by wood, but winter maintenance of these structures is critical for success; (2) organic debris can create continual maintenance problems when the culvert diameter is too small to freely pass floatable wood; and (3) culvert slopes of less than 3 percent may be prone to bedload sediment accumulation and reduced efficiency.

[20] Note that most previous guidelines (e.g., CDF 1983) specified a maximum HW/D ratio of 1.0.

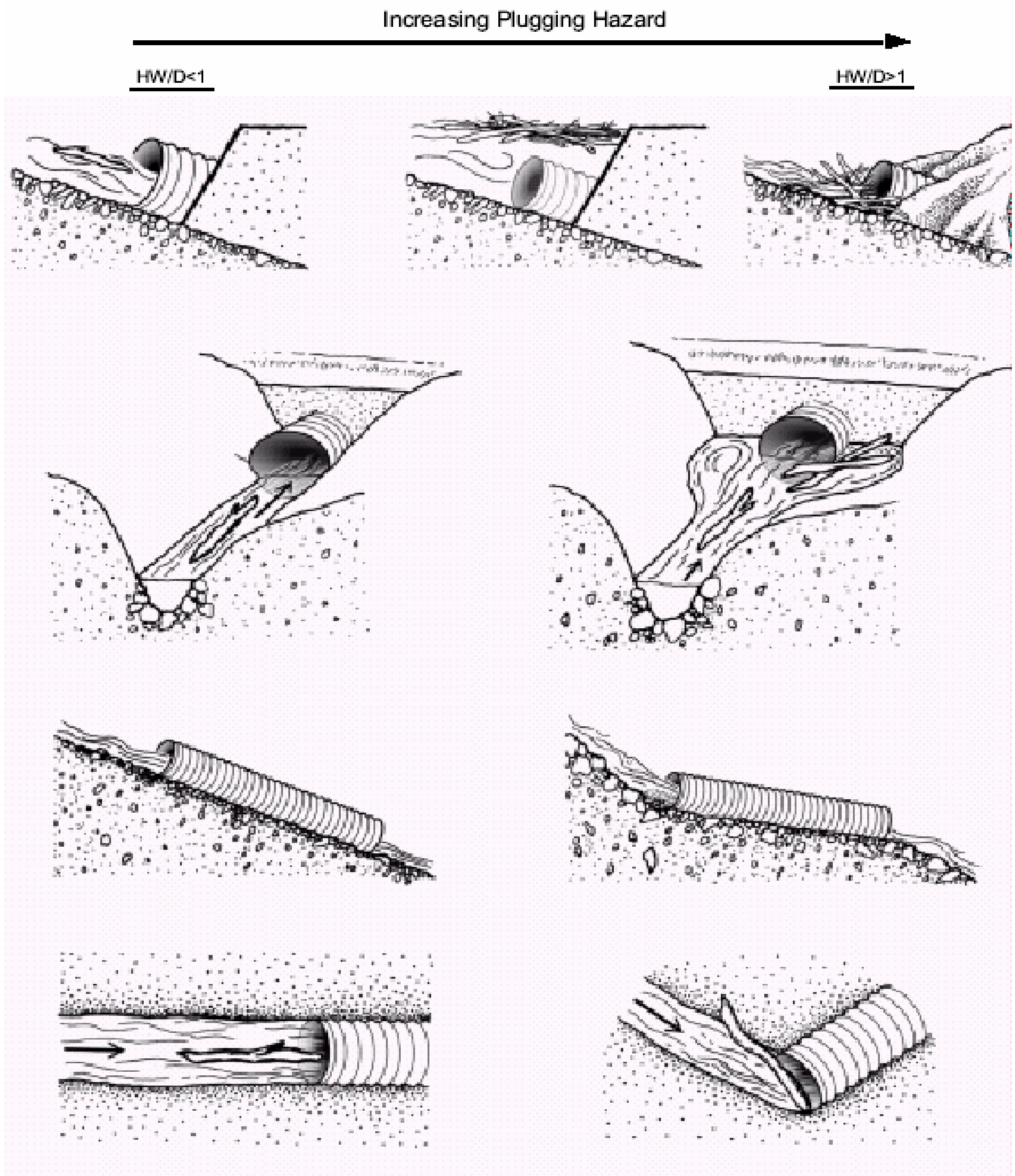

Figure 5. Reducing the probability of culvert failure due to woody debris and sediment involves not only careful consideration of culvert diameter, but configuration of the installed pipe as well. From top to bottom in the above figure, culverts should: (1) not pond water, (2) not create unusually wide areas near the inlet, (3) maintain channel grade, and (4) be placed on the same alignment as the natural stream channel (from Furniss and others 1998).

Recently conducted studies 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 and others 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 1996, Flanagan and others 1998).

For these more frequent storms, the dominant failure mechanism is wood accumulation at the inlet and typically the type of wood causing failures is small (i.e., twigs, sticks, and branches), not large logs. Pieces of wood initiating plugging are usually not much longer than the culvert diameter and often do not exceed the width of the channel (Figure 6). As stated above, culvert sizing should be driven by 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. For example, when a sample of culverts in northwestern California were sized for the 100-year peak flow, the resulting pipe diameters were, on average, only about two-thirds the channel width (i.e., culvert diameter/channel width $\approx$ 2/3). However, if the culvert is sized for wood passage (i.e., pipe is approximately equal to 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 that creates ponded conditions at the inlet (see Figure 5).

Additional Design Considerations to Reduce the Risk of Crossing Failure

Other elements can be incorporated into stream crossing design that can reduce the risk of crossing failure and potential impacts to watercourses if crossings fail. Proposed crossings should be adjusted to fit all of the field conditions present. For example, the height of fill that will exist above a culvert should be accounted for when determining the appropriate pipe diameter. As a rule of thumb, the pipe diameter should be increased by 6 inches for every 5 feet of fill above the pipe on the discharge side of the crossing.[21] For example, a pipe that is initially sized at 36 inches and would be covered by 10 feet of fill on the downstream side should be increased to 48 inches to reduce the risk of crossing failure and the potential discharge of a large amount of sediment into the stream if the culvert plugs. This approach also reduces the need for replacement of a failed crossing that would be relatively expensive compared to the cost of a slightly larger diameter pipe. It is also important to have crossing fill material adequately compacted so that overtopped pipes will have only a small part of their fill removed.

[21] This recommendation is based on the personal observations of Mr. Spittler, CGS, Santa Rosa.

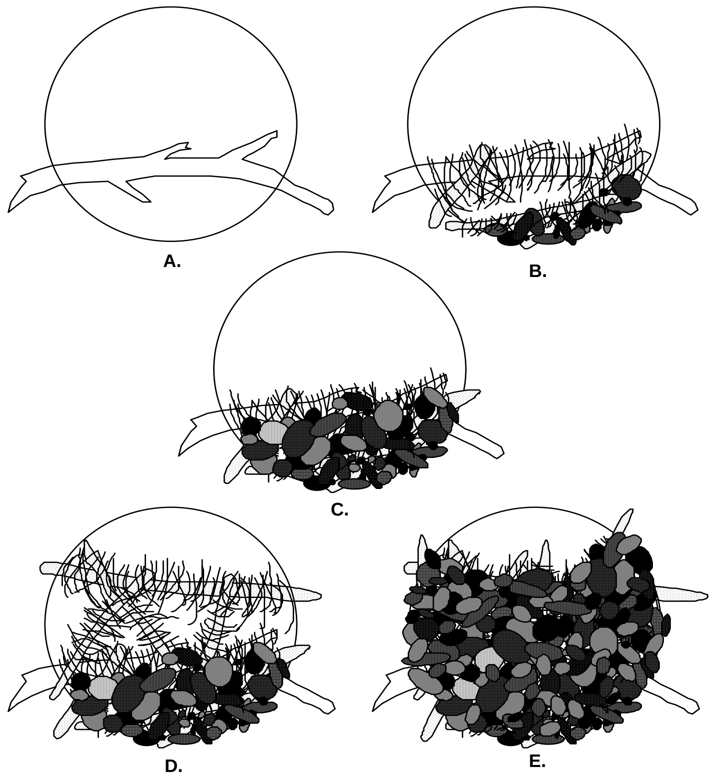

Figure 6. Plugging of culverts by wood is usually initiated by a single piece lodging across the inlet (a). This piece becomes a locus for the accumulation of detritus and sediment (b). As the plug grows, sediment and detritus seal off a portion of the inlet (c). The initiation process may be repeated with a second piece, allowing the plug to grow upwards (d and e) (Flanagan 1996).

Minimizing the amount of crossing fill and constructing a crossing without diversion potential can significantly reduce sediment impacts if a crossing were to fail (Furniss and others 1997). Constructing a crossing with no diversion potential is more cost and time effective than having to continually maintain a crossing in order to maintain flow and prevent a stream diversion. Waterbars rarely prevent stream diversions when culverts plug during a large storm and they also require long-term maintenance.

In contrast, a broad overflow dip (critical 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 overflow dip may be constructed to discharge either at the intersection of the crossing fill with the valley wall, or over the fill face in an armored spillway. The design and construction of the critical dip discharge is important to reduce the potential for overtopping flows to erode the crossing fill (Weaver and Hagans 1994). The California Forest Practice Rules [14 CCR § 923.3(f), 943.3(f), 963.3(f); 923.4(n), 943.4(n), 963.4(n)] require that permanent watercourse crossings are constructed or maintained to prevent diversion of stream overflow down the road should the drainage structure become plugged.

Road surface runoff and surface erosion need to be considered in the crossing and approach design to minimize the potential for sediment delivery to the watercourse being crossed. Where the road surface is outsloped or flat as it approaches and crosses the crossing fill, the downslope discharge area will need to be designed for prevention of surface erosion. Where the road surface is insloped, the drainage ditch needs to be designed to avoid eroding and/or discharging fine-grained sediment into the crossing inlet. Additional erosion control structures, such as rolling dips, water bars, or cross-drains, placed close to crossing fills are useful in minimizing the amount of surface runoff from the approach and sediment delivery to the watercourse. A filter area of vegetation should be established between the ditch relief culvert outlet (or other drainage structure) and the stream channel to catch the sediment from relief-culvert discharge 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 should be removed before new culvert placement. 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 gradient should be consistent with the natural 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. A minimum diameter of 24 inches is recommended for watercourse crossings in channels that receive flood flows (i.e., not crossings receiving discharge solely from small springs).

Rocked-lined fords are often a better replacement alternative than culverts for small headwater channels, particularly where winter maintenance is difficult and/or debris flows are likely (Spittler 1992, Warhol and Pyles 1989). Because natural stream bottoms better facilitate fish passage relative to the bare metal of culverts, bridges and other natural-bottomed watercourse crossing structures, such as arches and culverts buried with at least 20 percent of their diameter in the channel, should be installed in fish-bearing channels where standard culverts previously existed, rather than reinstalling new culverts at grade. ODF (2002b) provides guidance on how to determine the flow capacity of short and long-span bridges, as well as open-bottom pipe arch structures.

Evaluating Existing 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 recently completed on Timber Harvesting Plans (THPs) throughout California on non-federal commercial timberlands suggest that numerous existing crossings are at high risk for failure, with frequent watercourse crossing problems documented related to culvert plugging, stream diversion potential, fill slope erosion, scour at the outlet, and ineffective road surface drainage immediately above crossings (Cafferata and Munn 2002). About five percent of the randomly selected THPs evaluated from 1996 through 2001 had one or more catastrophic crossing failures present. Similarly, Bundros and others (2003) classified 20 percent 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 medium or higher plugging potential.

Crossing inventories are an important component of a road management plan that aims to reduce sediment yield to watercourses, as well as prevent road damage (see Flanagan and others (1998), Flanagan and Furniss (1997) for additional information). Examples of items to consider as part of a crossing inventory include:

Crossings at high risk from wood-related plugging

- culvert diameter divided by active channel width is less than 0.7[22]
- poor pipe alignment with the stream channel
- HW/D ratio is greater than 1.0
- unusually wide areas, including sediment basins, near the inlet of the pipe

Crossings at high risk for sediment blockage

- culvert gradient is less than 3 percent
- culvert gradient is less than natural stream channel gradient

Crossings at high risk for hydraulic capacity exceedance

- existing pipe capacity has less than 100-year flow capacity
- crushing and plugging of the pipe inlet is present
- evidence of insufficient hydraulic capacity is present. Examples include:
    - floodplain-like deposits of sediment immediately upstream of the crossing
    - evidence of overtopping of the crossing by peak flows

Crossings at high risk for causing significant gullying

- diversion potential exists (the road grade through the crossing is such that a stream will flow down the road and leave its natural channel if the culvert plugs or its capacity is exceeded)

Crossings in need of replacement due to age-related deterioration

- the length of time the culvert has been installed[23]
- moderate or high degree of corrosion

[22] Research conducted in northwestern California showed that culverts sized at 0.7 times the mean stream bed width will pass, on average, 95% of fluvially transported wood greater than 12 inches long (Flanagan 1996).
[23] The service life of a culvert varies depending on local corrosion rates, but culverts generally last at least 25 years (Pyles and others 1989).

Crossing with fish passage limitations [design criteria for fish passage are described in USFS (2000), NMFS (2001), ODF (2002a), Flosi and others (2003), and WDFW (2003)]

- outlet is elevated greater than 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 inventory, a schedule should be developed and funding secured to make needed corrections.

Conclusions

Several office techniques, based on empirical relationships between precipitation and watershed characteristics and runoff, are available to determine an estimated 100-year discharge. However, these 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 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.

Culvert sizes specified as part of a permitted project in California, such as a THP, 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. Spreadsheets are available for calculating discharges for the rational and USGS Magnitude and Frequency methods which can be cited in the plan (Figure 7).[24] This level of information assists both agency review of plans and reduces the need for follow-up questions.

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 and when they do fail.

Acknowledgements

Dr. Robert Ziemer, Chief Research Hydrologist (retired), USDA Forest Service, Pacific Southwest Research Station, Arcata; Dr. Lee MacDonald, Professor, Department of Earth Resources, Colorado State University, Fort Collins; and Dr.

[24] Contact Dr. Wopat at CGS for a copy of the Excel spreadsheet he has developed for calculating discharge with the rational and USGS Magnitude and Frequency methods. Additionally, Moore and others (1999) provide a review of existing software tools available for culvert design and analysis.

William Trush, Adjunct Professor, Humboldt State University, and Principal, McBain and Trush, Arcata, reviewed an earlier draft and provided helpful comments. Dr. Robert Beschta, Professor Emeritus, Oregon State University, Corvallis; John Munn, Soil Erosion Studies Project Leader, California Department of Forestry and Fire Protection (CDF), Sacramento; Tim Robards, State Forests Research Coordinator, CDF, Sacramento; and Gary Rynearson, California State Board of Forestry and Fire Protection, reviewed the second draft. Michael Furniss, Hydrologist/Soil Scientist, USDA Forest Service, Pacific Northwest Research Station, Corvallis, and Dr. Carlton Yee, Emeritus Professor of Forestry at Humboldt State University, Arcata, reviewed both the first and second drafts. Clay Brandow, Watershed Specialist, CDF, Sacramento, measured the cross sectional area of North Fork Caspar Creek subwatershed HEN and offered valuable comments on the second draft. Dr. Mary Ann Madej, Adjunct Professor, Humboldt State University, and Geologist, U.S. Geological Survey, Redwood Field Station, Arcata, provided the statistical analysis for the 3 X bankfull stage analysis.

Determination of 100-Year Flood Flow

Location: Coffee Creek, Trinity County, CA

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

Magnitude and Frequency Method for 100-year flood flow

								100-yr flood flow Q_{100} (cfs)			
No.	Crossing	Area (acres) A	Basin maximum elevation (ft)	Crossing elevation (ft)	Area (mi^2) A	Precipitation (in/yr) P	Elevation (ft/1000) H	North Coast(1) (NC)	Sierra(2) (S)	North-east(3) (NE)	Central Coast(4) (CC)
1	*A (real)*	412.5	7215	2800	0.645	58.38	5.0075	***325.5***	***354.6***	***96.5***	***239.5***
2	*B (hypothetical)*	25	3300	3050	0.039	58.38	3.175	***28.4***	***49.8***	***18.5***	***23.6***
3					0.000		0	***0.0***	***#DIV/0!***	***0.0***	***#DIV/0!***
4					0.000		0	***0.0***	***#DIV/0!***	***0.0***	***#DIV/0!***
5					0.000		0	***0.0***	***#DIV/0!***	***0.0***	***#DIV/0!***
6					0.000		0	***0.0***	***#DIV/0!***	***0.0***	***#DIV/0!***
7					0.000		0	***0.0***	***#DIV/0!***	***0.0***	***#DIV/0!***
8					0.000		0	***0.0***	***#DIV/0!***	***0.0***	***#DIV/0!***
9					0.000		0	***0.0***	***#DIV/0!***	***0.0***	***#DIV/0!***
10					0.000		0	***0.0***	***#DIV/0!***	***0.0***	***#DIV/0!***

See below for M&F equations

Rational Method for 100-year flood flow

		$T_c = 60((11.9 \times L^3)/H)$^0.385			$Q_{100} = CIA$			
No.	Crossing	Channel length (to top of basin) (mi) L	Elevation difference (ft) H	Concentration time (min) Tc	Runoff coefficient C	Precipitation (in/hr) I	Area (acres) A	100-yr flood flow (cfs) Q100
1	*A (real)*	1.97	4415	*13*	0.4	3.04	412.5	***501.6***
2	*B (hypothetical)*	0.13	250	*2*	0.4	3.04	25	***30.4***
3	***0***		0	*#DIV/0!*			0	***0.0***
4	***0***		0	*#DIV/0!*			0	***0.0***
5	***0***		0	*#DIV/0!*			0	***0.0***
6	***0***		0	*#DIV/0!*			0	***0.0***
7	***0***		0	*#DIV/0!*			0	***0.0***
8	***0***		0	*#DIV/0!*			0	***0.0***
9	***0***		0	*#DIV/0!*			0	***0.0***
10	***0***		0	*#DIV/0!*			0	***0.0***

Magnitude & Frequency Q_{100} equations

NC (1) $Q_{100} = 9.23\ (A)^{0.87}\ (P)^{0.97}$

S (2) $Q_{100} = 15.7\ (A)^{0.77}\ (P)^{1.02}\ (H)^{-0.43}$

NE (3) $Q_{100} = 125\ (A)^{0.59}$

CC (4) $Q_{100} = 19.7\ (A)^{0.88}\ (P)^{0.84}\ (H)^{-0.33}$

Template prepared by:
Michael Wopat
California Geological Survey
Redding, CA 96002
June 14, 2001

q100 calcs tmplt.xls, Sheet1 | Page 1 of 1 | 2/23/2003 3:03 PM

Figure 7. Spreadsheet available for determining estimated water discharge associated with a 100-year recurrence interval event using either the rational method or the USGS Magnitude and Frequency Method (Wopat 2003; developed by and available from M. Wopat, CGS, Redding, CA).

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Appendix -- Examples of Watercourse Crossing Sizing Methods

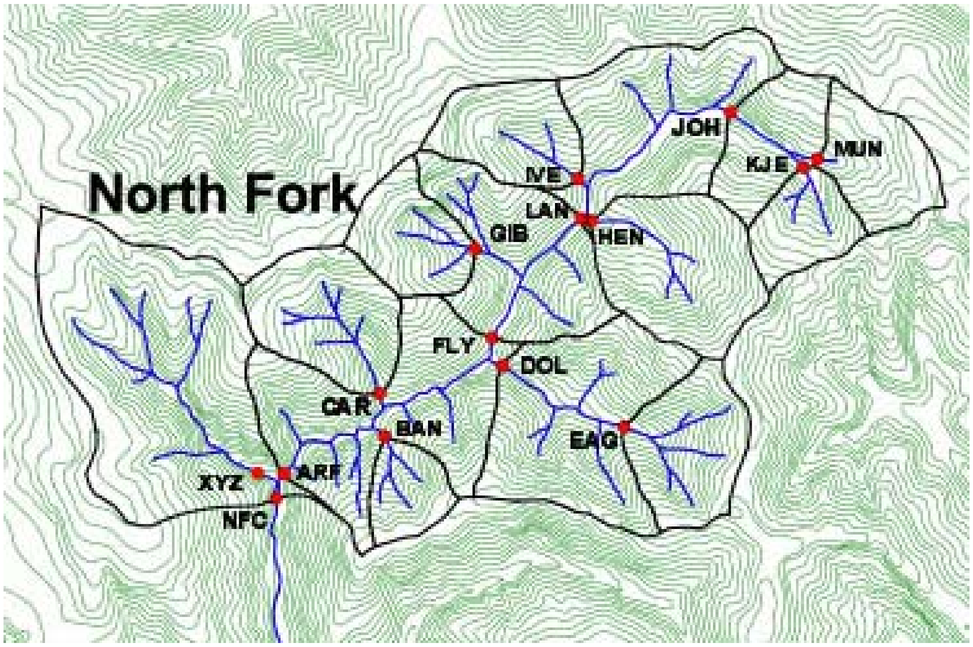

Figure 8. North Fork Caspar Creek Watershed (1168 acres), and control subwatershed HEN (96 acres) (from USFS-PSW Redwood Sciences Laboratory webpage).

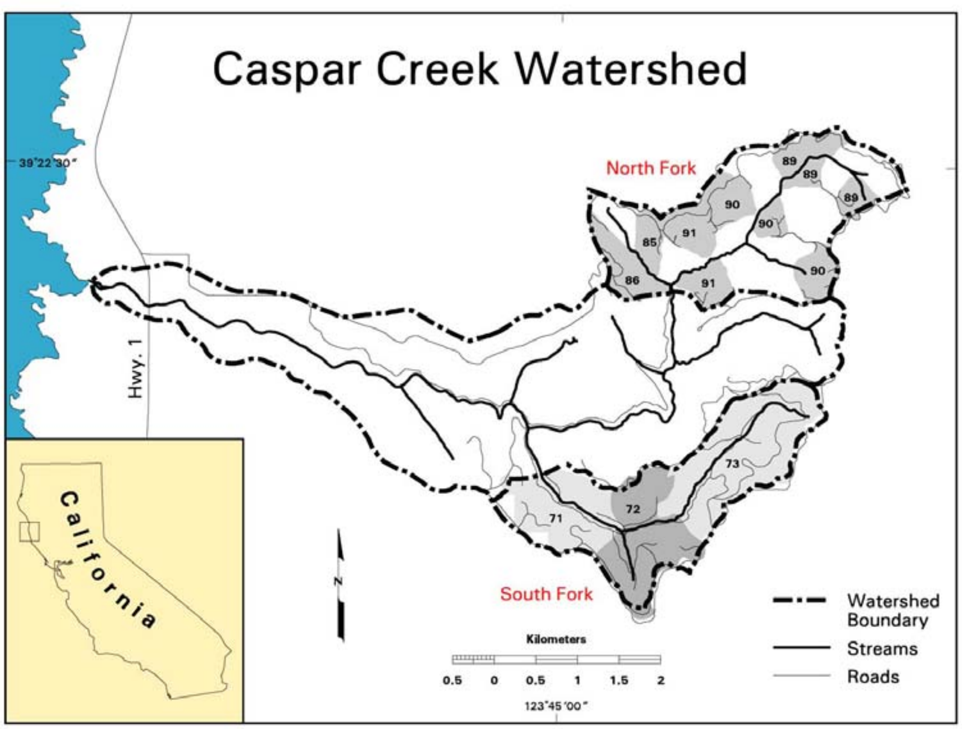

Figure 9. Location map of the entire Caspar Creek watershed (from USFS-PSW Redwood Sciences Laboratory webpage).

Part A. Predicting the 100-Year Recurrence Interval Discharge for Caspar Creek Subwatershed HEN (see Figures 8 and 9)

Rational Method

Known Information:

Drainage area (A) = 96 acres (Henry 1998) for HEN
100 yr 15 minute NF Caspar Creek rainfall = 0.76 inches/15 minutes (Goodridge 2000)
100 yr 30 minute NF Caspar Creek rainfall = 1.02 inches/30 minutes (Goodridge 2000)
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:

$Q_{100} = CIA$

Time of Concentration (using the Kirpich Formula):

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

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 hours or 6 minutes
T_c = 6 minutes. 15-minute rainfall-depth-duration-frequency data from Goodridge (2000) was used because 10-minute data was not available
0.76 inches/15 minutes x 60 minutes/hour = 3.04 inches/hour
I = 3.04 inches/hour
C = 0.3 (loam soil, Table 1)
Q_{100} = 0.3 x 3.04 inches/hour x 96 acres
Q_{100} = 87.6 or 88 cfs
Pipe diameter = 54 inches (assumes HW/D = 1.0 and projecting pipe)
Pipe diameter = 69 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 minutes, or approximately 30 minutes
1.02 inches/30 minutes x 60 minutes/hour = 2.04 inches/hour
I = 2.04 inches/hour
C = 0.3 (loam soil, Table 1)
Q_{100} = 0.3 x 2.04 inches/hour x 96 acres
Q_{100} = 58.8 or 59 cfs
Pipe diameter = 46 inches (assumes HW/D = 1.0 and projecting pipe)
Pipe diameter = 60 inches (assumes HW/D = 0.67 and projecting pipe)

USGS Magnitude and Frequency Method[25]

Known Information:

$A = 0.15$ miles^2

$P = 46.85$ inches/year (Henry 1998)

Calculate:

$Q_{100} = 9.23\ A^{0.87}\ P^{0.97}$

$Q_{100} = 9.23\ (0.15)^{0.87}\ (46.85)^{0.97}$

$Q_{100} = 74$ cfs

Pipe diameter = <u>51 inches</u> (assumes HW/D = 1.0 and projecting pipe)

Pipe diameter = <u>65 inches</u> (assumes HW/D = 0.67 and projecting pipe)

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 367.1 cfs (using USGS PEAKFQ program)

Calculate:

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

$Q_{100u} = 367.1$ cfs (96 acres/1168 acres)$^{0.87}$

$Q_{100u} = 42$ cfs

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

Pipe diameter = <u>52 inches</u> (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 367.1 cfs

Calculate:

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

$Q_{100u} = 367.1$ cfs (96 acres/1168 acres)

$Q_{100u} = 30$ cfs

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

Pipe diameter = <u>45 inches</u> (assumes HW/D = 0.67 and projecting pipe) (see Figure 12 for an example of using the culvert sizing nomograph for a discharge of 30 cfs)

3 X Bankfull Stage Method (see Figures 3 and 11)

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

Average channel depth at HEN is 0.95 feet

Average bankfull stream channel width (W_1) at HEN is 5.6 feet

Average active stream channel width (W_2) 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 feet^2

Calculate:

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

$D = 2[(4.75\ \text{feet}^2)^{½}]$

Pipe diameter (D) = 4.35 feet x 12 = <u>52 inches</u>

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25 The USGS National Flood Frequency Program (NFF, Version 3.2, available at http://water.usgs.gov/software/nff.html) uses the USGS Magnitude and Frequency equations to estimate flood flows in California. NFF shows there to be a standard error (SE) of 66% (= 49 cfs) for the 74-cfs Q_{100} estimate. $Q_{100} \pm 1$ SE = 74 cfs $\pm$ 49 cfs, resulting in a $\pm$ 1-SE range of 25 cfs to 123 cfs. Because the range $\pm$ 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 $Q_{100} \pm 1$ SE, that is, between 25 cfs and 123 cfs.

Active Channel Width Method

Known Information:

Average channel width above HEN is 4.4 feet (use W_2 width)

Calculate:

culvert diameter/channel width = 1.0
culvert diameter = 1.0 x channel width
culvert diameter = 1.0 x 4.4 feet
Pipe diameter (D) = 4.4 feet or 53 inches

Flow Frequency Analysis Method

Known Information:

Table 3. Annual peak discharges for station HEN from water years 1986 through 2003.

Year	Peak Q (cfs)	Rank
1986	12.8	6
1987	4.3	15
1988	7.6	11
1989	5.0	14
1990	12.3	7
1991	1.6	17
1992	4.3	15
1993	17.1	1
1994	4.1	16
1995	14.8	4
1996	11.6	8
1997	15.6	3
1998	13.0	5
1999	16.5	2
2000	6.6	12
2001	5.8	13
2002	10.1	10
2003	11.1	9

Calculate:

Table 4. Estimated discharges for various recurrence intervals (RIs), including the 10-year RI discharge (used in Appendix B) and the 100 year RI discharge (discharges estimated by the USGS PEAKFQ program).

RI (yr)	Q (cfs)	95% Confidence Limits Lower	Upper
2	9.0	7.3	11.2
5	13.7	11.0	18.5
10	**16.8**	**13.3**	**24.0**
25	20.6	15.8	31.3
50	23.4	17.5	36.9
100	**26.0**	**19.2**	**42.6**

Q_{100} = 26 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)

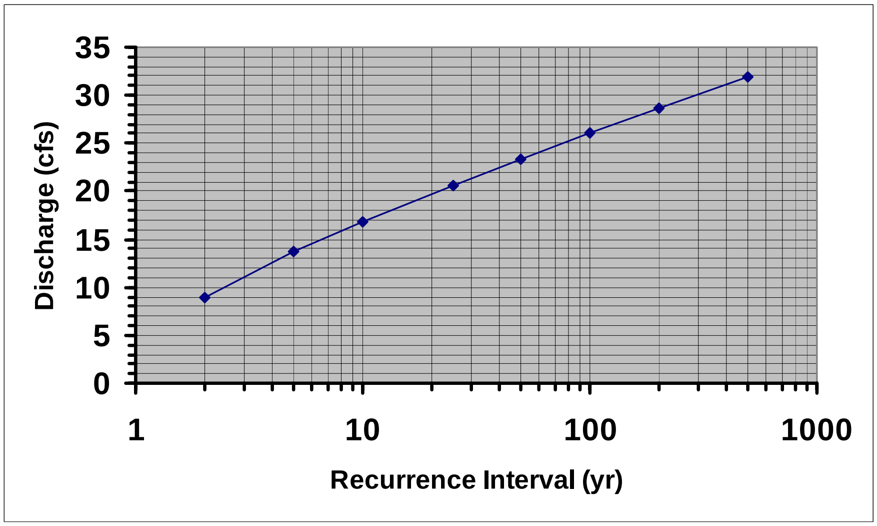

Figure 10. Plot of recurrence interval vs. discharge for Caspar Creek subwatershed HEN.

The rational method is recommended for basins less than 200 acres, while the USGS Magnitude and Frequency method is preferred over the rational method for drainage areas larger than 100 acres. Both methods are utilized for the HEN watershed for illustrative purposes. The direct flow transference method is preferred over both of these methods for HEN, however, since: (1) there are 40 years of discharge data for the downstream North Fork Caspar Creek gaging station available, (2) the subwatershed is within approximately one order of magnitude in size of the North Fork station, 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. Therefore, we utilized the direct flow transference method with a HW/D ratio of 0.67 and the 3 X bankfull stage 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 the results from these office methods and the field cross-sectional measurements, we recommend the selection of a **54 inch CMP**. The flow frequency analysis confirms that this is a reasonable estimate for this small watershed.

Table 5. Summary of the results using all the crossing sizing methods for determining the 100-year recurrence interval discharge and pipe diameters for subwatershed HEN.

Method	Predicted 100-Year Recurrence Interval Discharge (cfs)	Pipe Diameter (assuming HW/D Ratio = 0.67 for office-based methods)
Rational—Kirpich	**88**	**69**
Rational—Airport Drainage	**59**	**60**
USGS Magnitude and Frequency	**74**	**65**
Flow Transference	**42**	**52**
Direct Flow Transference	**30**	**45**
3 X Bankfull Stage	**--**	**52**
Active Channel Width	**--**	**53**
Flow Frequency Analysis	**26**	**43**

Figure 11. Clay Brandow, CDF Sacramento, measuring Caspar Creek sub-watershed HEN channel width for the 3 X bankfull stage calculation.

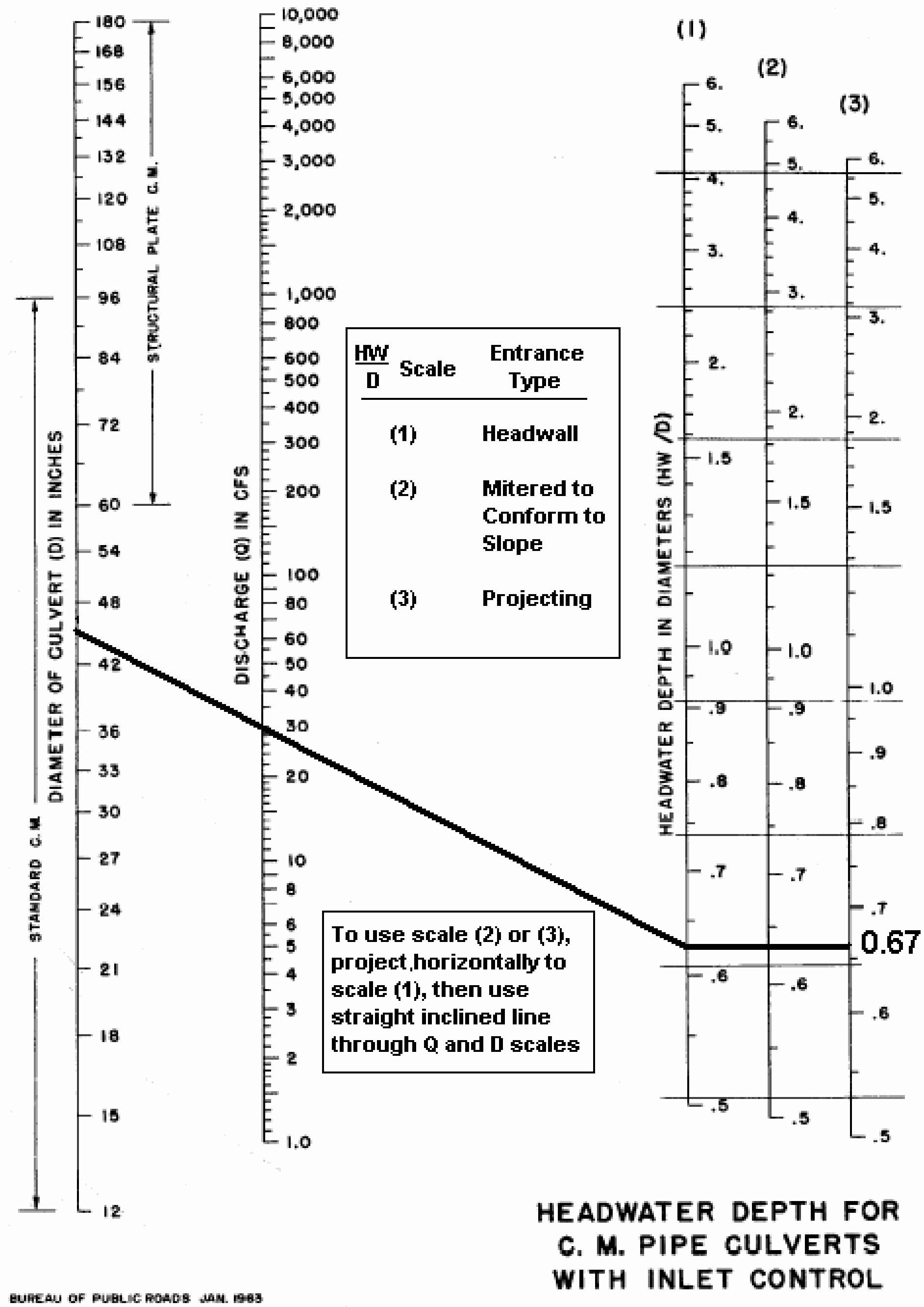

Figure 12. Normann and others (1985) culvert sizing nomograph for a round pipe with inlet control. For the watershed HEN example, using the direct transference method result of 30 cfs, a projecting pipe inlet, and a HW/D ratio of 0.67, the culvert size is 45 inches.

Part B. Predicting a 10-yr Recurrence Interval Event at 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 10-year recurrence interval event based on the flow frequency analysis presented in Part A of the Appendix (see Tables 3 and 4). **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 <u>actual</u> gaging station data (as calculated by the 10-year flood flow using the flow frequency analysis)**. It is assumed that: (1) the 18 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.

<u>Rational Method</u>

Known Information:

Drainage area (A) = 96 acres for HEN
10-yr 15 minute NF Caspar Creek rainfall = 0.54 inches/15 minutes (Goodridge 2000)
10-yr 30 minute NF Caspar Creek rainfall = 0.73 inches/30 minutes (Goodridge 2000)
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 Kirpich Formula):

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

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 hours or 6 minutes
T_c = 6 minutes. 15-minute rainfall-depth-duration-frequency data from Goodridge (2000) was used because 10-minute data was not available
0.54 inches/15 minutes x 60 minutes/hour = 2.16 inches/hour
I = 2.16 inches/hour
C = 0.3 (loam soil, Table 1)
Q_{10} = 0.3 x 2.16 inches/hour x 96 acres
Q_{10} = 62.2 or **<u>62 cfs</u>**

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

$T_c = ((1.8)\ (1.1 - 0.3)\ (2640^{0.5}))/(21^{0.33})$
T_c = 27 minutes, or approximately 30 minutes
0.73 inches/30 minutes x 60 minutes/hour = 1.46 inches/hour
I = 1.46 inches/hour
C = 0.3 (loam soil, Table 1)
Q_{10} = 0.3 x 1.46 inches/hour x 96 acres
Q_{10} = 42.0 or **42 cfs**

USGS Magnitude and Frequency Method (10-yr RI Equation)
Known Information:
A = 0.15 $miles^2$
P = 46.85 inches/year (Henry 1998)
H = 1.0 (North Coast region equations use a minimum value of 1.0 for the altitude index when $(H_{max} + H_{min})/2$ is less than 1000)
Calculate:
$Q_{10} = 6.21\ A^{0.88}\ P^{0.93}\ H^{-0.27}$
$Q_{10} = 6.21\ (0.15)^{0.88}\ (46.85)^{0.93}\ (1.0)^{-0.27}$
Q_{10} = 41.9 or **42 cfs**

Flow Transference Method (Waananen and Crippen 1977)
Known Information:
A = 96 acres for HEN; 1168 acres for the North Fork
Q_{10g} (10-year RI discharge at NF Caspar Creek weir) = 232.1 cfs (USGS PEAKFQ Program)
Calculate:
$Q_{10u} = Q_{10g}\ (A_u/A_g)^b$
Q_{10u} = 232.1 cfs $(96 \text{ acres}/1168 \text{ acres})^{0.88}$
Q_{10u} = 25.7 or **26 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) = 232.1 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} = 232.1 cfs (96 acres/1168 acres)
Q_{10u} = 19.1 or **19 cfs**

Table 6. Summary of the results comparing predicted 10-year discharges at HEN.

Method	Predicted 10-yr RI Discharge (cfs)
Rational—Kirpich	**62**
Rational—Airport Drainage	**42**
USGS Magnitude and Frequency – 10-yr RI equation	**42**
Flow Transference	**26**
Direct Flow Transference	**19**
Flow Frequency Analysis – 10 yr RI (see Appendix—Part A)	**17**

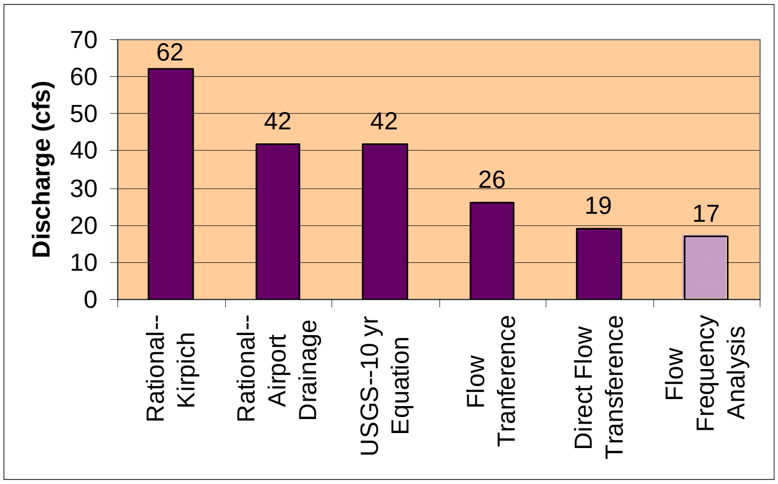

Figure 13. Predicted 10-year recurrence interval discharges and the 10-year RI event determined using flow frequency analysis (see Tables 3 and 4, Appendix—Part A).

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.
- 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 40 years.
- Based on these results, it is concluded that the direct flow transference method likely provides the best estimate of the 100-year RI discharge for subwatershed HEN.
- If the difference in gaged and ungaged watershed areas are larger than approximately one order of magnitude, 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 gaging station data. If this type of data exists, it should be used. Where it does not, the rational or USGS Magnitude and Frequency methods will be required, subject to the acreage limitations previously specified.

Arnold Schwarzenegger
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State of California

Michael Chrisman
Secretary for Resources
The Resources Agency

Andrea E. Tuttle
Director
California Department of Forestry and Fire Protection