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Arid Lands: Hydrology, Scour, and Water Quality

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Contents

Flood-Hazard Zonation in Arid Lands

H. W. HJALMARSON

Potential flood hazards in arid southern and western Arizona stem from different geomorphic and hydrologic characteristics and can be grouped into zones. The zonation is based on the physical features of the terrain, the sources of flooding, the expected frequency of flooding, and the expected erosion and sediment deposition. Various combinations of these factors create differing degrees of hazard. Distributary flow areas have stream channels that convey only a small fraction of the 100-year peak discharge and channels that can completely fill with sediments during a single flood. A basic understanding of the common and different flood hazards of areas in southwestern Arizona can lead to effective flood-plain management and design of hydraulic structures.

Desert floods in the southwestern United States result from large amounts of intense rainfall in the steep headwater areas. When this happens, the normally dry channels can suddenly host dangerous, debris-laden torrents (*1*). Typical floods are characterized by a rapid rise and cessation of discharge that are dramatically referred to as flash floods. Discharge generally is decreased by infiltration as the flood wave moves downstream over sandy alluvial channels (*2*). Large amounts of debris are carried down the channels, and the shapes of the channels generally change during flooding. Channels scour and fill during flooding, and channel banks wetted by floodwater often collapse after flooding.

Bridges on base-level streams often fail because of scour. Culverts located in aggrading alluvial areas fill with alluvial debris, and bank protection is ineffective. Many lives have been lost because of bridge failure, and damage to public and private property has been considerable.

This paper presents some generalizations about the nature of flooding in the deserts of southern Arizona that are based largely on the relationship between flood hazards and desert landforms. Flood hazards unique to the desert areas are described, and zones of potential hazard are characterized. Limitations of Federal Emergency Management Agency guidelines (*3*) are identified.

GENERAL CHARACTERISTICS

Degrees and types of potential flood hazard in the desert are related to geomorphic characteristics. Figure 1 illustrates the relationship between geomorphology and flood hazard and lists some general characteristics of the flood-hazard zones. Zone 1 is defined as the area inundated by the 100-year flood on base-level streams, which conforms to the present regulatory flood used by the Federal Emergency Management Agency (FEMA) (*3*). Zone 2 includes land adjacent to zone 1 that is subject to erosion by floods but not subject to inundation by the 100-year flood. Zone 3 includes relatively flat undissected areas where floodflow is shallow and unconfined; it includes former flood plains of base-level streams. Zone 4 includes areas of distributary flow, such as alluvial fans, where the amount of floodflow at a particular location is impossible to predict. Zones 5 and 6 include a variety of landforms where the 100-year flood is confined to rigid channels that generally drain areas less than 100 mi^2.

The mountainous areas (zone 6) are the source of weathered rock debris, and the stream channels usually have very little fine-grained material. A sharp break is often present in the gradient at the junction of the mountain front and the piedmont plain (zone 5) (fig. 2). Pediment areas are sparsely covered by a thin veneer of detritus, and stream channels have a mixture of fine- and coarse-grained material, including boulders. The alluvial fan and the base-level plain (fig. 1) have a wide variety of forms caused by natural and human-induced erosion and deposition that have occurred along the entire desert profile including base-level streams (*4*).

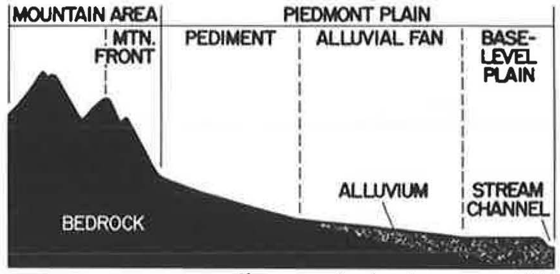
The channels of several alluvial streams have become entrenched because a balance was not maintained between factors such as flow, sediment discharge, slope, meander pattern, channel cross-section, and roughness. For example, minor fluctuations in meteorological conditions over a few years can alter the movement, transport, and production of sediment in a basin. During drier years, sediment can accumulate in stream channels, and subsequent wetter years may cause the sediment to be flushed from the basin. Reaches of channel with conditions of both uniform flow and nonuniform flow may appear to be aggrading or degrading. Thus, a reach of channel on an alluvial stream will not necessarily remain stable over a period of a few years.

ZONE 1

Zone 1 includes the channel and parts of the flood plain that would be inundated by the 100-year flood on playas, base-level streams, and larger tributaries. This zone has a high potential for flooding because floodflow normally is concentrated in defined channels and land adjacent to the channels. The velocity of flow in the channels is high, and the adjacent land is susceptible to erosion.

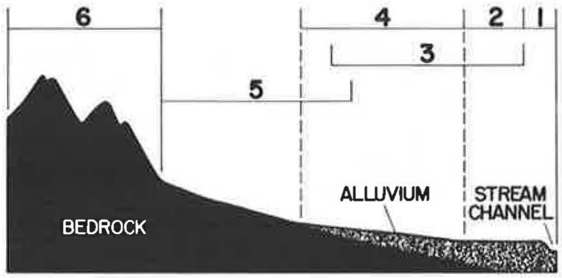
Historic information indicates that the current defined channels for base-level streams were not present until late in the nineteenth century and early in the twentieth century when some channels became entrenched (*3, 5*). The cause of entrenchment is the subject of considerable debate among hydrologists, but a strong argument can be made for change

U.S. Geological Survey, 300 West Congress Street, Tucson, Arizona 85701-1393.



Characteristic desert profile modified from (3)

A. Geomorphic components



B. Flood-hazard zones

Zone	Description
1	Extent of the 100-year flood on base-level stream.
2	Part of flood plain that may be inundated by rare large floods and (or) eroded by frequent small floods.
3	Flooding from sheetflow, standing water, and water that collects in depressions.
4	Flooding in channels and sheetflow on slightly dissected alluvial plains. Flow can be distributary and there is a greater than average chance of sediment deposition.
5	Flooding confined to defined channels of small tributary streams.
6	Sheetflow and flooding in defined clean-scoured channels.

FIGURE 1 Geomorphic features and flood-hazard zones of typical mountain-plain desert profile.

FIGURE 2 View looking north at the western slopes of the Tortolita Mountains. The sharp break in land slope at the junction of the mountain front and piedmont plain is typical of mountain-plain deserts.

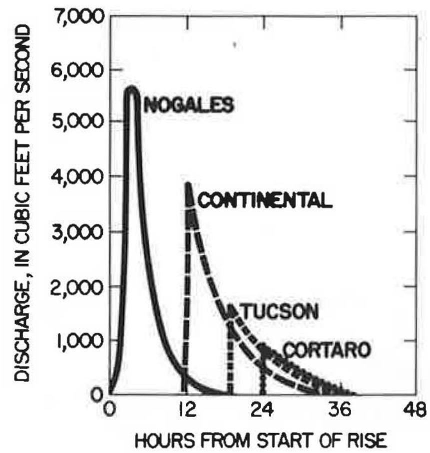
of climate. Floodflow in entrenched channels is more confined and the channel beds are less rough. Flood-wave celerity is greater and wave dispersion is less than for pre-entrenchment conditions. The entrenchment has had a significant effect on the flood characteristics of several base-level streams. Channel beds and banks can scour greatly in short periods during floodflow.

Zone 1 includes a variety of trenched and untrenched channels. Floodwater that is confined within a vertical walled arroyo only a few hundred feet wide can spread over an unchanneled valley for several miles downstream (figs. 3 and 4). Runoff that enters the desert-plain areas crosses progressively more alluvium where there is a great potential for infiltration (fig. 5). Burkham (*2*) found that the amount of loss along channels in the Santa Cruz River basin is related to the length of reach and the infiltration capacity of the channel.

Bridges on base-level streams become vulnerable to failure when the stream channel that supports the bridge is scoured. The abutments of many bridges in southern Arizona failed

FIGURE 3 View looking downstream at the entrenched channel of the Santa Cruz River at Tucson, Arizona. Floodwater of the 100-year flood is confined within the channel of the reach. Lateral erosion of the channel banks is restricted by massive soil-cement banks. Interstate 10 is located to the left of the 200-foot-wide by 20-foot-deep river channel. Since 1914, the channel has widened about 100 feet and deepened about 15 feet.

FIGURE 4 View looking east along Interstate 8 at the Santa Cruz River downstream from Tucson near Casa Grande, Arizona. The width of the flooding in zones 1 and 2 on October 4, 1983, was about 8 miles. Some water is on the road.



Gaging station	Miles from Nogales	Average annual runoff, in percent[1]
Continental	50	29
Tucson	79	19
Cortaro	88	14

[1]Amount of the average runoff at the Nogales gage that reached the indicated gage (1940-46, 1952-68).

FIGURE 5 Typical flow event showing transmission losses and attenuation of peaks for the Santa Cruz River, a base-level stream in southern Arizona (*7*).

during the flooding of October 1983 (figs. 6 and 7). Local scour around abutments and piers is a major cause of bridge failure on base-level streams in Arizona and throughout the United States (*8*).

Playa surfaces are rather flat, generally smooth, and composed of silt and clay. Many small, poorly defined channels are distributary or serve as distributary channels during floodflow as water crosses low divides. For example, during the large storm of early October 1983, runoff from Ash Creek, which is an unentrenched stream draining an area of about 500 square miles, spread laterally for more than 3 miles as floodflow entered the Willcox Playa. Nearly 2 miles of Interstate 10 near the town of Willcox was inundated with shallow floodwater, which resulted in highway closure for a few hours.

ZONE 2

Zone 2 includes areas adjacent to Zone 1 that could potentially be inundated by rare floods larger than the 100-year flood if the conveyance of the main channel changed or the hydraulic gradient changed or was eroded by floodflow. The potential hazard resulting from inundation is less than for areas in zone 1. For areas subject to erosion, the potential hazard is variable and can be greater than that for zone 1. Land adjacent to banks on the outside of bends or at constrictions or obstructions can erode quickly and extensively during frequent small flows of long duration (fig. 8).

Hazards in zone 2 are related more to lateral bank erosion than to inundation, and, at present, FEMA does not include

FIGURE 6 View looking south at one of many abutment failures resulting from floodwaters of October 1983 in southeastern Arizona. The scene is Interstate 10 at the Gila River on October 4, 1983. Flow is to the right.

FIGURE 8 View looking south and upstream at the Santa Cruz River at Interstate 19 on October 3, 1983. The right bank abutment of the northbound lane failed and the left bank abutment of the bridge to the right of Interstate 19 was destroyed during flooding on October 1 and 2. The dashed line approximately represents the location of the left bank of the entrenched channel before the flood.

expected bank movement in the definition of hazard degree. In fact, FEMA does not accept water-surface computations reflecting channel scour even where scour during floodflow is a common occurrence. Many models that predict channel scour, such as HEC-6, are in use, but the models do not consistently produce reliable results for all channels. Thus, improved models are needed to reliably define bank erosion for non-arbitrary flood-plain management of zone 2.

Many zone 2 floods originate in the surrounding mountains, where there is little soil and much exposed rock. Floodflow from these areas may carry sediment that is greater than the load. When floods confined in the channels reach the base-level streams (zone 1), the water picks up sediment from the channel banks. Floodflow in the steep, smooth channels can carry much sediment; thus, the banks in zone 2 areas can erode laterally tens of feet and even 100 feet or more during a single flood.

ZONE 3

Zone 3 is former flood plain of base-level streams and other relatively flat undissected areas. Areas are subject to sheetflow of a few inches to about 2 feet deep from floodflow originating in higher zones (figs. 9 and 10). Sheetflow a few inches deep can result from direct rainfall. Runoff generally is unconfined, and flow velocities generally are less than 2 or 3 square feet. The erosion hazard is low except along the few short incised channels.

Floodwater entering zone 3 spreads laterally and coalesces with floodwater entering the zone at other locations. Decreasing depth and velocity of flow as the width increases results in a reduced sediment-carrying capacity. Large amounts of sediment are deposited because of this spreading. Another

FIGURE 7 View looking downstream at the right bank of Rillito Creek at the Southern Pacific and Interstate 10 bridges at Tucson, Arizona. The failure of the wire-rock revetment at the abutments is typical for base-level streams in the area.

FIGURE 9 View looking northeast at floodwater from a small confined wash debouching onto land in zone 3. Floodflow spread to a width of more than 1 mile about half a mile downstream from the confinement. Flooding was on June 22, 1972, upstream from the Arizona canal east of Scottsdale, Arizona.

FIGURE 10 View looking south and downstream at sheetflow in zone 3 on June 22, 1972. The scene is in northeast Phoenix at 44th Street between Bell and Greenway Roads.

factor contributing to sediment deposition is loss of flow due to infiltration.

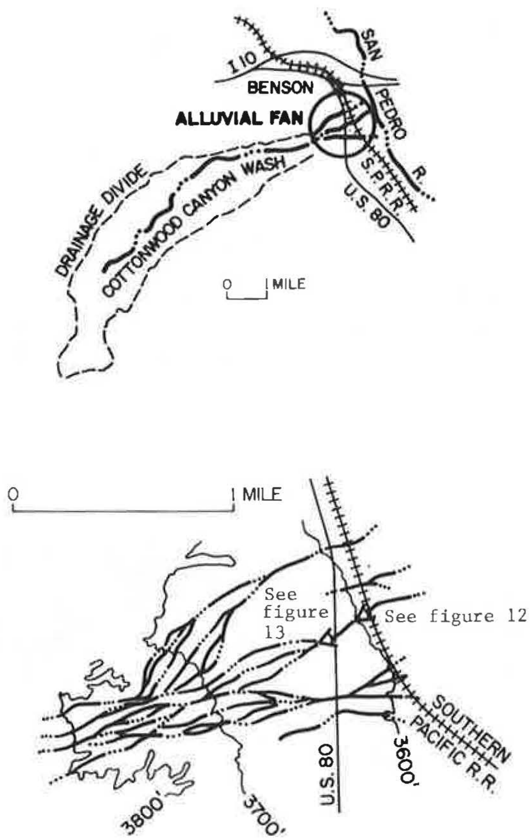
Culverts and bridges in zone 3 are usually not subject to serious erosion hazards unless the structure causes excessive backwater. Where excessive backwater does occur, the high head and corresponding high velocities through the structure opening can result in hazardous erosion of material supporting the structure. Sediment deposition resulting in the filling of structure openings, such as culverts, with debris is an occasional problem.

ZONE 4

Floodwater entering zone 4 from confined channels in zones 5 and 6 spreads into distributary channels (fig. 11) with a corresponding decrease of velocity and depth. The amount of flow also is decreased by infiltration into the sandy beds. There is less water and less energy to transport sediment, and thus sediment is deposited in and along the channels to form a mound of alluvial material. Channels completely fill during flash flows, and culvert and bridge openings become ineffective (figs. 12 and 13). Frequent cleaning of culvert and bridge openings is needed at many stream channels in zone 4.

Zone 4 includes the slightly dissected alluvial slopes that commonly exhibit a distributary drainage system. The flood potential of zone 4 has often been overlooked (*9*). Bajadas and single alluvial fans (fig. 14) are typical landforms in the aggrading area. The rate of sediment deposition, one aspect of the dynamic behavior of the fans, is complex and variable (*3, 5*). Some fans seem to aggrade at a rapid rate, and the active channels change frequently. Many of the fans in southern Arizona appear to be less dynamic than fans in areas of southern California (*10*) and Nevada (*11*), where tectonic activity is greater. Also, on the basis of soil characteristics such as the age of the bajada soils (*12*), the alluvial slopes in some areas are relatively stable; apparently, little aggradation or degradation occurred during the Holocene epoch (about the past 10,000 years). Many alluvial fans are present in southern Arizona (*13*), and they may occupy about 30 to 40 percent of the area.

FEMA has presented methods for evaluating flood hazards on alluvial fans that assume channels downstream from the fan apex are equally likely to occur any place on the fan



Location and view angle of photograph.

FIGURE 11 Alluvial fan showing contours and distributary channels on Cottonwood Canyon Wash at Benson, Arizona.

FIGURE 12 View looking downstream at railroad bridge in south Benson, Arizona. The opening was completely filled during a 1-hour flash flood on July 6, 1981. Note the depth of the channel where the filled material has been removed about 100 yards downstream from the bridge. See figure 11 for location of photograph.

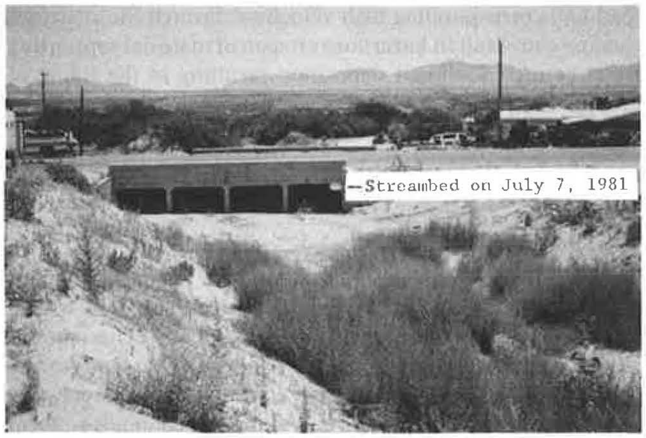


FIGURE 13 View looking downstream at culvert on U.S. Highway 80 in south Benson, Arizona. The opening was nearly filled during the flash flood of July 6, 1981. Floodflow velocities in the main channel downstream from the culvert were very high and a local resident observed two standing waves about 20 feet apart at the flood peak. See figure 11 for location of photograph.

surface (*4*). Although this assumption may be valid for estimating the flood hazard of highly active fans, it may not be applicable for the many fan surfaces in southern Arizona that are relatively inactive. The more stable fans have a defined network of distributary channels with some abandoned channels that presently head on the fan surface. Floodflow is more likely in the defined channels that head in mountains, less likely in the abandoned channels, and unlikely on much of the high ground between the channels. Although the amount of discharge in a particular branch of a divided channel is difficult to determine, the likelihood of floodflow at any location on the fan surface is not equal.

The topographic relief across single alluvial fans and bajadas is variable and is an index of the age of the landform. The local relief between channels in zone 4 is commonly less than 5 feet but occasionally more than 20 feet. Alluvial fans with small local relief tend to be more active than alluvial slopes with large relief.

The filling of the stream channel shown in figures 12 and 13 may be offsetting the potentially hazardous headcutting of the channel. The stream is tributary to the San Pedro River, which is entrenched. Tributaries to the San Pedro River also have become entrenched near the river (fig. 15). The hazardous conditions shown in figures 14 and 15 are representative of the variable and dynamic behavior of streams in southern Arizona.

Floodwater on inactive fans generally is in entrenched channels that anastomose, divide, and combine. Much of the land clearly is above the 100-year flood, but flood hazards on fans are unpredictable. Possible consequences of floods in the low-lying land and channels include:

1. Channel erosion and lateral bank movement.
2. Channel filling with deposited sediment and the associated increased flooding of adjacent flood plain.
3. Lateral shifting (avulsion) among distributary channels.

The FEMA type of flood hazard assessment (random distribution of flood depth and velocity) may not be applicable. Flood hazard assessment for bridge or culvert design is difficult because flood response at any given location on channels in zone 4 is unpredictable.

ZONE 5

Zone 5 is defined as the pediment and upper alluvial plain areas with defined channels that commonly form a tributary system. The surface of the pediment areas is a complex mixture of rock, alluvium, and thin soils of various ages. Stream channels commonly have slopes from 0.02 to 0.04 with an upper limit of about 0.2 (*3*). Channel beds in the pediment or upper area of the zone are often composed of scattered boulders with cobbles, gravel, and some sand. Channel beds in the upper alluvial areas tend to have fewer boulders and more sand. The potential for significant scour of the channel

FIGURE 14 View looking east at distributary channels of zone 4 on the western slopes of the Tortolita Mountains north of Tucson, Arizona. The land in about the top quarter of the photograph is in zone 5.

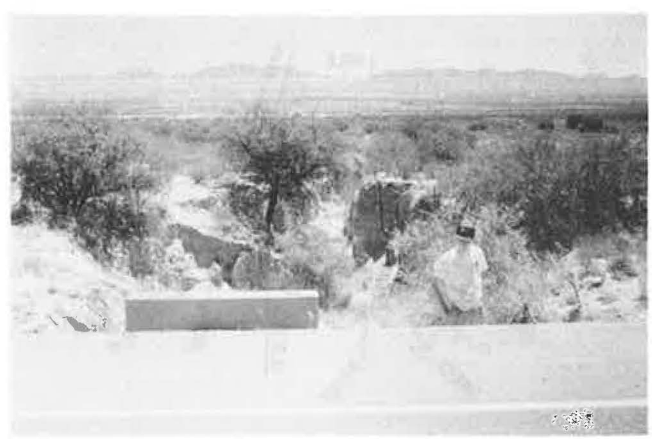


FIGURE 15 View looking downstream from U.S. Highway 80 at small scoured channel of a tributary to the San Pedro River located 0.6 mile south of the filled channel shown in figures 12 and 13.

bed and banks in the pediment area is low. Marked scour along some channels in the upper alluvial plain area can occur, but the general potential for scour is not great. Debris flows, defined here as slurries of sediment and water with a sediment weight-percentage above 80 percent, that are potentially hazardous can occur in zone 5.

The boundary between zones 4 and 5 generally coincides with the boundary between Quaternary and Tertiary valley-fill deposits. In some places, the tributary-defined channels characteristic of zone 5 extend into the Quaternary deposits. The small distributary channels of zone 4 rarely extend upslope in the Tertiary deposits. In some places, the boundary that separates zones 4 and 5 is a transition area several hundred feet wide.

The greatest potential hazard in zone 5 is from flooding in the channels and narrow flood plains that occupy the lowlands between the defined ridges. Marked scouring occurs along some of the channels and flood plains, and floods carry large amounts of sediment. In many channels, the depth of flooding depends on the amount of erosion and deposition that takes place during the flood. The depth of flooding generally does not exceed 10 ft except where channels are obstructed, on the outside of sharp bends, and on the few channels that drain areas of more than about 100 mi². The depth of floodwater also increases behind debris jams and manmade obstructions. The degree of potential flood hazard of the larger washes in zone 5 is similar to that in zone 1 but with less potential for scour. The main channel of some washes is deceptively small, and large amounts of floodwater will spread over wide areas adjacent to the channel.

ZONE 6

Mountain areas that include steep, well-drained slopes composed mostly of rock are characteristic of zone 6. Interspersed among the rock surface are scattered thin debris mantles and thin soils. Stream channels are steep, scoured, and rocky. Channels of streams draining basins of a few tenths of a square mile are well defined.

The dominant hazard is along the defined channels where flood velocities are high; velocities in the large channels may be as much as 15 feet per second. Sheetflow accompanied by debris flow may occur along some steep slopes. Peak-discharge rates of as much as 500 cubic feet per second from a 0.1-square-mile area can be expected an average of once every 100 years. A large part of the flood-hazard potential in this zone can be attributed to sudden flooding from summer thunderstorms and the high velocity of flow.

If the potential for debris flows exists, then the hazard associated with a debris flow may be the greatest in this zone. The potential for debris flows is directly related to the amount and size of unconsolidated material on steep, nonvegetated slopes.

TABLE 1 TYPE AND DEGREE OF FLOOD HAZARD FOR ZONES

Type of hazard	Flood-hazard zone					
	[1]1	2	3	4	5	6
Inundation of land along channels	high[2]	moderate	moderate	high[2]	moderate	low
Velocity of floodflow	high	moderate	low	high[2]	high	high
Scour of channel bed	high	moderate	low	moderate	low[3]	low
Lateral bank erosion	high	high[2]	low	high[2]	low	low
Sediment deposition	low[4]	low	high[5]	high[5]	low	low
Debris flows	low	low	low	low	moderate	high

[1]High incidence of bridge failure because of scour of piers, abutments, and roadway approaches.

[2]The assumption on which FEMA guidelines is based may not be applicable for fan surfaces that are relatively inactive.

[3]Moderate in upper alluvial plain areas and in large channels.

[4]Moderate to high in unchanneled reaches.

[5]Conveyance of many culverts and bridges reduced because of sediment deposition.

DISCUSSION AND SUMMARY

Geomorphology plays an important role in determining flood hazard. Although this fact is common knowledge, structures continue to fail or become less effective, at least in part because of flood-plain management regulations that may not be applicable for some zones. The hazards that commonly plague engineering works are the lateral bank erosion in zone 2, the scour of channel beds in zone 1, and the sediment deposition and unpredictable flow paths in zone 4.

The relative degree and type of hazard for the six zones are summarized in table 1.

The zonation is based on distinct geomorphic and hydrologic differences between the zones, but there is some overlap (see fig. 1). Zones 2 and 3, for example, can define the hazard of the same land where there is a potential for lateral movement of the banks of channels in zone 1 and also for sheetflow from local rainfall or from runoff from zones 4 or 5. Alluvial fans have a wide variety of flood characteristics, and thus specific areas can be best described by zones 3, 4, or 5. In general, large areas of fans will exhibit characteristics of a single zone.

This general zonation is not intended to replace the detailed engineering definition of hydrologic and geologic characteristics of a particular site of interest. Rather, the zonation of flood hazards can be useful to practicing engineers for the general identification of the type and degree of flood hazard.

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Paleoflood Hydrologic Research in the Southwestern United States

ROBERT H. WEBB AND SARA L. RATHBURN

Paleoflood hydrology is a cross-discipline between geomorphology and hydrology that uses geologic evidence to estimate discharges for historic and (or) prehistoric floods. Analysis of fine-grained flood deposits has been used to estimate the magnitude and date of occurrence of floods on rivers and streams in the southwestern United States. These flood deposits and other geologic evidence of floods, termed paleostage indicators, are emplaced along the margins of channels from streamflows with high concentrations of suspended sediment. Dates for floods are determined from historic records, analysis of scarred or damaged trees, relative ages of soils developed on flood deposits, and radiocarbon dating of organic material entrained in flood deposits. Flood discharges can be estimated from paleostage indicators by using the step-backwater method in which the channel is either assumed or demonstrated to be stable. A comparison of the techniques that have been used reveals inconsistencies caused by a rapidly evolving discipline, and standardized procedures are needed for the analysis of flood deposits. The three case studies presented illustrate the use of paleohydrologic data with gaging data for estimating flood frequency using maximum-likelihood techniques. Maximum-likelihood techniques for fitting probability distributions can explicitly account for the uncertainties inherent in paleoflood data and provide greater flexibility in the use of paleoflood records with gaging records in flood-frequency analysis, thereby yielding improved estimates of design floods with large recurrence intervals.

Fluvial paleohydrology is the interdisciplinary study of the past movement and distribution of water and sediment in river channels. Paleohydrology links conventional engineering and hydrologic techniques of discharge calculation and flood-frequency analysis with the geologic emphasis on stratigraphy, sedimentology, and geomorphology. One approach used in paleohydrologic research is the documentation of fluctuations in channel morphology with time to estimate changes in hydrologic conditions (*1*); this approach is used mainly in paleoclimatic studies on alluvial rivers (*2, 3, 4*). A second approach uses geologic evidence to reconstruct the magnitude and frequency of past floods in bedrock channels (*5–9*). Although its origins can be traced to the early 19th century (*10*), paleohydrologic research has grown substantially in the last decade.

Estimation of the magnitude and frequency of past streamflow floods (paleofloods) has been based on bankfull-discharge relations (*1, 11*), the size of boulders transported during floods (*9*), or geologic evidence of maximum flood stages (*5–7*). In the southwestern United States (fig. 1) the most common technique for estimating discharges uses relict geologic evidence of water-surface elevation, or paleostage indicators. These paleostage indicators include fine-grained flood deposits (termed "slackwater deposits," (*5*), see fig. 2), silt lines correlative with flood deposits (*12*), and erosional scars cut into hillslopes (*13*). Flood discharges required to emplace the paleostage indicators are estimated from hydraulic equations, and the date of the flood is determined by a variety of techniques, including radiocarbon dating.

Paleohydrologic research using paleostage indicators has increased steadily since the late 1970s. Rivers in Texas (*14–16*), Arizona (*13, 17–20*), and Utah (*12, 21, 22*) in the United States (fig. 1, table 1) and in the Northern Territory of Australia (*23–25*) have been studied. As research involving paleostage indicators has developed, many techniques have been used in estimating discharges and dating floods. Development of efficient statistical techniques for incorporating paleohydrologic and historic information in flood-frequency analyses (*26–30*) will undoubtedly encourage further applications of paleohydrology to estimate the magnitude and frequency of floods.

This paper reviews the techniques used in paleohydrologic studies of rivers in the arid and semiarid southwestern United States and illustrates the use of paleoflood data derived from analysis of fine-grained flood deposits with gaging data in flood-frequency analysis. Other reviews of paleohydrologic research in this region are given in references elsewhere (*5–7*). The inaccuracies and imprecision in paleohydrologic techniques are emphasized and are explicitly incorporated in frequency analyses of gaging records from three rivers in the Southwest.

PALEOSTAGE RESEARCH TECHNIQUES

Accumulation Sites and Stratigraphy

Fine-grained flood deposits are commonly preserved along bedrock channels in the southwestern United States. These sediments are deposited from sediment-laden waters in zones of reduced flow velocities during floods (*5*). Tributary mouths, channel margins upstream of contractions and downstream of expansions, and rock shelters or caves above the low-flow channel are typical accumulation sites for flood deposits (fig. 2). If the depositional sites are protected from subsequent erosion, thick sequences of deposits will accumulate as successive floods either overtop or erode into the side of the deposit (*5, 12*).

The stratigraphy of fine-grained flood deposits contains evi-

U.S. Geological Survey, 300 West Congress Street, Tucson, Arizona 85701-1393.

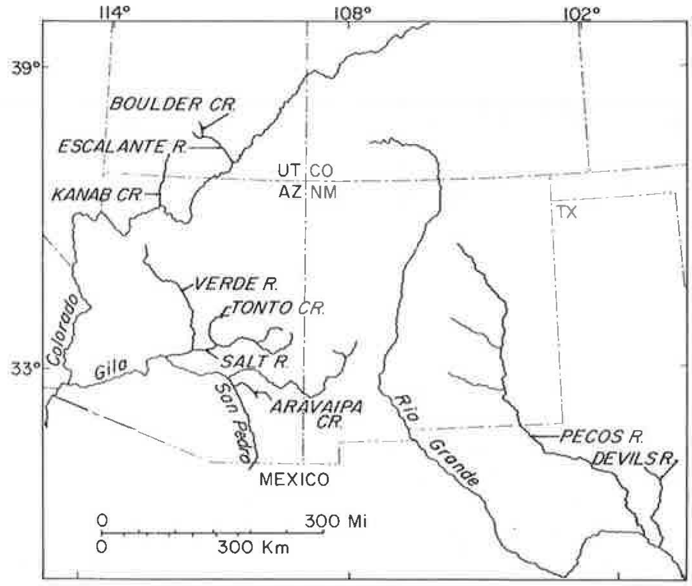

FIGURE 1 Rivers in the southwestern United States for which paleohydrologic data have been collected.

dence of discrete floods, and organic material entrained in the deposits can be used to develop a chronology of such floods. Flood deposits may have characteristic sedimentary structures, such as climbing ripples or crossbedding, or they may have no structures (*5*). Deposits created by unique floods may be separated using many criteria, including drapes of silt or organic debris between layers representing different floods, abrupt stratigraphic breaks including unconformities, differences in grain-size distributions, differences in weathering characteristics, and changes in color of sediments (*5*). Discontinuous flood deposits along a channel may be correlated using stratigraphic position, radiocarbon dating, color, or li-

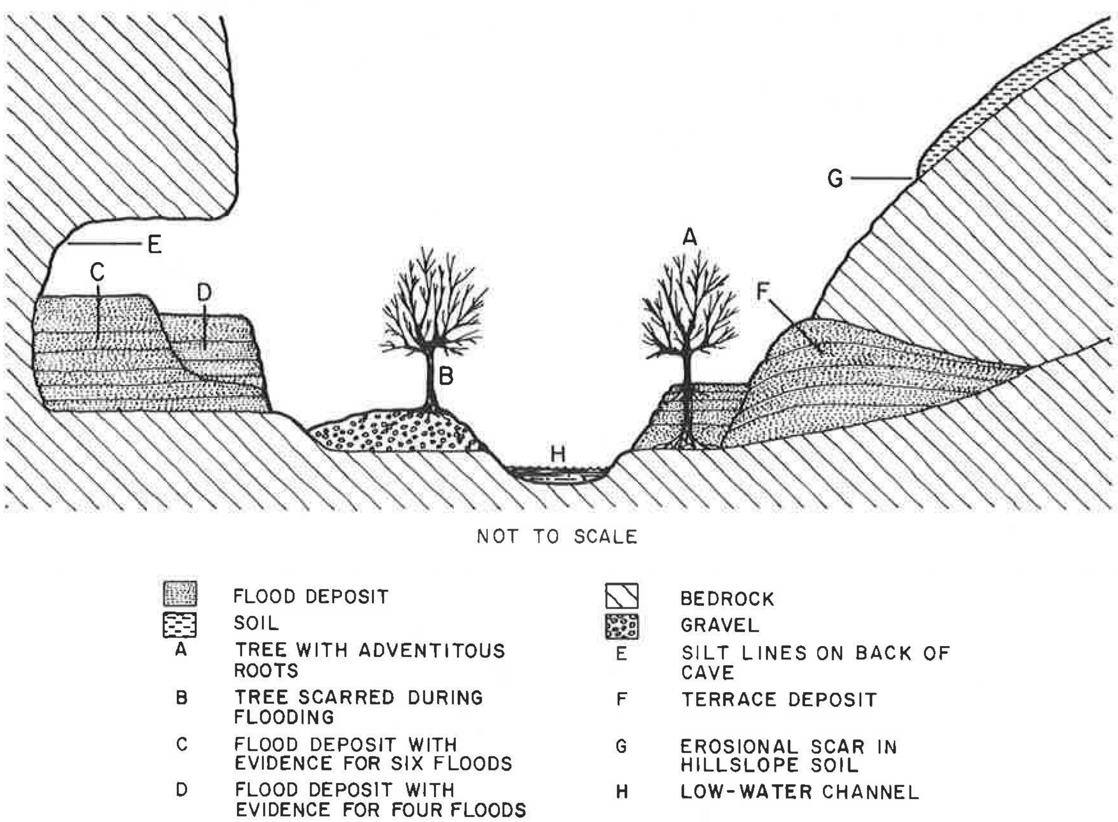

FIGURE 2 Schematic diagram of a typical flood-deposit site showing the types of evidence used in dating floods and estimating discharges [modified from Baker (*6*)].

TABLE 1 SUMMARY OF PREVIOUS STUDIES OF PALEOHYDROLOGY OF RIVERS OR CREEKS IN THE SOUTHWESTERN UNITED STATES

Name	Area (km^2)	Number of paleo-floods	Largest recorded flood (m^3/s)	Date	Length of paleoflood record (years)	Method used for estimating discharge	Number of sections	Reference
				TEXAS				
Pecos	9,500	30	27,400	1954	9,530	ME	1	14
Devils	7,100	0	16,700	1954	8,730	ME	1	14
				UTAH				
Escalante	810	18	720	950	2,100	SBW	10	22
Boulder Cr	450	3	400	1650-1950	1,000-1,100	SBW	27	21
				ARIZONA				
Verde	15,000	10	5,400	<950	1,010	SBW	20	17
Salt	11,000	14	4,600	<950	>600	SBW	24	13
Salt	21,000	27	11,900	850	1,100	SBW	12	18
Aravaipa	1,340	6	970[a]	unknown	1,100	SBW	28	20
Tonto Cr	1,630	4-5	1,000[a]	1980	280	SBW	13	19
Kanab Cr	5,370	10-12	600	1550	500	SBW	11	

[a]Discharges are lower than maximum discharge reported by U.S. Geological Survey.

[Dates of floods are given in calendar years A.D. that were obtained in some cases from conversion of radiocarbon dates. ME--Manning's equation, SBW--step-backwater method. Data from Kanab Creek is from S.S. Smith, University of Arizona, unpublished data]

thology. Flood deposits may represent vertically-aggrading sections, where each successive flood must be larger than the previous flood (*5, 14, 15*), or inset deposits within the flood deposits may imply that all floods exceeding a fixed stage will have stratigraphic evidence preserved (fig. 2) (*12, 21, 22*).

The type of flood deposit studied is important to the statistical treatment and determination of a censoring threshold, which is a discharge that is either exceeded or not exceeded over a known length of time (*28*). The main objective is to verify that all floods that exceeded the censoring threshold left stratigraphic evidence. Such verification is difficult to obtain, but studies of deposits associated with known historic floods (*12, 13, 14, 17, 19, 20, 23, 25*) suggest that stratigraphic evidence would have been preserved if other larger floods had occurred. The censoring threshold in paleoflood data typically is high in terms of recurrence interval, as the examples presented later in this paper show.

Dating Techniques

Dates for paleofloods are obtained using several different techniques, each of which has an associated degree of uncertainty. Radiocarbon dating of organic material entrained in or associated with the flood deposit is the standard method for dating flood deposits (*5*). Radiocarbon dates are reported in years before present (yrs BP) with A.D. 1950 as the calendric date for a radiocarbon date of 0 yrs BP. Use of radiocarbon analyses allows the dating of floods that have occurred over the past 50,000 years, if evidence for the floods is preserved. Radiocarbon dates typically have an uncertainty inherent in the counting techniques that ranges at best from ±40 years on young (less than 10,000 yrs BP) samples to ±1,000 to 2,000 years on older (greater than 20,000 yrs BP) samples. Uncertainties can be much larger if very small samples of organic material are analyzed. Floods that occurred after A.D. 1950 can be dated to the nearest year using ultramodern radiocarbon dates, which are based on the excess amounts of 14C in the atmosphere that resulted from aboveground nuclear tests (*24*).

Radiocarbon dating of flood deposits assumes that organic material in flood deposits is contemporaneous with the flood that deposited it (*5, 22*). This assumption may or may not be valid. For example, charcoal in a flood deposit may have been created by a fire during the year in which the flood occurred, or the charcoal may have been reworked from older alluvial deposits and may yield a date much older than the date of the flood. Organic debris in the deposit, especially leaves and small twigs, represent the most recent growing season before the flood and thus accurately represent the date of the flood (*6*). Radiocarbon dating of organic litter, or material that accumulated on the surface of a deposit between floods, has been used to constrain the dates for subsequent floods (*14, 22*). Contemporaneity of organic material with the flood deposit is the most important assumption involved in radiocarbon dating of past floods.

Other methods used for obtaining absolute or relative ages of flood deposits include dendrochronology (*31*), soil development (*13, 14*), association of historical or archaeological artifacts in deposits (*12–14, 18, 21*), and historic records such as personal accounts or newspaper articles (*14, 22*). Dendrochronology is used to date floods to the nearest year by studying the damage inflicted on trees growing adjacent to the channel (fig. 2). Burial of trees in flood deposits encourages the growth of adventitious roots, which sprout from the buried trunk of the tree and can be dated using dendrochronologic methods (*31*). Trees unaffected by floods, but which bracket the age of deposits by their growth position, also have been used to constrain the date of floods (*13, 19*). Use of soil development as a relative dating tool introduces large uncertainties in the dating of floods because soils cannot be accurately dated; however, soil development is a useful correlation tool in the absence of organic material. Artifacts can be used to determine whether floods are historic, and archaeological artifacts can be used to date flood deposits as accurately as radiocarbon dating if the artifacts can be placed within a regional archaeological chronology (*13, 21*).

Estimation of Discharge

Two methods have been used to estimate discharges from paleostage evidence. The first estimates of discharges for paleofloods in the southwestern United States (*5, 14*) were obtained using Manning's equation:

$$Q = \frac{1}{n} A\, R^{2/3}\, S^{1/2} \tag{1}$$

where Q = discharge in cubic meters per second (m^3/s); n = a roughness coefficient called Manning's n; A = cross-sectional area (m^2); R = hydraulic radius (m); and S = friction slope of the water surface. The channel conveyance, K, is defined as:

$$K = \frac{A\, R^{2/3}}{n} \tag{2}$$

For solution of equations 1 and 2, one cross-sectional area is required with the top of the flood deposits serving as a water-surface elevation (*14*). Use of only one cross section implies that the section used is representative of the local channel conditions, which may not be accurate. Because many flood deposits were in tributary mouths away from the main channel flow (*14*), the height of some deposits may represent a fraction of the total energy of flow instead of just the energy associated with the velocity. The water-surface slope used in Manning's equation was first assumed to equal the channel slope (uniform flow), and it was increased to 1.25 times the channel slope after the initial discharge estimates were found to be much less than the discharges recorded at gaging stations (*14*). Finally, a least-squares curve fitting revealed that discharges estimated from known water-surface elevations were 1.6 to 2.0 times larger than the discharges estimated using the height of flood deposits as the water-surface elevation (*14*). The results of this study (*14*) show that considerable uncertainty in discharge estimates can result from use of Manning's equation alone.

The step-backwater method (*32*) is most commonly used for estimating the discharges of paleofloods from paleostage indicators (*12, 13, 17, 18, 19, 20, 21, 22, 23, 25*). The step-backwater method combines energy losses associated with channel resistance, as calculated from equation 2, with conservation of energy and mass equations. Conservation of energy is calculated from the equation:

$$h_1 = h_2 + (h_L)_{12} + \epsilon\Delta(h_v)_{12} \tag{3}$$

where h, the total flow head, is calculated from:

$$h = z + \frac{V^2}{2g} \tag{4}$$

and where $(h_L)_{12}$ = head loss, which is proportional to the discharge and reach length between sections 1 and 2 and inversely proportional to K; ε = energy-loss coefficient, which conventionally is 0.5 in expanding reaches and 0.0 in contracting reaches (*32*); $\Delta(h_v)_{12}$ = difference in velocity head between sections 1 and 2; z = water-surface elevation; V = flow velocity (the correction coefficient, α, is assumed equal to 1 (*32*)); and g = gravitational acceleration. The velocity head is calculated from:

$$h_v = \frac{V^2}{2g} \tag{5}$$

The conservation of mass equation is:

$$Q = A_1V_1 - A_2V_2 \tag{6}$$

Details of the calculations in the step-backwater method are presented elsewhere (*32, 33*). In the step-backwater method, stages are estimated at specific cross sections for given discharges. Water-surface profiles for various discharges are compared with the heights of flood deposits or other paleostage indicators until a discharge that best approximates the paleostage indicators is accepted.

With the step-backwater method, the stages associated with various discharges are estimated independently from paleostage indicators, because the method is independent of evidence for water-surface elevation. Flood deposits represent minimum water-surface elevations (*5*), and definition of cross-sectional areas by paleostage indicators results in minimal estimates of discharge. Paleostage indicators may not provide a continuous record of water-surface profile through a reach; however, by using the step-backwater method, additional cross sections without paleostages can be used to account for nonuniform flow conditions.

Assumptions concerning the effective flow area, hydraulic interpretation of paleostage indicators, and choice of energy-loss coefficients cause uncertainties in discharge estimates. Use of the step-backwater method requires the assumption of one-dimensional flow in the channel reaches. Uncertainties in the sizes of ineffective flow areas or eddies cause errors in the discharge estimates. Flood deposits are assumed to represent the water-surface profile of the mean channel flow. Use of flood deposits in tributary canyons, which are removed from the flow in the main channel, will introduce errors in discharge because these deposits are indicative of both the main-channel velocity head and some fraction of the total energy head. Finally, choice of Manning's *n* values and energy-loss coefficients introduces additional uncertainty in discharge estimates, although uncertainties associated with reasonable values of Manning's *n* and energy-loss coefficients are much less than the uncertainties in cross-sectional data (*33*).

Discharges estimated for paleofloods have inherent uncertainty because of the potential for channel change between floods. Channel cross sections are measured decades to thousands of years after the flood has occurred. For this reason, paleofloods are studied in bedrock channels where the channel generally is stable (fig. 3) and cross sections change little between floods. The little historic change in sediment storage along rivers with drainage areas greater than 10,000 km^2 in southern Utah found by Graf (*34*) indicates that long-term aggradation and degradation are minimal. Changes in alluvial channels during the period of paleohydrologic record have been estimated and used in discharge estimates for paleofloods (*18, 22*). Uncertainties in discharge data estimated from paleostage indicators should be carefully evaluated before the data are used in flood-frequency analysis.

Discharge estimates for historic floods based on paleostage indicators compare favorably with discharges recorded at gaging stations or discharges estimated immediately after the flood, although some notable discrepancies have been found (*13, 19, 20*). These discrepancies—usually an underestimation of the discharge recorded at gaging stations—have been explained either by differences in discharge-estimation technique (*19*) or by flow from tributaries between the flood-deposit site and the gaging station (*13*).

(a) (b)

FIGURE 3 The Pinnacle, Kanab Canyon, Utah. (a) Taken in October 1871 by Jack Hillers (Hillers photograph number 629, U.S. Geological Survey photograph library, Denver, Colorado). (b) Taken in September 1985 by Robert H. Webb. The channel morphology has changed little since 1871.

USE OF PALEOFLOOD DATA IN FLOOD-FREQUENCY ANALYSIS

Previous Analyses Using Paleohydrologic Data

Flood frequency analysis in the United States is done by fitting a Pearson type III distribution (*35*) to log-transformed flood data. The merits and disadvantages of this method have been discussed elsewhere (*28, 36, 37, 40*), and the fitting of other probability distributions has been attempted in only one paleohydrologic study in the southwestern United States (*39*). The Water Resources Council methods (*35*) are based on a method of moments fitting of the log-Pearson type III distribution. Without historical or paleoflood data, the gaging data are fitted to the distribution by calculating the mean, standard deviation, and skew coefficient of the log-transformed discharges. The log-Pearson type III curve is defined by:

$$\log_{10}(Q) = \overline{X} + k\overline{S} \tag{7}$$

where Q = discharge; $\overline{X}$ = mean of the discharge data; $\overline{S}$ = standard deviation of the discharge data; and k = a factor that is a function of the skew coefficient and the recurrence interval. Equation 5 is modified when historical or paleoflood data are included in the analysis (*35*). Criticisms of this method for use with historical information are presented in papers by Cohn and Stedinger (*26–29*).

Plotting-position formulae are used to graphically present the data, and one general equation for plotting position given in the Water Resources Council methods (*35*) is:

$$P = \frac{(m - a)}{(N - 2a + 1)} \tag{8}$$

where P = probability of exceedence; m = rank of discharges with m = 1 for the largest; N = number of years of record; and a = factors that are dependent upon the distribution. For the commonly used Weibull plotting position, a = 0. Equation 8 is modified in several different ways to account for statistical differences between systematic gaging data and historical or paleoflood data (*28, 35, 38*). Previous attempts to extend the effective length of gaging records with paleoflood information generally have used either Weibull plotting positions or a method of moments fitting of the log-Pearson type III distribution (*5, 12, 13, 14, 15, 16, 22*). The type of plotting position used in plotting paleohydrologic data with gaging data creates less variability than the inherent variability in the sampling of the population from which the flood data arise (*38*).

Methods that differ from the Water Resources Council methods have been proposed for the treatment of paleoflood information (*26, 28, 29, 38*). Stedinger and Cohn (*28*) propose the maximum-likelihood method for using paleoflood and gaging record data to fit probability distributions. With that method, paleoflood information can be treated either as Type I censored data (*26, 29*) or binomial-censored data (*26, 28*). Paleofloods can be described by a single discharge, a range in discharges, or a threshold exceedence. The lack of floods over a given time period (negative evidence) can be used in this type of analysis by establishing a threshold with no exceedences. The paleoflood information is used with or without gaging data in a likelihood function, which is maximized to obtain parameter estimates of a selected probability distribution. Details of the technique applied to several probability distributions are given in 26 and 27. General maximum-likelihood estimators for the log-Pearson type III distribution have been developed and applied to paleoflood data for the Salt and Verde Rivers in Arizona (*39*).

The maximum-likelihood method using censored data has been shown to be more flexible, efficient, and robust in Monte Carlo tests (*28*) than the Water Resources Council methods (*35*). Use of paleoflood data increases the effective record length, or number of years of gaging data that would produce the same mean square error as a combination of gaging and paleoflood data (*28*). The effective record length increases from the length of the gaging record to two-thirds the length of the paleoflood record when fitting the two-parameter log-normal distribution if the censoring threshold is at the 90th percentile (*28*). For a censoring threshold at the 99th percentile, the effective record length is approximately one-fifth the length of the paleoflood record. Similar increases in effective record length have been obtained for three-parameter distributions (*29*).

Standard errors of estimate are a function of the mean, standard deviation, and length of record and can be used to describe the precision of flood recurrence-interval estimates. In this paper, standard errors are calculated using the methods presented in 41 for the log-Pearson type III distribution. The standard error of estimate of distributions fit with gaging records versus distributions fit with gaging and paleoflood records are compared to show the ability of paleoflood data to decrease the standard error, and thus increase the precision, of recurrence-interval estimates.

Because of uncertainties in discharge estimates and dating of floods in paleoflood studies, and because thresholds can be used explicitly, maximum-likelihood methods appear to be ideal for the analysis of paleoflood and gaging-record information. The following three case studies demonstrate the use of maximum-likelihood techniques in fitting the log-Pearson type III distribution to gaging data and paleoflood information.

Example 1: The Pecos River, Texas

In studies of the Pecos River in western Texas (fig. 1), flood-frequency distributions have been estimated from a long gaging record and paleoflood data near the river's juncture with the Rio Grande (*5, 15, 16, 40*). The gaging record for the Pecos River is unusual because of two extreme floods. Hurricanes in 1954 and 1974 resulted in floods with peak discharges of 27,400 m³/s and 16,100 m³/s near the juncture with the Rio Grande (*5*). When the Water Resources Council methods (*35*) are applied, the two floods are identified as outliers, or floods that do not appear to arise from the same population as the other floods. Estimates of the 100-year flood ranged from 2,300 to 35,000 m³/s using nine different treatments of the data and four methods of estimating flood quantiles from the data (*5*). Estimates of the recurrence interval for the 1954 flood have ranged from 80 to 10,000,000 years (*16*). Lane's (*40*) estimate for the recurrence interval of the 1954 flood ranged from 380 to 85,000 years.

The paleoflood record for the Pecos River consists of 9,600 years of record with evidence of 20 floods above 1,600 m³/s (fig. 4). Because of the potential for nonstationarity caused

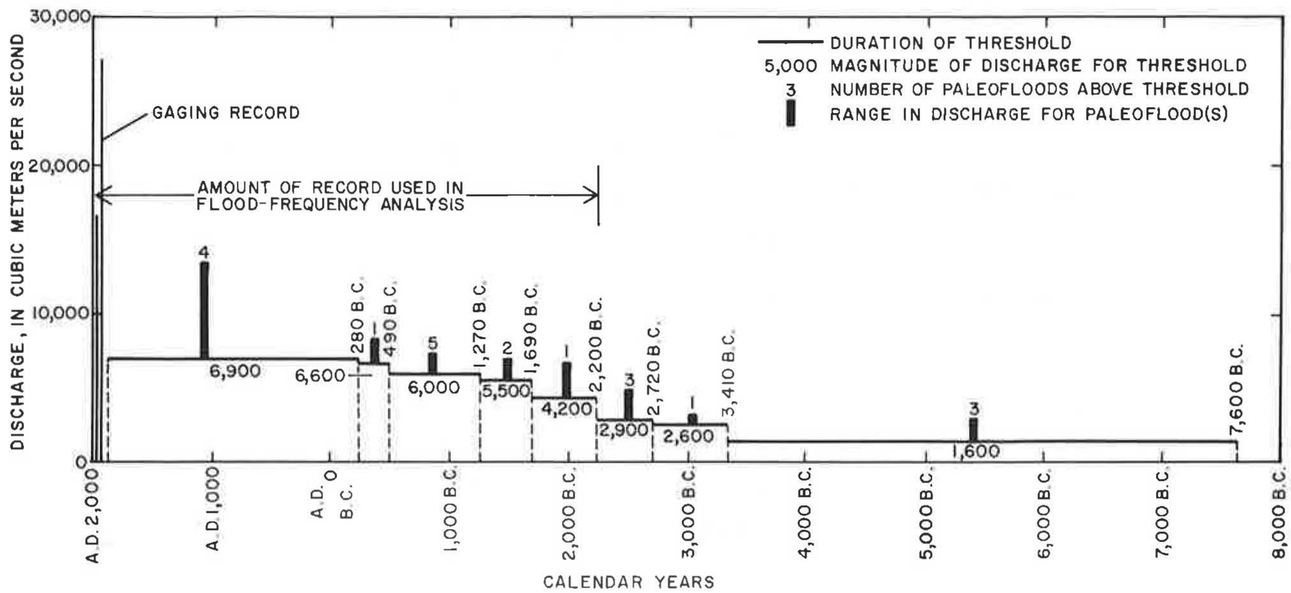

FIGURE 4 The paleoflood record for the Pecos River, Texas.

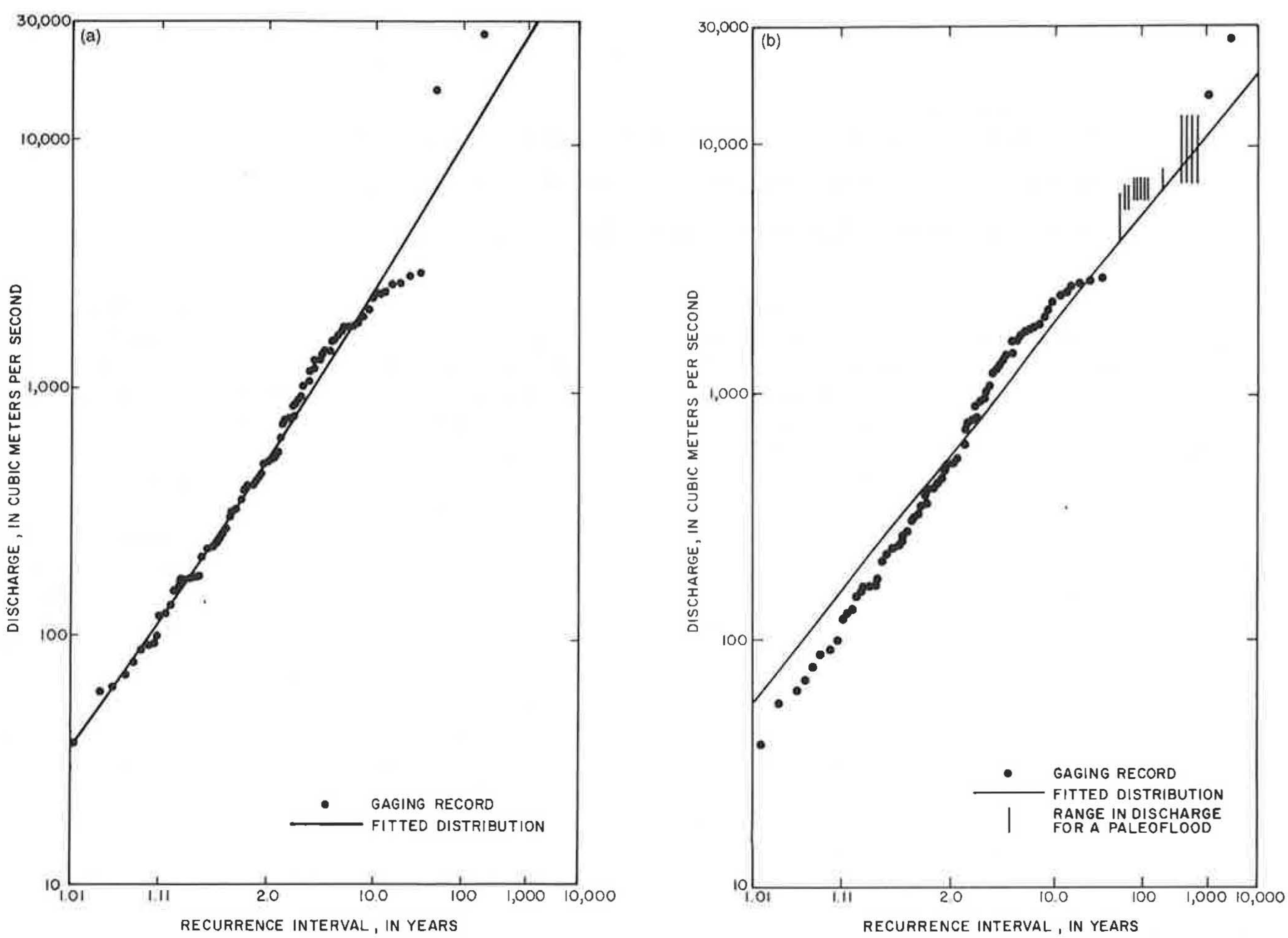

FIGURE 5 Flood-frequency analysis for the Pecos River, Texas. (a) Maximum-likelihood analysis using the gaging record alone. (b) Maximum-likelihood analysis using the gaging record and 4,236 years of paleoflood record as censored data.

TABLE 2 SUMMARY OF FLOOD-FREQUENCY ANALYSES FOR THE ESCALANTE, PECOS, AND DEVILS RIVERS USING PALEOHYDROLOGIC INFORMATION

Record used	Method	Total record length (years)	Mean of log discharge	Standard deviation of log discharge	Skew	100-year flood (m^3/s)
			PECOS RIVER, TEXAS			
G	MM	86	2.74	0.53	0.17	11,200
G	ML	86	2.74	0.52	0.10	9,900
GP	MLC	4,236	2.71	0.39	-0.09	4,400
			DEVILS RIVER, TEXAS			
G	MM	77	2.70	0.76	0.02	30,100
G	ML	77	2.77	0.74	-0.21	23,400
GP	MLC	1,300	2.74	0.53	-0.10	4,500
			ESCALANTE RIVER, UTAH			
G	MM	28	1.29	0.51	-0.50	190
GP	MLC	1,536	1.35	0.51	0.05	370

[G, gaging record only, GP, gaging and paleoflood records, MM, method of moments, ML, maximum likelihood method, MLC, maximum likelihood method with censored paleoflood data. Logarithms are base 10.]

by climatic change during the last 10,000 years (*15, 16*), only the last 4,000 years of the record were used in the maximum-likelihood analysis. Discharges were estimated for paleofloods using Manning's equation and one cross section (*14*). Seven radiocarbon dates were used for dating floods spanning the last 4,000 years. Because the discharges were estimated using Manning's equation and one cross section, and because the water-surface slope was considered equal to or 1.25 times the channel slope, the reported discharges (*14*) were considered imprecise. In order to account for the imprecision in discharge estimates in the maximum-likelihood analysis, all of the floods observed in each time interval were considered to have the same range in discharge. The upper limit of the range was the value of the largest flood estimate that occurred in the time interval. In order to account for the possibility of a lower water-surface slope, the threshold and lower limit of the range were established by decreasing the water-surface slope to 0.75 times the channel slope. The gaging record used in this study is from A.D. 1900 to 1985 (86 years). The influence of dams in the headwaters is considered negligible for peak flood flows (*40*).

Comparison of maximum-likelihood analyses of the gaging record of the Pecos River alone (fig. 5A) and the gaging record with paleoflood information (fig. 5B) indicates a difference in the estimated recurrence intervals for the 1954 flood. The moments of the log-Pearson type III distribution for both treatments are given in table 2. Use of the maximum-likelihood technique with paleoflood information provides a better fit for the 1954 flood with the rest of the population than the fit obtained from an analysis of the gaging record alone (fig. 5B). Standard errors for the 50- and 100-year floods are 10 and 11 percent, respectively, for the distribution obtained using the gaging record. Standard errors for the 50- and 100-year floods decrease to 7 percent for both recurrence intervals when the paleoflood information is included in the maximum-likelihood analysis (table 3). The lowest threshold for the paleoflood record—4,200 m^3/s (fig. 4)—is at the 97 percent quantile on the distribution fitted to gaging and paleoflood data (fig. 5B). Extrapolating from data for three-parameter distributions presented in 29, the effective systematic record length for the Pecos River is as much as 2,600 years when the paleohydrologic and gaging data are used together in the maximum-likelihood analysis.

Case 2: The Devils River, Texas

The Devils River in west Texas (fig. 1) provides a different test for the use of paleoflood data in flood-frequency analysis. The gaging record for the Devils River is 77 years in length.

TABLE 3 SUMMARY OF STANDARD ERRORS FOR THE 50- AND 100-YEAR FLOODS ESTIMATED FOR THE PECOS, DEVILS, AND ESCALANTE RIVERS

	STANDARD ERRORS FOR 50-YEAR FLOOD			
	Gaging record		Gaging and paleoflood records	
River	(m^3/s)	(percent)	(m^3/s)	(percent)
Pecos River, Texas	730	10	240	7
Devils River, Texas	4,520	29	420	6
Escalante River, Utah	80	32	31	12

	STANDARD ERRORS FOR 100-YEAR FLOOD			
	Gaging record		Gaging and paleoflood records	
River	(m^3/s)	(percent)	(m^3/s)	(percent)
Pecos River, Texas	1,130	11	300	7
Devils River, Texas	9,020	38	420	7
Escalante River, Utah	140	43	46	12

[Standard errors are calculated using the methods presented in 41, equations 16-21]

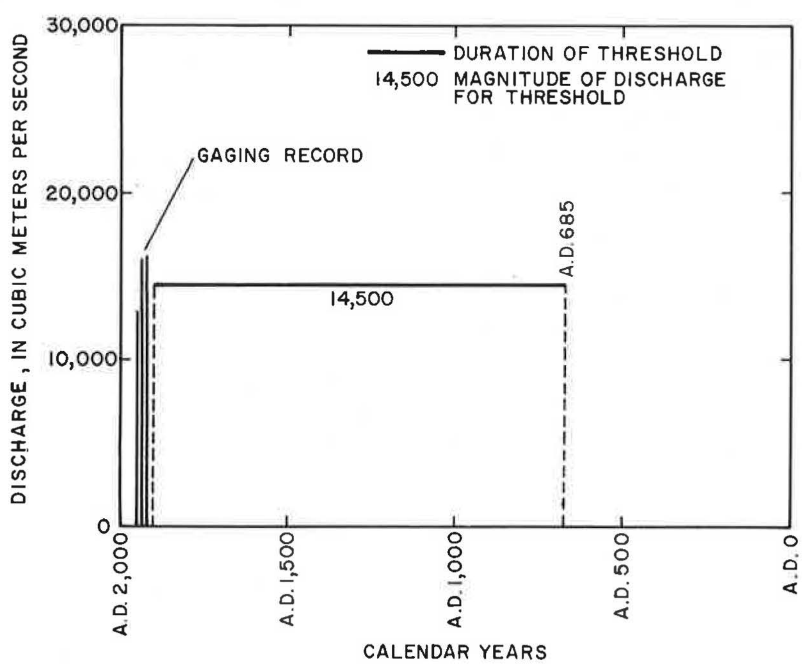

FIGURE 6 The paleoflood record for the Devils River, Texas.

No floods are identified as outliers; however, three unusually large floods of 16,700, 16,400, and 13,300 m³/s have been recorded (*14*). The 100-year floods estimated by the method of moments and the maximum-likelihood fitting are 30,100 and 23,400 m³/s, respectively (table 2). The difference between the two methods is primarily in the value estimated for the skew coefficient. The skew estimated from the Water Resources Council methods is 0.02, whereas the skew estimated for maximum-likelihood analysis is −0.21 (table 2).

The paleohydrologic record for the Devils River is complex and cannot be correlated temporally with the record for the adjacent Pecos River (*14*). Since A.D. 685, however, no paleofloods larger than approximately 14,500 m³/s (fig. 6) have occurred (*14*). This lack of exceedence (negative evidence) can be used to define a nonexceedence censoring threshold in a maximum-likelihood analysis. The resulting flood-frequency relation (fig. 7) indicates that the recurrence intervals for a given discharge are generally greater when the paleoflood information is used. For example, the estimate of the 100-year flood decreases from 23,400 to 4,500 m³/s when paleoflood information is used in the maximum-likelihood analysis. Standard errors for the 50- and 100-year floods are 29 and 38 percent, respectively, for the distribution estimated from the gaging record. Standard errors for the 50- and 100-year floods decrease to 6 and 7 percent, respectively, when the paleoflood information is used in the analysis (table 3). The censoring threshold of 14,500 m³/s is at the 99.2 percentile in the fitted distribution (fig. 7). Effective record length, as extrapolated from 29, increases from 77 to as much as 670 years with the use of the paleoflood evidence, even though no floods were observed.

Case 3: The Escalante River, Utah

The Escalante River in south-central Utah (fig. 1) has a potentially nonstationary record of flood frequency (*42*) that may be better explained by paleohydrologic data (fig. 8). The alluvial tributaries in the headwaters became deeply incised during to flooding between A.D. 1909 and 1940 (*12, 42*). Discharges of four historic floods that occurred between A.D. 1909 and 1932, which were reconstructed using paleohydrological techniques (*22*), are five to six times larger than the largest floods recorded in a discontinuous 28-year gaging record from A.D. 1943 to 1955 and 1972 to 1986. Analysis of the four historic floods with the gaging record using the Water Resources Council methods (*35*) resulted in a 100-year flood more than twice the 100-year flood estimated from the gaging record alone (fig. 9A; *22*). Analysis using the gaging record and an increasingly larger length of historic record, on the basis of historic and paleoflood information that indicates a lack of large floods, resulted in a monotonic decrease in the 100-year flood to that estimated using the gaging record alone (*42*).

The paleoflood record for the Escalante River is approximately 2,000 years, and evidence has been found for nine prehistoric and four historic floods (fig. 8; 22). Paleofloods were dated using 12 radiocarbon dates, tree-ring evidence indicating the lack of floods, and historic records (*22*); and discharges were estimated from one site the step-backwater method. The paleoflood information can be interpreted in terms of censoring thresholds, and discharges can be assigned

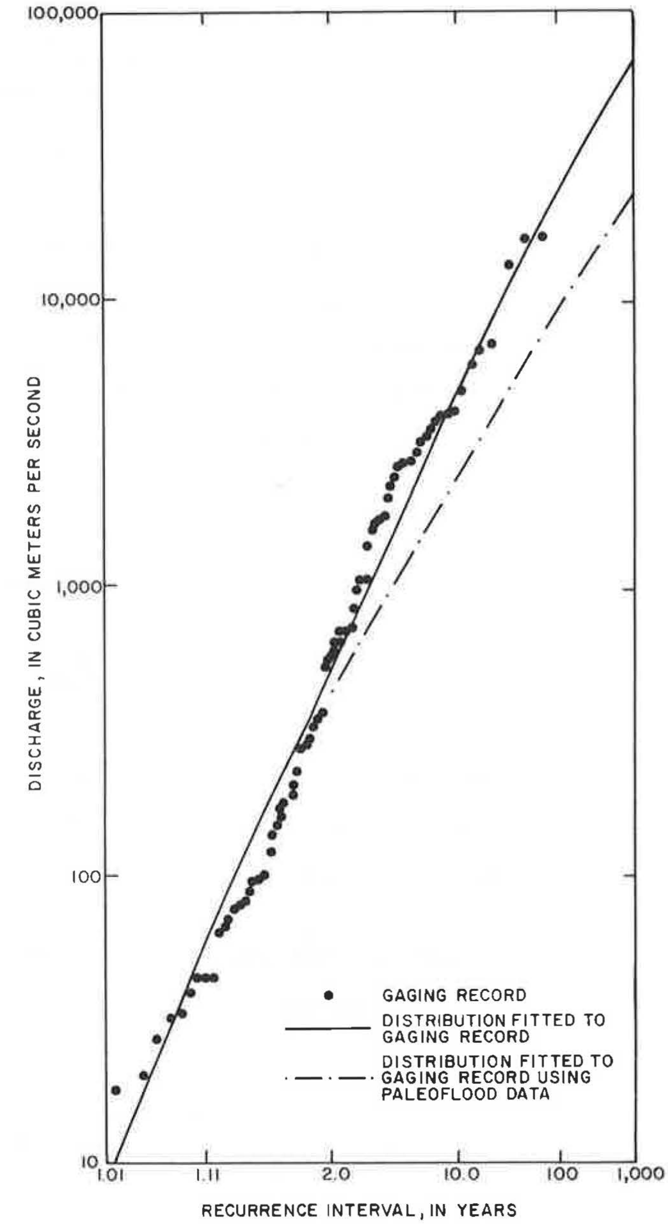

FIGURE 7 Maximum-likelihood analysis of flood frequency for the Devils River, Texas. Plotting positions for discharges are based on gaging record only.

ranges to reflect the imprecision of discharge estimates (fig. 8). Thresholds were established as the discharges required to reach the bases of flood deposits because stratigraphically inset relations in the deposits (see fig. 2) suggest that any floods greater than the bases of the deposits would leave depositional evidence. Discharges for the paleofloods were given ranges according to the uncertainty in the discharge estimates (*22*). Whereas the historic flood record may be nonstationary in the time domain, nonstationarity is reduced in the time and frequency domain of the entire paleoflood record (fig. 8).

Use of the maximum-likelihood fitting resulted in an estimate of the 100-year flood (370 m³/s) that is between the estimates of the 100-year flood from the gaging record alone

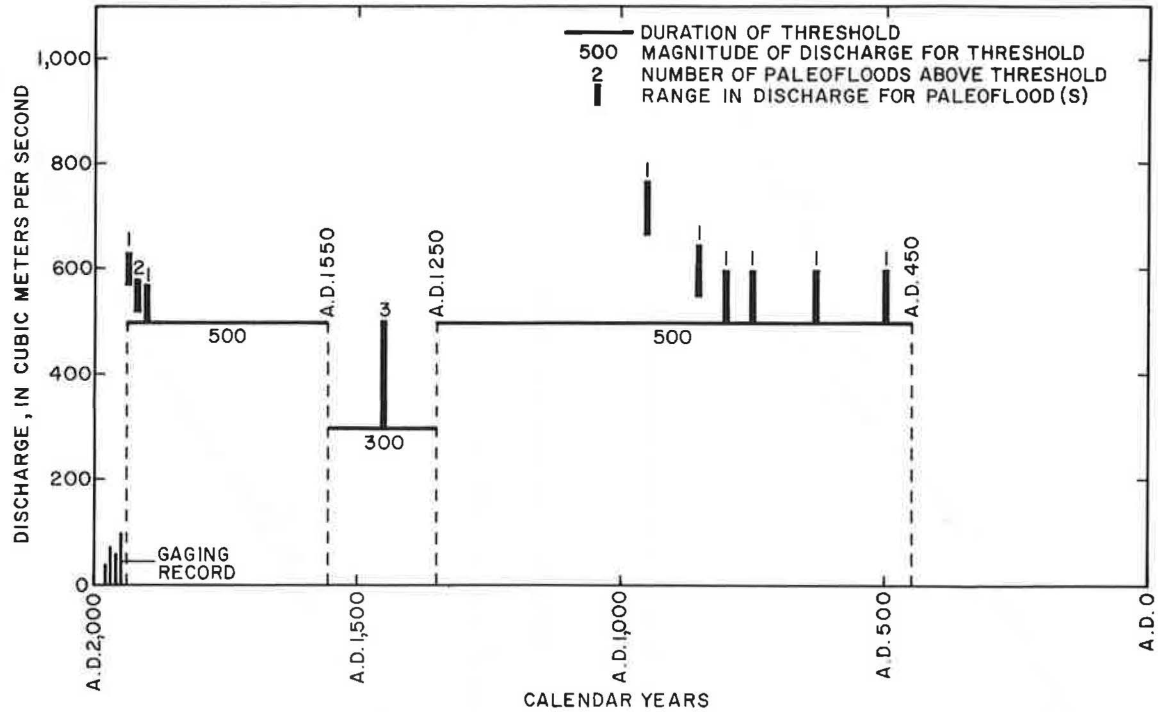

FIGURE 8 The paleoflood record for the Escalante River, Utah.

(190 to 330 m³/s) and from the combination of gaging record and historic flood records (480 to 700 m³/s, (*42*)). The skew coefficient estimated from the gaging record, using the Water Resources Council methods, is −0.5 compared with near 0 or positive skew coefficients estimated from the gaging and paleoflood records (table 2). The contrast between the analyses using gaging data alone and gaging data and historic flood data (fig. 9A) is largely a result of the negative skew coefficient estimated from the gaging data alone. Standard errors for the 50- and 100-year floods are 32 and 43 percent, respectively, when the distribution is fitted using the gaging record. Standard errors for the 50- and 100-year floods decrease to 12 percent when the entire paleoflood record is used. Because the lowest censoring threshold is approximately at the 98 percent quantile, the effective record length, extrapolated from 29, may be as much as 1,000 years when the paleoflood information is included.

DISCUSSION AND CONCLUSIONS

Recent paleoflood hydrologic studies of the southwestern United States use paleostage indicators to reconstruct the dates and discharges for past floods. As the science has evolved, different techniques have been used in the analysis of paleofloods. Early studies used Manning's equation with one cross section, and the water-surface slope was estimated from the present channel slope. The most recent studies have used the step-backwater method with as many as 27 cross sections. The step-backwater method explicitly accounts for nonuniform flow and eliminates the need for assumptions concerning the water-surface slope. Both methods require the assumption of a stable channel, which can usually be demonstrated in bedrock channels.

Methods used in dating paleofloods have not changed as much as methods used in estimating discharges. Radiocarbon dating of organic material entrained in flood deposits is used primarily to estimate the age of paleofloods. Radiocarbon dates are measured with a standard deviation and therefore have an explicit uncertainty. The contemporaneity of floods with the organic material entrained in the flood deposit is the most important source of uncertainty in the dating of paleofloods. Other methods, including dendrochronology, soil development, and correlation of artifacts, also have been used in dating of paleofloods.

Paleohydrologic information, although imprecise, is useful in flood-frequency analyses using maximum-likelihood techniques. As shown in the case study of the Pecos River, imprecise data consisting of large ranges in the possible discharges for floods can still be used in a flood-frequency analysis. Lack of paleofloods, or negative evidence, can also be useful in flood-frequency estimates, as shown in the case study of the Devils River. Finally, the case study of the Escalante River showed that use of paleoflood information may be helpful in flood-frequency analyses of a potentially nonstationary record by increasing the length of record and decreasing the effects of large floods on the shape of the distribution. In all cases, the effective record length of the paleoflood and gaging data would be expected to be several times the length of the gaging record alone.

Use of the maximum-likelihood method gives an appropriate weight to paleohydrologic data in flood-frequency analysis. The discharge of a paleoflood can be expressed as a range instead of a value, which may imply a misleading accuracy. Because maximum-likelihood analysis can use ranges, the uncertainty inherent in paleohydrologic data can be built into flood-frequency analyses. As shown in table 3, use of paleoflood data significantly decreases the standard error of

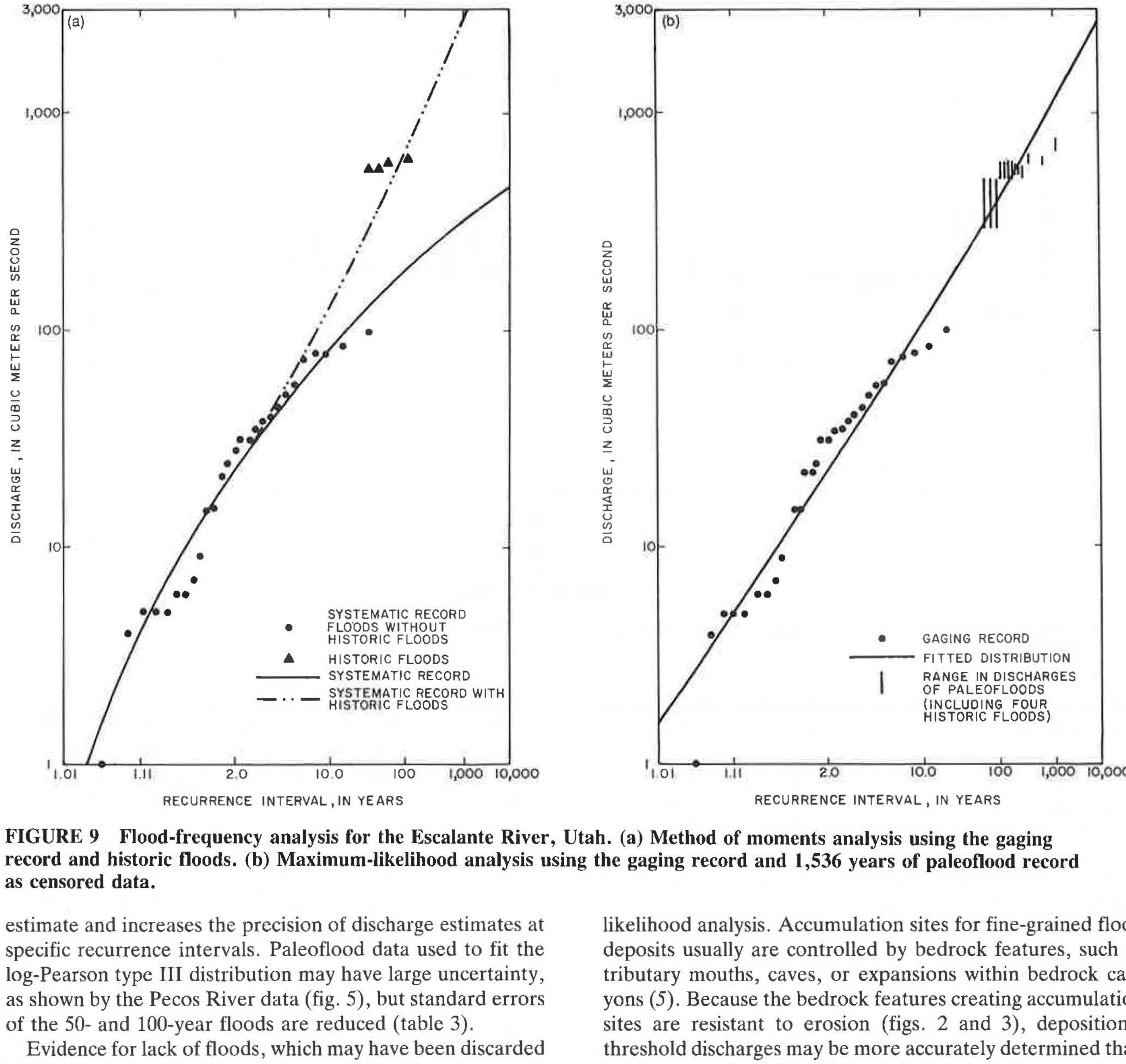

FIGURE 9 Flood-frequency analysis for the Escalante River, Utah. (a) Method of moments analysis using the gaging record and historic floods. (b) Maximum-likelihood analysis using the gaging record and 1,536 years of paleoflood record as censored data.

estimate and increases the precision of discharge estimates at specific recurrence intervals. Paleoflood data used to fit the log-Pearson type III distribution may have large uncertainty, as shown by the Pecos River data (fig. 5), but standard errors of the 50- and 100-year floods are reduced (table 3).

Evidence for lack of floods, which may have been discarded in paleohydrologic studies, may be used in flood-frequency analyses and provide useful information. In the case of the Devils River, lack of large floods over 1,267 years of paleoflood record significantly decreased the estimate for the 100-year flood by 80 percent. The decrease is caused by a decrease in the standard deviation (table 2). The fact that floods above a certain threshold did not occur on a river, as indicated by the lack of paleoflood stratigraphy in certain time intervals, can be important in describing the flood history and estimating flood frequency. If flood-frequency estimates are the desired goal of paleoflood hydrologic studies, the data requirements for maximum-likelihood analysis can be used to guide field data collection. Dating of individual floods is not required; the most important age controls on paleofloods are the beginning and ending dates for discharge thresholds or minimum discharge required for the preservation of paleostage indicators. These thresholds are not usually explicitly determined in paleoflood studies, although this information alone is important in determining the censoring levels for maximum-likelihood analysis. Accumulation sites for fine-grained flood deposits usually are controlled by bedrock features, such as tributary mouths, caves, or expansions within bedrock canyons (*5*). Because the bedrock features creating accumulation sites are resistant to erosion (figs. 2 and 3), depositional threshold discharges may be more accurately determined than discharges estimated from flood deposits.

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Need for New Rainfall Intensity Atlas Analyses in the West

BRIAN M. REICH

Estimates of short-duration rainfall intensities provide critical information for the design of highway drainage systems. Rapid growth of urban population in the arid West has exposed more drivers to the effects of undersized drainage structures. Contemporary stormwater computations on small, semi-arid or urban watersheds require the estimation of rainfall intensities at ten-minute intervals in runoff-producing storms. Earlier methods, such as the rational formula, required less complex rain data. Precipitation-frequency atlases (NOAA-2) for eleven western states were prepared from data prior to 1970 (*1*). These atlases were used daily rainfall observations, regression against topographic parameters, and hydrometeorological considerations to draw isohyetal maps for 24- and 6-hour intervals. The 24-hour maxima may follow topographic features because they usually occur in winter. Visual observation in Arizona does not confirm this to be true with summer convective storms. These produce design floods on semi-arid watersheds smaller than 100 or 200 square miles. Regional variation of short-duration rainfall may be less sensitive to desert mountains than the 24-hour maps developed in NOAA-2. The shortest duration analyzed for NOAA-2 was one hour, used at about ten percent of the sites. For example, in Arizona only 38 autographic recorders were used, over periods averaging less than 20 years. Re-analysis in that state could consider recording gages from over 100 additional sites, for durations as short as 10 minutes. Two decades of new data at NOAA-2 sites could now reduce the error of estimate at these points. Improved statistical methods are also available. Highway and other engineers in ten Western states would gain from new analyses of short-duration intensity data available from recording rain gages operated by many agencies. This paper attempts to serve administrators, drainage engineers, and others who may consider the implementation of re-studies in a western state. The 16-year-old western-rainfall intensity atlas should be replaced after the short-duration recorder data now available have been analyzed. High intensities for durations as short as 10-minutes are particularly important to Western drainage design. Analysis of such data was absent from NOAA-2.

Urban hydrograph models and modern computer synthesis of flood peaks from small (less than about 100 sq. mi.) desert watersheds depend on rainfall input. Rainfall intensity during flood-producing storms, varies rapidly in time and space. Watersheds are being rapidly urbanized, and heavy urban highway use is increasing during times normally associated with intense rainfall (usually late afternoons).

Many high-priced developments border on highway rights-of-way, and property owners fear that improper highway design will lead to flood damage. Pavement drainage, culvert sizing, storm drains, channel design, bank protection, and bridging of larger streams could all be improved by better rainstorm data. Regional flood-frequency studies for large non-urbanized watersheds also require rainfall intensity as an independent variable when regressing stream gage flood estimates against environmental parameters.

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COMMUNITY INTEREST IN RESTUDY THROUGHOUT THE ARID WEST

Surveys of the need to improve rainfall-intensity manuals are becoming common in southwestern communities. Phoenix, Arizona, had such an investigation (*2*) performed at the same time that Maricopa County, in which it is located, commissioned another engineering firm to perform a similar investigation (*3*). In a Clark County, Nevada, study (*4*), frequency plots suggested a 45-percent increase in short-duration rainfall. The study recommends that "NOAA should periodically update the atlases where the period of records is short." Re-analysis of 45 years of National Weather Service (NWS) data at Billings, Montana, showed that 5- to 15-minute estimates for 5- to 100-year frequencies were 1.6 to 1.3 times greater than NOAA-2 values respectively (*5*). Many more examples could be given of the concern felt by local communities in relying entirely on NOAA-2 for small-watershed designs.

A rash of independent studies poses other problems. Many cities or counties have more rain gages than they are able to analyze, but one Arizona county comprising over 10,000 square miles has not even one gage! Very few small jurisdictions have the specialized personnel to either select or adequately supervise a consultant for this work. Analysis of data from a single gage, even if performed correctly with appropriate statistical methods, lacks the validation provided by a regional study. Individual point values may reflect unfortunate sampling error. A state-wide approach would overcome this problem and facilitate standardized data collection and processing. This scale of effort should engender close cooperation with other agencies with data sources, including the NWS, and be able to achieve the necessary conformity at state boundaries.

JUSTIFICATION FOR STUDY OF NEWLY-ACQUIRED RAIN DATA

Accelerated urbanization in the southwest is constantly increasing driver-exposure to hazards associated with high rainfall intensities. The urgency was much less serious when NOAA-2 was planned, almost two decades ago. At that time the rational formula prevailed, and urban hydrology was in its infancy. Investigations of flash floods, as well as the short duration and the compact aerial extent of desert convective storms, were at an early stage.

In the past twenty years, design engineers have become equipped with powerful hydrology software for personal computers. Runoff computations can now use data comprising spatially and temporally varied rainfall intensities for durations far shorter than one hour, the period used for the NOAA-2 atlases. Highway-drainage engineering for the future justifies new analyses of the large pool of break-point rainfall data that has accumulated in the last three decades. Deterministic runoff models might lull users into a false sense of security, if used with the short-duration rainfall estimates derived from NOAA-2. Today's technology deserves updated regional analyses of the increased number of recording rain gages and years of data. A recent pre-print (*6*) indicated possibilities. The number of stations for individual states included: Arizona, 5; New Mexico, 2; Nevada, 4; and Utah, 5. The authors of a similar study promulgated a finer delineation by dividing California into eight regions (*7*). It used 250 stations, averaging more than 27 years.

RE-STUDIES OF OLD DATA, ONGOING NWS CONTRIBUTIONS

The eastern portion of the United States did receive a restudy (*8*), although its previous database (*9*) was much more substantial than is presently available for the arid west. Likewise, California updated NOAA-2 three years after its release. In fact, that State produced its own three-volume study (*10*). Other restudies are more numerous than generally known. Many western small watershed hydrologists only know of NOAA-2. Some are aware of Hershfield's (*9*) "Rainfall Frequency Atlas of the United States" (TP-40). Recently, NWS published a very useful Technical Memorandum, which confusingly also has the number "40" in its title HYDRO-40 (*11*). It extends some of the depth-area work of the Agricultural Research Service (*12*) across all of Arizona and the western half of New Mexico. NOAA Technical Report, NWS 27, deals with inter-durational storms, between one and 24 hours (*13*). Most highway drainage designs in the arid West involve durations of less than 60 minutes.

OPPORTUNITY TO AUGMENT NWS DATABASE

Except for California, the eleven western states are poorly equipped with recording rain gages. Consequently, data shortage was a basic problem when NOAA-2 was prepared for the U.S. Soil Conservation Service using pre-1970 data (*1*). Some western states are have fewer recording rain gages than others. The NWS maintains fewer than 40 recording rain gages in each of Utah, Nevada, and Arizona. Most intense summer rainstorms in the Southwest cover very small areas. Fortunately, longstanding experimental rangeland and desert watershed research has been pursued for many decades by the Agricultural Research Service (ARS), the U.S. Forest Service (USFS), and several universities. These scientific endeavors have included operating dense networks containing many recording rain gages spaced only a few miles apart. Most of these dense networks are located at higher elevations than the rain gages in the NWS network. They contain information on areal and temporal characteristics of short-duration, flood-producing storms that should be analyzed and presented to designers. Some of these data fill regional voids in the statewide NWS networks, and there is a need to process it in a consistent format.

Arizona, for which a state-of-the-art study was recently completed, is used as an example (*14*). Figure 1 shows the 38 NWS recording rain gages from which 731 station-years of data were used in NOAA-2. Only 29 stations are maintained by the NWS today (fig. 2). Data were limited by being tabulated at the end of each clock-hour. On average, each gage sampled 3,100 square miles. The information shortage is made worse by the rare occurrence and spotty nature of heavy rainfall centers in an arid region. Research in southeastern Arizona showed, for 30 minute rains, that the highest point rainfall can be more than twice the average depth across an 80-square-mile storm (*15*). NOAA-2 was limited by a gross spatial network, which missed many larger point values. Moreover, the average record length of less than 20 years produced unreliable estimates at each gage site, particularly for less-frequent events such as a 50-year storm.

Today, 104 recording gages are operated by agencies throughout Arizona (fig. 3 and table 1). Some of these data are digitized each time storm intensity changes (break-point data). This is the type of information needed by designers. Future analysis should focus on processing data in "break-point" format. Longer histories and additional sites add up to about 3,000 station-years of additional data in Arizona alone. The longer records will provide more reliable 100-year point estimates.

NWS has added, or replaced weighing-bucket chart recorders with, Fisher & Porter punched paper-tape gages at 17 sites (indicated by closed circles in fig. 2). Table 1 shows that other agencies have increased the number of recording gages in Arizona markedly. Many are located in areas that supplement the NWS network, such as the Mogollon Rim (an extensive topographic feature with relatively high annual rainfall) and southeastern Arizona closer to the Gulf of Mexico moisture source. Since some of the older records were fairly long before they were terminated, an Arizona re-study could use about a hundred recording rain gages (fig. 3).

ANTICIPATED RESULTS OF NEW RAINFALL INTENSITY STUDIES

In addition to providing western states, counties, and local government agencies with updated analyses from an expanded database of short-duration rainfall measurements, the contemplated investigation should yield additional design tools to those found in TP-40 or NOAA-2. Research in Arizona has shown that flood peaks and flood hydrographs are heavily influenced by the high-intensity rainfalls that last for thirty minutes or less (*12*). With expanding urbanization in the desert southwest, these flash floods may become more frequent and severe. Expanding population means that more people will suffer the effects of mudflows and floods. Therefore, financial and societal benefits will result from regional studies of rainfall recorded autographically during the last few decades.

Second, the dichotomy which presently exists between the

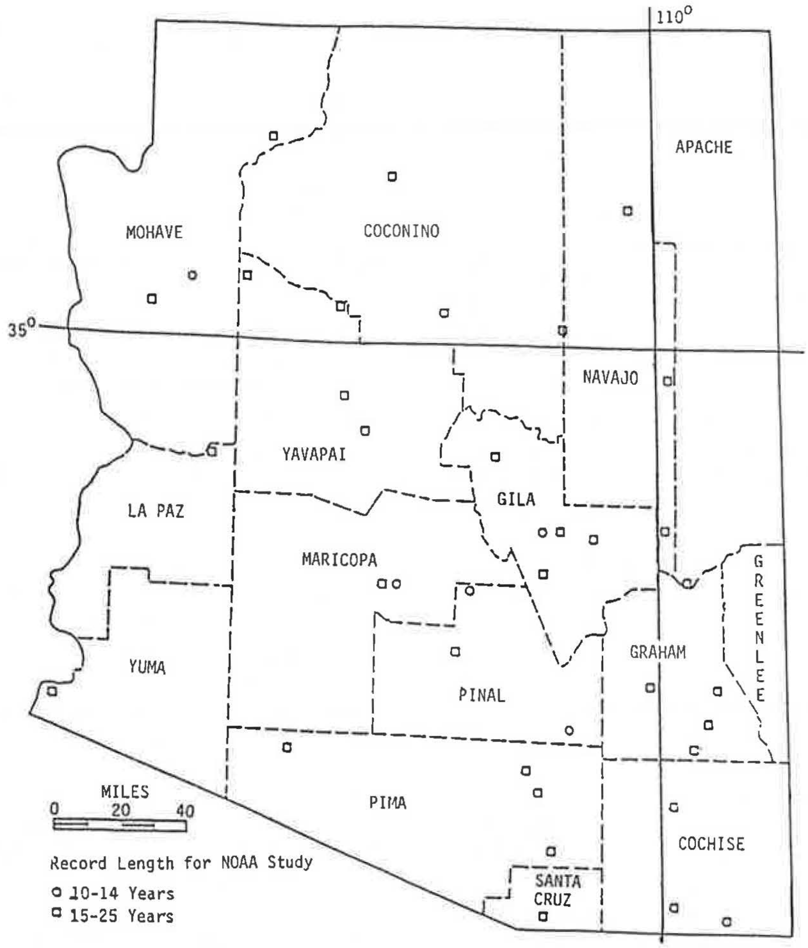

FIGURE 1 Arizona sites of NOAA-2 clock hour data use.

two latest NWS IDF-atlases, TP-40 and NOAA-2, can be resolved. Reference is made to the introduction in NOAA-2 of complex isolines (fig. 4), which suggest large differences in rainfall intensities switching back and forth many times over small distances, even within a county. Application of these contorted isolines requires considerable time, and enhances the opportunity for error in engineering or hydrologic design. Paucity of short-duration data in the NOAA-2 study, coupled with the apparently random spatial occurrence of thunder storms, leads observers of desert phenomena to question NOAA-2's high, short-rainfall maxima on lone desert mountains. Some engineers prefer TP40's smoothed user-friendly maps (fig. 5).

Third, statewide studies would reduce the tendency for smaller communities to re-analyze update records of one local gage.

A fourth benefit of a unified study is that it would provide an opportunity to consolidate the expert knowledge of scientists whose missions are other than highway design (*16*). These researchers might offer valuable advice and possibly solve some highway design problems. One component of the proposed projects would be to develop hydrology/hydraulic engineering manuals from research results that might otherwise be only partially reported in scientific journals.

A final benefit to highway engineers from the proposed research is a digital database, allowing instant site-specific answers to be obtained from a personal computer. This would greatly reduce time and errors in executing drainage calculations.

PRIME REFERENCES AND CONCEPTS OF DESIGN RAINFALL

A scientific review recently reported 137 papers dealing with the analysis of storm rainfall and hydrometeorology (*14*). About one-third of the references included Arizona data. A similar proportion concerned other parts of the arid, western United States. The non-arid United States and overseas studies comprised a quarter of the documents, and the remainder dealt with selected topics in hydrometeorology—primarily the interaction of rainfall with regional topography in the interpretation of statistically anomalous results. This body of physical science can provide essential help in drawing isolines among the sparse network of recording rain gages (*17–19*).

Time Distribution Within Storms

State-side, unified studies would also provide new methods for estimating intensities within design storms situated over small watersheds. Sequences of rainfall increments within a

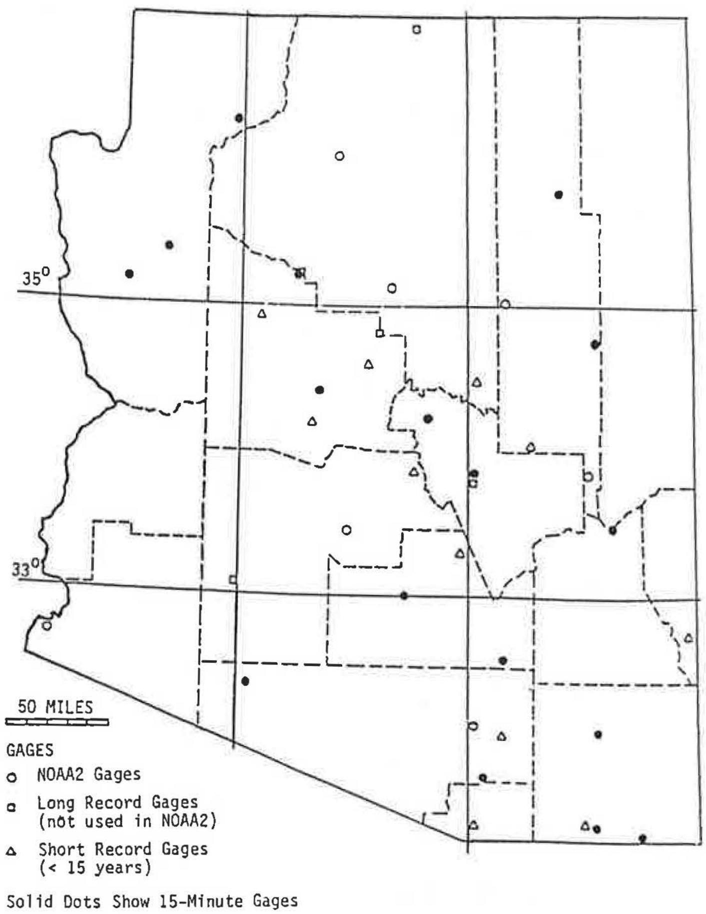

FIGURE 2 Various types of Arizona NWS recording raingages, 1987.

runoff-producing storm provide necessary input for deterministic hydrograph models. Such sequences may be used on ungaged urban or rural watersheds to predict flood volumes, flood peaks, and flood forecasting algorithms, as well as for predicting scour or deposition via computerized sediment transport models. A recently produced national design storm involved data from only five Arizona sites (*20*). The design storm values were based on an average of 30 rains per year at each site. The time-distribution of many small events represents an entirely different universe from the few flood-producing storms of interest to the highway designer. New investigations of flood-producing storms, based on the large pool of break-point data available from various agencies, are needed in the west.

It seems appropriate to take advantage of the additional number of rain gages and their longer records. State-wide or regional efforts to digitize the many unprocessed rainfall recorder charts for 10- through 60-minute and 2-, 6-, and 24-hour annual maximum series should begin. In Arizona, the database has quadrupled since NOAA-2 was developed. NOAA-2 analysis was restricted largely to daily observations, supplemented by clock-hour accumulations at a few short-record stations in all western states, except California. Today, investigations could analyze much more short-duration data in the ten western states.

TP40: An Early National IDF Atlas

In 1961 the Weather Bureau of the U.S. Department of Commerce published Technical Paper No. 40 (TP40), "Rainfall Frequency Atlas of the United States, for Durations from 30 Minutes to 24 Hours and Return Periods from 1 to 100 Years" (*9*). For its time, this was a monumental effort. For the entire country, it used 2,081 stations that had clock-hour data. The records were for the period 1938 through 1957, none being less than 5 years. The Arizona subset comprised 38 stations. A nationwide subset of 200 stations was used to interrelate rainfall intensities for intervals as short as 30 minutes by establishing average relationships to clock-hour amounts. Sixty-minute amounts are distinguished from clock-hour amounts in that the former represent the maximum 60-minute depth regardless of when the continuous period occurred. A relationship was developed between the 1-hour depth and the 6- and 24-hour depths for return-periods from 1- to 100-years.

Mapping relied on a larger network of 6,185 daily gages nationwide. Maximum annual rainfall data for selected durations at each of these sites yielded a series. Each of these was then fitted by an extreme value (EV) probability distribution (*21*). Thereafter, generalized relationships between the daily estimates and those for durations of 30-minutes, 1-, 2-, 3-, 6-, 12-, and 24-hour estimates and 1-, 2-, 5-, 10-, 25-, 50-, and

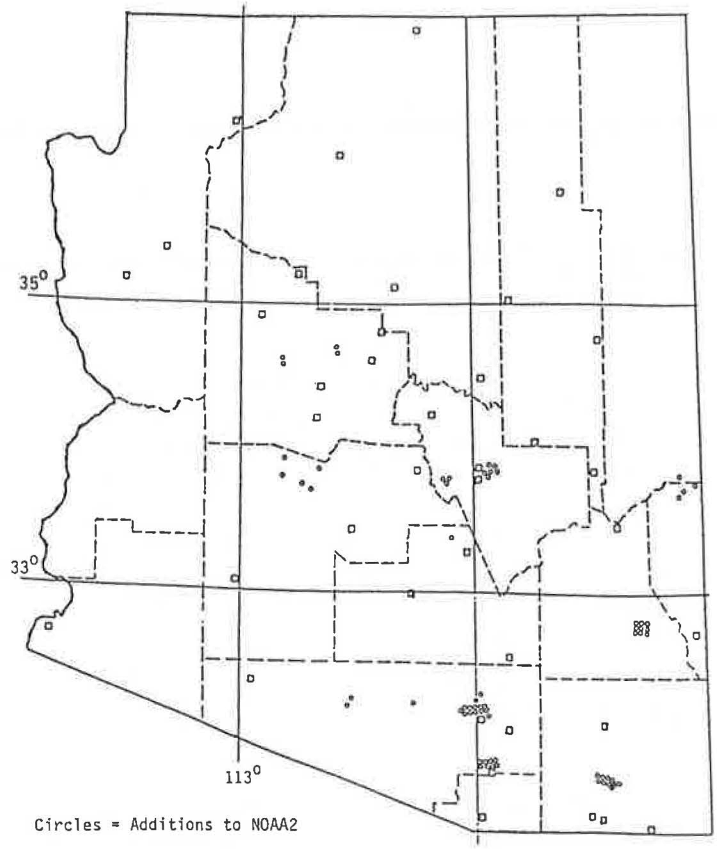

FIGURE 3 Recording raingages in Arizona available for a new study.

100-year frequencies were used to complete the atlas. The isolines were smoothed through the national network of daily rain gages.

Arizona was represented on the completed maps by about five square inches. For example, the 100-year 1-hour (P100y1h) isoline for 3, 2.5, 2, and 1.5 inches depicted four smooth, continuous, concentric areas (see fig. 5). This early report provided a useful, user-friendly design manual. In the pre-computer era, it extracted a high level of information from a minimal history of short-duration rainfall-intensity measurements. The transition of storm estimates across Arizona was sufficiently gradual that most countries were contained within a pair of isolines. For example, the 1-hour 100-year return period changes gradually from 2 inches on Maricopa County's western boundary to 3 inches at its eastern extremity. The isoline trends are north-south, with very gentle curvature.

TABLE 1 RECORDING RAINGAGES IN ARIZONA WITH ADEQUATE DATA FOR ANALYSIS IN 1987

Agency	No. of Raingages
NWS	38
USFS	17
ARS	26
UOA	14
Various	9
Total	104

NOAA-2: Computer Attempt to Synthesize Inside Same Gages

NOAA-2 had the advantage of mainframe computers and about ten years of additional records in developing maps for eleven western states. In a similar manner to TP40, this study provided some benefits to each state from stations beyond its borders. The NOAA-2 study developed relationships between 24-hour maxima and the following factors: terrain slope, annual moisture, location, and roughness. The equations were applied to a dense grid on topographic maps. This information was subsequently fitted by tight, contorted isolines for every two-tenths of an inch. NOAA-2 presented a set of very detailed synthetic maps for selected return periods (2-year to 100-year) for both 6- and 24-hour durations. The maps of each state were about 110 square inches in area. The effect of this high degree of preciseness on potential users can be appreciated by examining figure 4 for P100y6h. Hydrologists who endeavor to determine floods from NOAA-2 are confronted with averaging among the intricately scalloped isolines for P100y6h or similar maps from the atlas. Nevertheless, some of the overall trends that are apparent despite the detail have been explained elsewhere on hydrometeorological grounds (*17*). For instance, the decrease in isohyetal rise in the northeast quarter of Ari-

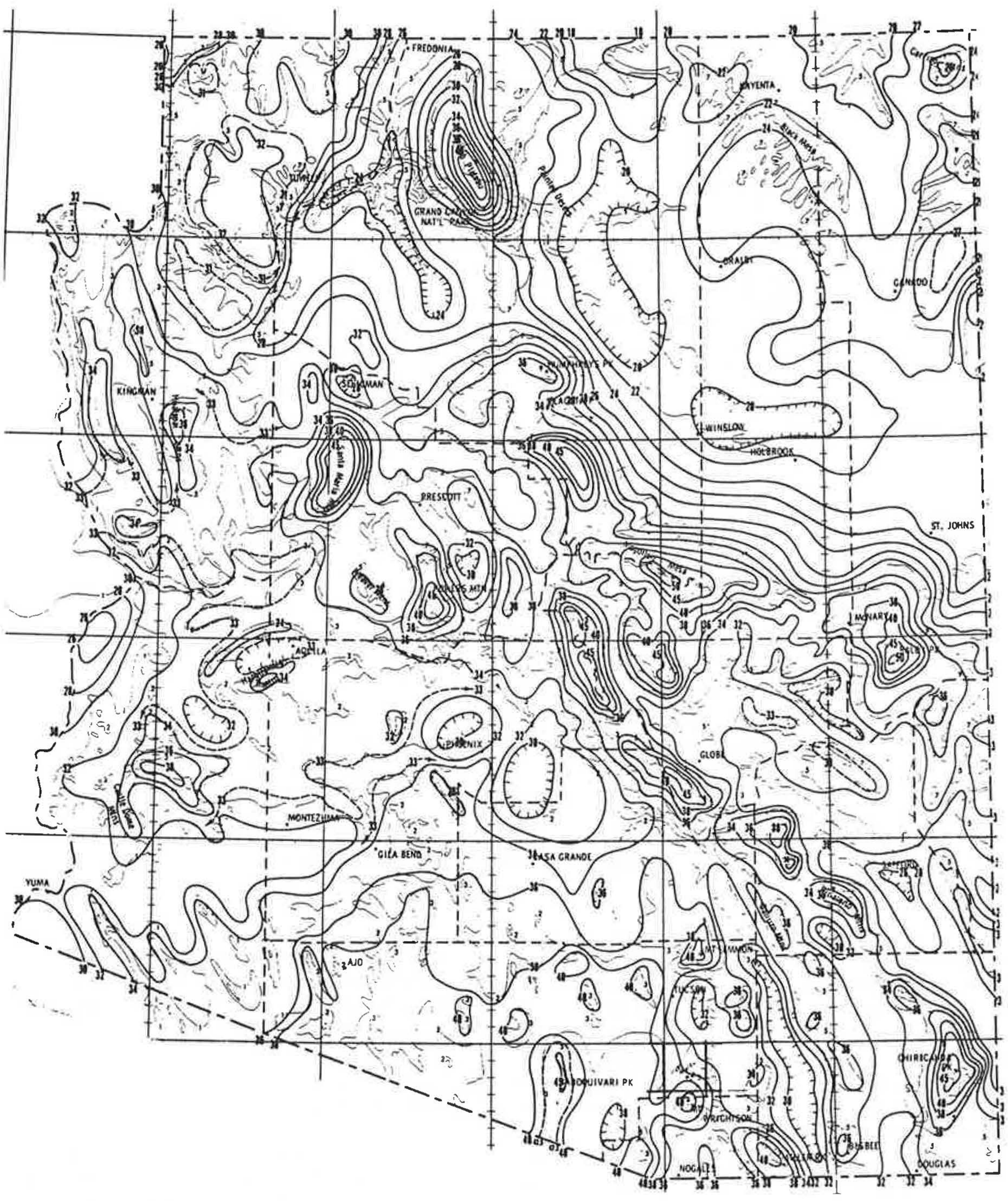

FIGURE 4 NOAA-2 map for 100-year 6-hour rainfall, tenths of an inch.

zona on the lee of the Mogollon Rim is caused by atmospheric moisture depletion.

Hydrometeorology

Excellent descriptions of the physical processes responsible for major storms were also given in Hydrometeorological Report 49 (*17*). That report included 26 major parts of five states. Of most importance to the highway program is the section describing ideal storms, which are defined as "heavy rains, exceeding 3 inches in 3 hours or less, that are reasonably isolated from surrounding rains." It is very unlikely for such a local storm, much less its epicenter, to occur in a rain gage of the sparse NWS network gages. One of these storms produced 8 inches within 45 minutes near Fort Mohave, Arizona. This HMR 50 (*18*) also discussed "cloud mergers," where synergistic effects produce far greater rain than the sum of the water from separate clouds that do not collide. These complex mechanisms produce very heavy short-duration rain

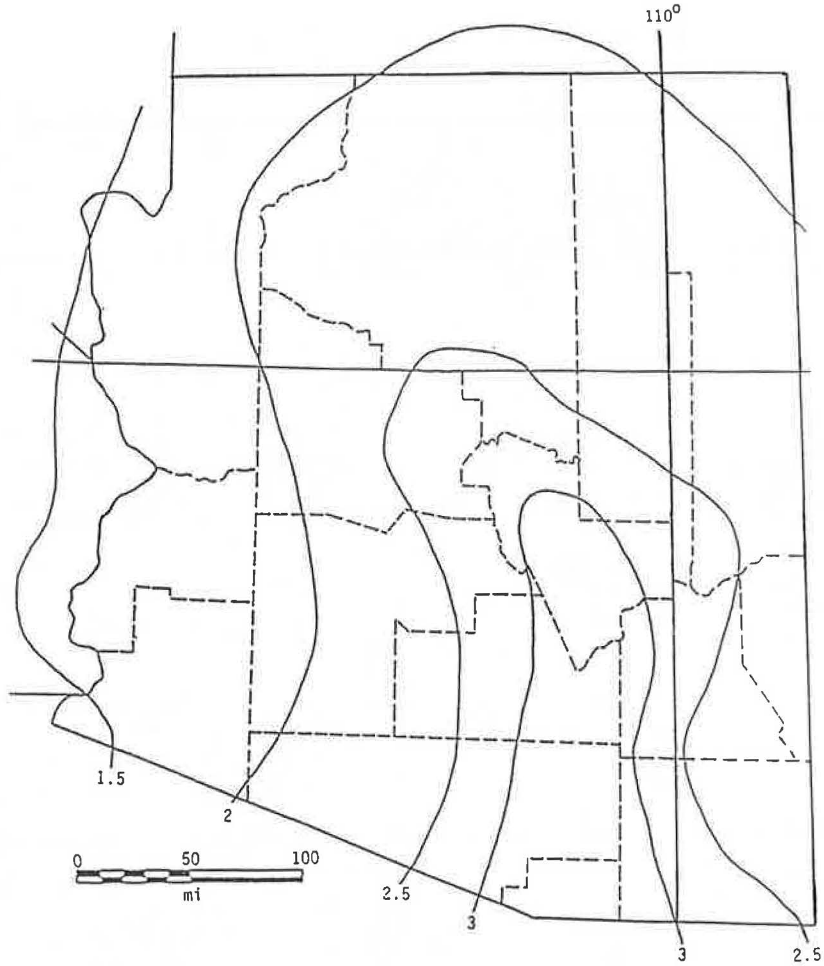

FIGURE 5 TP-40 map for 100-year 1-hour rainfall, inches.

which affects the ever-expanding road network. These violent, local, short-duration storms produce floods in the huge number of watersheds smaller than about 100 square miles (*22*).

Future analyses should consider hydroclimatological zones for which single values apply. Watersheds straddling a number of isolines can be simply handled as the design point moves close to zonal boundaries. If the proposed Arizona analysis is able to solve these problems, other arid states will be able to plan their re-analyses of short-duration rainfall intensities more effectively.

Time Distribution and Areal Reduction

Studies are also needed that include the interduration variation of rainfall amounts for different durations and frequencies and the reduction of point values to an area. Normally, hydrologic design involves the order in which amounts for selected durations occur, in addition to the average depth of precipitation over a particular drainage area.

The southeastern U.S. was fortunate to get a study (*23*) that meets such design needs. It suggests that the short-interval interdurational variability of rain storms in the southwestern United States is critical. Several studies of within-storm rain at Walnut Gulch Watershed in southeastern Arizona have been prepared by the ARS (*15*). Data from additional small networks in the west should be studied.

CONCLUSIONS

There is a tremendous amount of short-duration rainfall intensity data available today for western states compared with what was available for the NOAA-2 publication in the early 1970s. Moreover, that study relied heavily on daily rain gage data. Today, there is great need for estimating rainfall intensities for the 10-, 15-, and 30-minute durations with which arid-region flood peaks and hydrographs are strongly correlated. Highway hydraulic engineers will be able to enter the microcomputer era of hydrologic design only if the temporal and spatial properties of storm rainfall are described quantitatively. The technology to process and statistically analyze these data is available. It has been estimated that about one-third of a million dollars would be needed in order to perform such a task for Arizona in the next two years (*14*). Most other western states have less data and may require less funding.

Some economies could be achieved by consolidating studies for several western states. It is time that this third of the country has an updated rainfall design atlas, such as has been available for the eastern United States for over 10 years (*8, 22*).

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Peak-Flow Data-Collection Methods for Streams in Arid Areas

Ernest D. Cobb

This paper describes methods of determining peak streamflows in arid areas, where unstable channels and ephemeral flows characterize many streams. Where these exist, special methods of obtaining peak flows are needed. Usually, flow is determined from a recorded stage and a relational curve of stage and discharge. In unstable channels, it may be difficult to obtain a peak stage because the flow may move horizontally away from the gage, or the sediment in the flow may bury the stage sensor. If the peak stage is measured, the flow may be difficult to determine because of the unstable rating. As a result, the peak flow determination for streams with unstable channels commonly has a high degree of uncertainty. A variety of techniques are used to reduce the uncertainty of peak-flow data. A variety of instruments that are used to measure a peak stage, streambed elevations, and velocities can improve data collection in some, but not all, streams. Attempts can be made to stabilize some channels in the area of measurement. Systems that alert field personnel when stream flow is high allow time for site measurement of discharge and verification of stage measurements. Ephemeral streams create special problems in maintaining the equipment during the dry season so that it will operate during periods of flow. Also, human activities such as bridge maintenance in the dry channels can add to the instability of the channel.

The purpose of this paper is to discuss methods used in determining peak streamflow in arid areas and conditions unique to collecting peak streamflow data in these areas. Some common characteristics of streams in arid areas are that many of them have unstable channels and that they frequently go dry. Gaging streams in arid areas with stable channels is in many ways no different than gaging those in humid areas.

A usual method for determining the peak flow at a site is to develop a stage-discharge relation (as shown in fig. 1), determine stage of the peak, and, from the relation, determine the peak discharge (*1, 2*). In streams with unstable channels, the stage may be difficult to monitor, adequate discharge measurements may be difficult to obtain, and the stage-discharge relation is unstable. The uncertainty of the peak flow is therefore often much greater for streams in arid areas than for streams in other areas.

Because the method of determining the entire flow hydrograph is often similar to that of determining the peak flow, this paper will generally refer to methods of collecting flow data in streams with unstable channels. It is also noted that there is an increasing demand to define discharge for the entire hydrograph.

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GAGE-SITE CONSIDERATIONS

The location of the gaging site will be restrained by where the data are needed. This limits the stream reach that can be considered. Access and stream-channel characteristics are of primary concern within this reach. The gage site needs to be accessible under most conditions so the gage can be maintained, discharge measurements made, and data can be obtained from the gage. Channel and control stability are highly desirable features for a gaging station. There also needs to be a place nearby where discharge measurements can be made. If at all possible, the flow will be in a single narrow channel at all stages to provide for a sensitive rating.

Bridges are frequently selected as gaging sites because of easy access and because the bridge can sometimes provide a stabilizing effect on the stream channel. Furthermore, a bridge can sometimes provide a place from which to make a discharge measurement and on which to mount the gage structure.

On the other hand, bridges can, in places, create situations that are not hydraulically acceptable for a gage. Drift tends to collect on piers and the bridge may be at a large angle to the flow, which makes it difficult to take measurements. High velocity flow through bridges may accentuate scour in the measurement section.

There may be regulations against mounting a gage structure on a bridge, and bridge traffic can make for measurement hazardous if traffic is heavy and (or) the bridge is narrow.

Sometimes a gage is placed on or at a bridge, but, because of regulations that restrict work on the bridge, safety problems, or problems with flow conditions at the bridge, high-water discharge measurements are made from a boat or a nearby cableway.

It is important that all flow past the gage be in a single channel. If it is in two or more channels, the stage record obtained in one of the channels may not be representative of the stage or discharge in the other channels.

The flow should be tranquil past the gage, if at all possible. Fast, turbulent flow may result in pileup or drawdown at the sensor, and this can create problems in getting an accurate stage record.

An ideal gage site is seldom found, especially in unstable channel streams. Cost may be an overriding factor in site selection, and compromises often must be made with many of the factors that make a gage site desirable. Commonly, there are no sites on a stream reach having all, or even most, of the desired characteristics; and the best site is selected with little regard for cost.

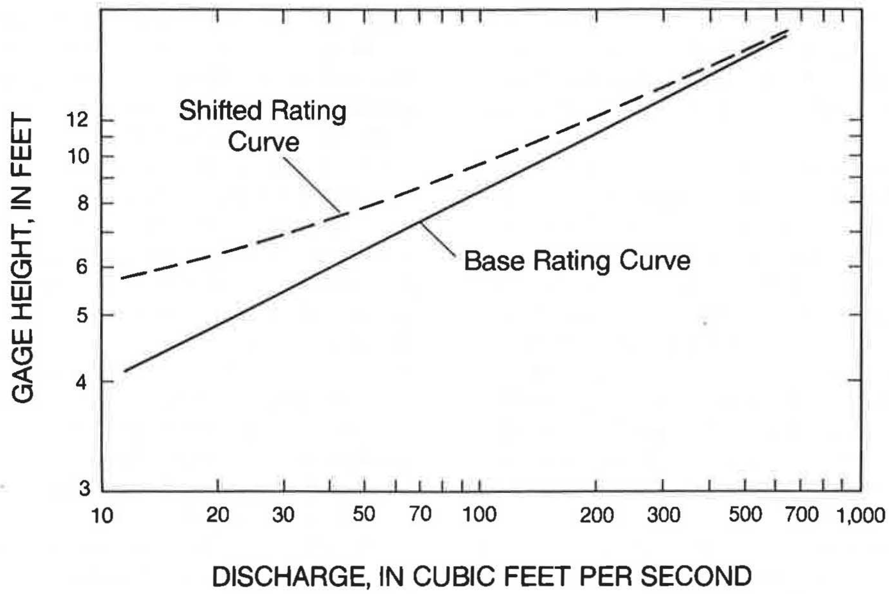

FIGURE 1 Base rating curve and a rating curve resulting from aggradation of the stream channel.

STAGE MEASUREMENT

Probably the most common method of monitoring stage is with a float and recorder in a stilling well (*2, 3*). The well may have openings directly to the stream, it may have intakes from a well set in the bank to the streamflow as shown in figure 2, or the bottom of the well may be open to the stream.

Streams in arid areas commonly transport large sediment loads. Stilling wells that are directly open to the stream may quickly fill with silt and have to be cleaned frequently. Wells with intakes can have problems with the intakes becoming buried or filled with sediment. Stilling wells are usually constructed with a sump or storage area below the lowest intake to hold the sediment that accumulates between cleanings. A trap may also be located in the intake system to collect the sediment before it gets into the well. Wells with the bottom open become plugged only if the channel fills with sediment around the gage.

Well intakes may be cleaned out with a special flushing system attached directly to the intake pipes. At times, the

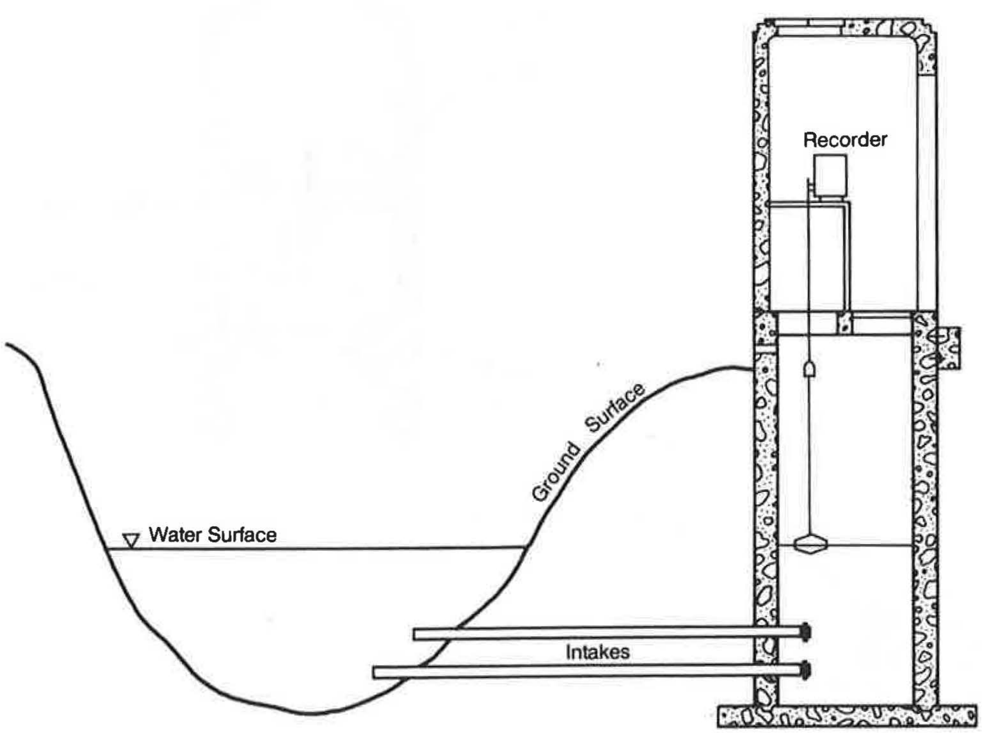

FIGURE 2 Typical stilling well with intakes installation.

intakes and wells are both cleaned out by pumping water directly into the well until the pressure of the increasing hydraulic head forces the sediment out of the pipes. Sometimes, the sediment in a well must be bucketed out of the well.

Another method of monitoring stage is to use a bubbling gas system and a manometer (*2, 3*). The system bubbles gas through a line into the stream, as shown in figure 3, while the manometer measures the back pressure resulting from the head of the water over the bubbler orifice. This system has some obvious advantages over the stilling-well system. There are no intake pipes or stilling well to fill with sediment. If the bubbler orifice becomes buried, it is a relatively easy matter to move the orifice. A related advantage is that the orifice can readily be relocated if the flow moves laterally across the channel. The use of the bubbler-manometer system is a common method of measuring stage in streams with unstable channels.

A system designed for storm sewers (*4*) is being investigated by the U.S. Geological Survey for use in open channels, which, if accepted, will replace the manometer. This system uses a pressure transducer instead of a manometer to measure the back pressure from the stream.

Crest-stage gages (CSGs) are used to determine the peak stage or to confirm the peak stage obtained by the recording system (*2, 3*). This is especially recommended with bubble gages. The CSG usually consists of a 2-inch pipe with a stick inside fixed on a pin of known stage. Ground cork located in the bottom of the CSG floats as water enters the gage through holes in the bottom of the CSG. The cork adheres to the stick at the highest stage and remains there as the water stage recedes. The CSG is an economical way of obtaining a peak stage and has been used extensively on small streams throughout the United States. The use of a CSG by itself will provide data only on the highest stage reached by the stream. Frequently the hydrograph is desired, and for this a continuously recording system is needed.

The Geological Survey uses an ultrasonic ranger at some sites that have large sediment loads. The ultrasonic ranger, shown in figure 4, is a device that sends an ultrasonic signal from a transmitter mounted on a bridge or other suitable structure to the water surface. The returning, or "bounced" signal, is monitored, and the time the signal travels is a measure of the distance of the water surface below the sensor. There are a variety of similar commercial devices on the market. The ultrasonic ranger is accurate to about 0.1 foot and can measure stages up to 35 feet below the sensor. When the sensor is at its maximum distance of 35 feet above the water surface, there must be a 13-foot diameter clear water-surface area under the sensor, as shown in figure 4.

The ultrasonic ranger has the advantage of not having any equipment in the water; therefore, it cannot be filled with or buried by sediment. The device can be mounted on a bridge, it can be suspended from a cable, or it can be placed on a cantilever over the stream. On all but the cantilevered device, the sensor can be readily moved across the channel so that it is directly over the water surface if the flow moves laterally.

Another device similar to the ultrasonic ranger is being tested by the Geological Survey. The device uses millimeter waves, and its accuracy is about 0.01 foot—more accurate than the ranger. The cost is higher than the cost of the ranger unit. Insufficient testing has been done with this unit to determine all its characteristics, but it appears to be promising.

It is desirable to confirm the peak stage, where possible,

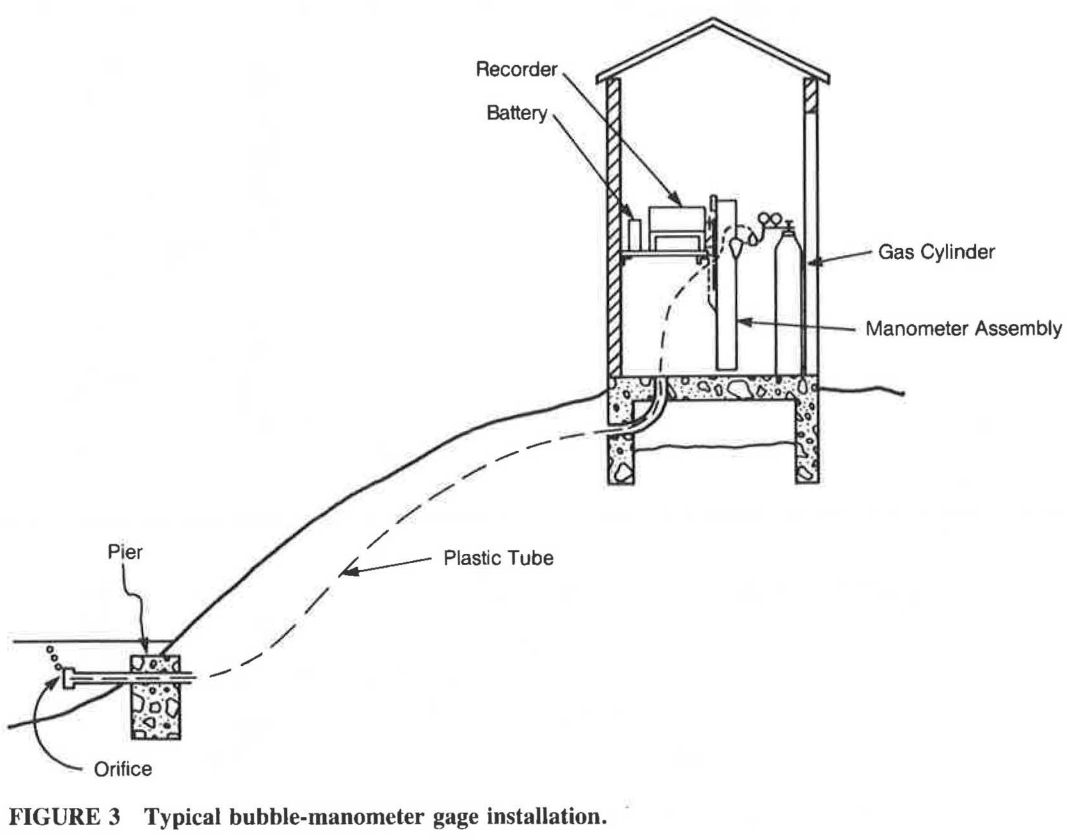

FIGURE 3 Typical bubble-manometer gage installation.

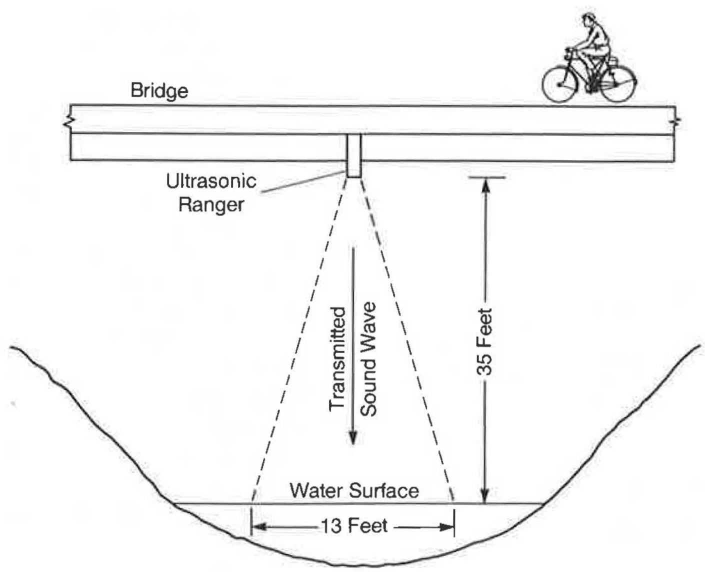

FIGURE 4 Ultrasonic ranger.

by checking for outside high-water marks, such as drift lines, and determining the elevation of any observed marks. This provides confirmation that the equipment was operating properly during the high-flow event.

Sometimes an observer is hired to read the gage or to provide information on high flows so that a stream gager can go to the gage to make a measurement. Such observers can act as a deterrent to vandals and can also notify the person responsible for maintaining the gage if there is a malfunction of the equipment or if other problems with the gage are detected.

Data-collection platforms (DCPs) can be useful in providing a central office with near real-time data on the stage of a stream at a remote site. Many Geological Survey offices query the stage at all stations with DCPs in their area each morning to determine the stage and the operating condition of the gage. This prompts the field person to visit the gage when the stage data from the DCP indicates high stages at the stream site or that the stream is rising. The DCP data also alerts the field person that a visit to the station may be needed for maintenance or repair.

One problem occurs with gaging some streams that have unstable channels when the flow moves laterally in its channel. This is commonly a problem at low flows, but it can also be a problem at high flows on some streams. There are a variety of ways to handle this problem. One is to dig a trench from the flow to the gage intake or orifice; another is to locate several stilling wells across the stream channel to try to have one where the flow will be. Bubbler orifices can be readily moved to where the flow is. If the gage is an ultrasonic device mounted on a bridge, it can be moved easily to a point over the water.

STREAMBED MEASUREMENTS

The stage of the water surface of some streams may not be a good indicator of the flow in that stream because the streambed may be filling and scouring. Streams have been observed where the flow changed drastically while the water-surface stage remained fairly constant as a result of scour or degradation of the streambed. In these situations, unless the streambed elevation, or depth, can be monitored, there is little hope for relating flow to a stage observation alone.

Two very similar devices to monitor streambed elevation have been developed. Both devices consist of pipes mounted vertically in the stream channel with the bottom of the pipe buried in the channel. Both devices mount sensors at selected intervals throughout the length of the pipe. One device uses conductivity sensors, and the other uses a device that puts out a heat pulse that is then monitored to determine its die-off characteristics. Both devices assume that the sensor will detect different responses if the sensor is in water rather than if it is buried in sediment. These devices are still being tested.

A limitation to these devices is that they monitor the bed elevation only at the location of the monitor. They can also interfere with the flow, resulting in local scour around the sensor. On some streams, this may be representative of the entire streambed, but on others it may not be. One way to determine what is going on with the streambed in a more general way is to place several monitors across the channel. This can be expensive, however, and also can require considerable data storage.

Various sounding devices using acoustic or ultrasonic signals have been used to monitor streambeds. This is usually done only when a person is at the site to operate the equipment. The author is not aware of any field sites where such equipment is operated on a continuous basis without an operator present.

It has been suggested that an ultrasonic velocity meter, as shown in figure 5, used to send a signal across a stream to measure water velocity also has a signal component that bounces off the streambed. This signal component could be monitored to provide a measure of the streambed elevation. This methodology has not yet received much study. The method would monitor the bed elevation primarily near the center of the stream.

Another factor that affects the stage for a given discharge is the bed regime of the channel (*2*). It has been suggested

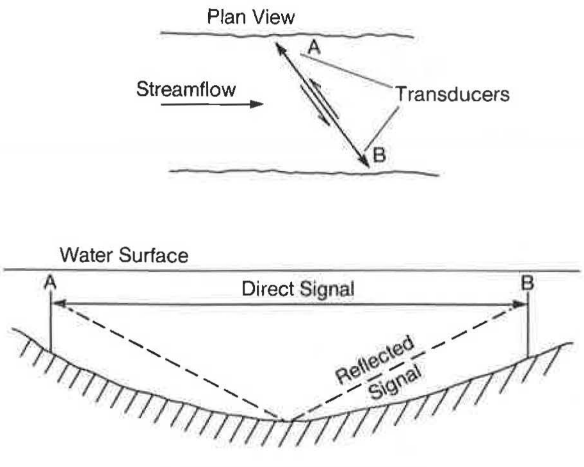

FIGURE 5 Ultrasonic velocity meter.

that time-lapse photography of the stream surface could provide clues to what is happening to the stream-bed. Sand dunes on the bed create boils in the water surface, whereas a plain bed will have a smooth water surface. Other bed features also provide their imprint on the water surface.

DISCHARGE MEASUREMENTS

Unstable channels present difficulties in making discharge measurements. The streambed can change significantly while a measurement is being made, adding uncertainty to the measurement. Conventionally, velocity observations are made at 0.2 and 0.8 depths and averaged or, for shallow depths, at the 0.6 depth to represent the average velocity in a vertical. This is based on the assumption that there is a normal vertical-velocity distribution, as shown in figure 6. The presence of sand dunes and holes in the streambed add uncertainty to assumptions concerning the vertical-velocity distribution. Instead of making observations at the 0.6 depth or the 0.2 and 0.8 depths, it may be necessary to make additional velocity observations in the vertical section.

Debris can also be a problem when making a measurement but, in general, it is no more a problem in a stream with an unstable channel than it is in other streams. In the case of either a fairly rapidly changing streambed or of heavy debris, the field person may use short-cut methods when making a discharge measurement. This involves reducing the times for velocity observations and using fewer sections for depth and velocity observation. In such cases, the measurement should be repeated using different verticals to measure depth and velocity.

Unstable channels can present safety hazards for personnel making discharge measurements. If the measurement is made by wading, a scour hole can present a hazard. If a boat measurement is being made, turbulence from various bed forms can create hazardous navigation conditions, which, in turn, usually makes for poorer quality measurements.

Some surface-velocity measuring devices have been developed. The devices measure the velocity of the water surface, and a coefficient is applied to obtain a mean velocity in the vertical. One such device, an optical current meter, uses a system of rotating mirrors to fix on floating objects (*2*). The speed of rotation of the mirrors is adjusted so that the objects appear to stand still. The speed of rotation is proportional to the velocity of the floating object. Consideration is being given in the Geological Survey to the use of radar guns, similar to those used by law enforcement agencies to check the speed of cars, for use on floating objects in the water.

In some streams, dilution methods of flow measurement can be used effectively (*5*). In this method, a tracer is injected into the stream at a constant rate, and after it has mixed with the flow samples are taken at a downstream location. The flow is determined by measuring the dilution of the tracer. This method does not depend on knowing the channel shape or having smooth uniform flow. In fact, turbulence is desirable since it promotes mixing of the tracer in the total flow. This method has been used for making discharge measurements, but not much has been done with this method to continuously measure the flow of a stream. Fine sediment affects some tracers and reduces their effectiveness for discharge measurements. In addition, the amount of tracer required for continuous measurement may be cost prohibitive for all but very small streams. It is likely the method of continuous measurement using tracers may be usable only on a few streams.

Indirect measurements of peak flow are often made when it is not possible to obtain a direct observation of flow at or near the flow peak (*2, 6*). The lack of a direct measurement may be the result of not being able to get to the site; or it may be that the flow is too turbulent or the physical facilities to make a measurement are not available, as when a bridge used for measuring has washed out.

Making indirect measurements depends on knowing about the channel geometry, the hydraulic roughness of the channel and, in some cases, the water slope of the stream at the peak. Conventionally, high-water marks are found along a reach of stream on both banks and surveyed in for location and elevation. These marks are used to determine stage, water-surface slope, and for contracted-opening type measurements,

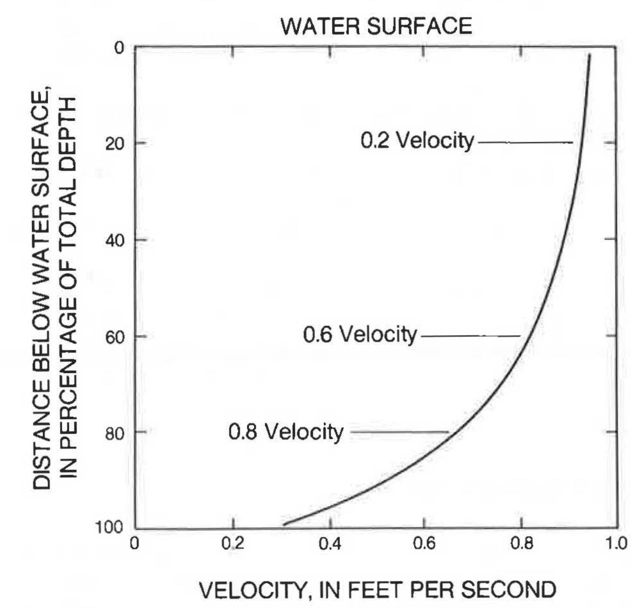

FIGURE 6 Typical vertical-velocity curve.

the hydraulic head drop through the contraction. A series of CSGs can be used to help obtain high-water marks in a stream reach, established in advance for indirect measurements.

Unstable channels add uncertainty to indirect measurements. There is commonly some question as to where the streambed was at the time of the peak. The *n*-value (roughness coefficient) for movable-bed streams is not accurately known. The bed forms at the time of the peak and the effect that they might have on the channel roughness coefficient are usually not known. Nevertheless, the peak-flow determination may have to depend on an indirect measurement. In that situation, the user needs to be aware of the factors affecting the accuracy of the peak-flow determination.

STAGE-DISCHARGE RELATION

The stage-discharge relation may shift considerably in an unstable channel (*1, 2, 7*). Usually the channel shifts most at high flows, but the impact of the shifted channel is most noticeable on the low-flow rating curve, as shown in figure 1. The effect, however, can be substantial on the high-flow rating as well. In order to define these shifts, many discharge measurements may be needed. If a number of peaks occur between discharge measurements, the uncertainty of the flows between peaks may be greatly increased.

At times, the shifting may be so great that there is no definable stage-discharge relation. When this occurs, the gage may need to be moved or the flow determined entirely from discrete discharge measurements, which requires many measurements.

STREAM CHANNEL AND CONTROL STABILIZATION

In most streams with unstable channels, it is not practical to install a laboratory weir or flume. One is usually content to simply stabilize the channel or control section affecting the stage-discharge relationship.

Usually, concrete controls are not used in unstable channels because there is a tendency for the underlying material to settle or move; this causes the control to crack and fail. There also is a tendency for the water to undercut the downstream side of the structure and thus cause the structure to fail unless deep cutoff walls are constructed. Gabions can be used successfully in some channels; however, they are not successful in channels where material underlying the streambed is unstable for tens of feet under the surface. Gabions should be placed so they do not create much of a fall of water over the gabion or so that undercutting will not occur on the downstream side of the gabions; otherwise the gabions could sink into the resulting scour hole. Gabions, as well as any other material or structure, should be keyed well back into the stream banks to prevent a washout around the ends of the gabions. The lack of adequate keying into the stream bank is a common cause of failure of stabilization efforts.

Sheet piling can be used on some streams to help stabilize the channel. Again, the pilings should be keyed deeply into the banks to prevent washouts around the ends of the piling. Such sheet piling weirs should be placed low in the channel with a minimum of fall in the water surface, and they should be designed for submergence at high flows.

A dual-weir concept has been used at some installations using sheet piling or other material as the structural component. The idea is to use an upstream and a downstream weir with the downstream weir lower than the upstream weir. The flow over the upstream weir creates increased turbulence that keeps the space between the weirs free from deposited sediment. The water surface is measured between the weirs with the downstream weir being the rated weir. A problem with this approach is that the dual weirs behave differently for different flows. For example, at low flows the spacing between weirs may be great enough for deposition of sediment to take place and for the flow between weirs to be fairly calm. At higher flows, the water between the weirs may be turbulent; this keeps the sediment from depositing between the weirs but makes it difficult to measure the stage between the weirs. The spacing should be planned for the flows of greatest interest. The dual weir concept is acceptable for small to intermediate size streams, especially streams with stable material near the surface of the bed.

When streams need to be gaged where the flow meanders back and forth across the channel, channel stabilization measures are to be avoided. Aside from cost, there is increased probability of the stream washing around the sides of the stabilization structure.

Concrete weirs and flumes can be placed in some unstable channels (*8*). For all but very small streams, the cost is likely to be high and there will be increased backwater (increased heads) upstream from the structure.

EPHEMERAL STREAMS

Ephemeral streams can create special problems for obtaining flow information. During the dry season, channels near bridges are sometimes worked in with tractors doing bridge maintenance work, thereby changing the stage-discharge relation established at an earlier time. Insects and other small creatures tend to get into dry wells, intakes, and orifices, plugging them so they do not work when flows occur. If there is a distinct seasonal flow, as there is on the west coast of the United States, the station may be visited and made ready to operate near the end of the dry season but before flow is expected.

Ephemeral streams are often flashy, that is, flow quickly peaks and recedes. This makes them difficult to get to during a flow event and may make it difficult to obtain a discharge measurement.

DATA UNCERTAINTIES

Peak-flow data for unstable streams in arid areas may contain large uncertainties. The reasons for these uncertainties include the following:

1. The stage-discharge relation is unstable.
2. A continuous stage record cannot be obtained because—
   a. the flow moves away from the gage,
   b. the stilling well fills with silt, or
   c. the intakes or orifice are covered with silt.
3. The flow at the gage is in multiple channels.

4. Scour occurs around bridge piers and debris accumulates on bridge piers where discharge measurements are made.
5. The vertical-velocity distribution is not normal when a discharge measurement is made because of sand dunes and scour holes in the measurement section.
6. The hydraulic roughness and the vertical location of the streambed are not well known, hindering indirect measurements.
7. Multiple peaks occur between discharge measurements, and the amount of change in the rating is not defined during these periods.

The following practices can be used to reduce these uncertainties:

1. Select a site with a relatively stable channel.
2. Stabilize the channel in the vicinity of the gage where possible.
3. Use a noncontact stage sensor such as an ultrasonic ranger, where large sediment loads are likely to fill a stilling well with silt or cover intakes and orifices.
4. Avoid making discharge measurements in the vicinity of bridge piers when possible.
5. Determine if the vertical-velocity distribution is normal, and obtain additional measurements of velocity in the vertical if it is not.
6. Continue to look for equipment that can continuously measure the bed elevation or depth of flow.
7. Inspect the gage and make discharge measurements more frequently.

In general, the determination of flow in unstable channels, which are commonly found in arid areas, is more expensive and the data contain more uncertainties than for stable channels.

SUMMARY

Streams in arid areas commonly have unstable channels, frequently are ephemeral, and, when flowing, transport large sediment loads.

A streamflow gage should be placed in the area of greatest channel stability. This, possibly more than anything else, will help to produce acceptable streamflow records.

Stilling wells and bubble gage-manometer systems are most commonly used for obtaining measurements of stream stage. The heavy sediment loads transported by many arid area streams can fill a stilling well with sediment or bury intakes and orifices. For this reason, gages that do not have to be in the water, such as the ultrasonic ranger, are sometimes used to measure stream stage.

Commonly, stream discharge is determined from a stage-discharge relation. In streams with severely unstable channels, such a relation does not exist. In such situations, it may be more appropriate to relate discharge to the depth of flow. A measure of the depth of flow can be obtained by measuring the stage of the water surface and the stage of the streambed. Some work has been done to measure the stage of the streambed with a scour meter. It also has been proposed that the signal reflected from the streambed by an ultrasonic velocity meter also may be used to monitor the stage of the streambed.

Streambed stabilization is sometimes possible on small- and medium-size streams. Usually, because of the large sediment load transported through the stream system, it is not practical to totally stabilize the channel in the area to be gaged. A dual weir is useful in providing a fairly stable stage-discharge relation in some streams.

Discharge measurements may be more complex on sand-channel streams with dunes and scour holes than on other streams. This is partly because the vertical-velocity curve is not normal, and additional velocity observations may be needed.

Many arid-area streams are ephemeral and are flashy, making it difficult to obtain discharge measurements. Dry stream channels near bridges commonly are disturbed by maintenance crews; this alters stage-discharge relations.

Many factors, such as shifting stream channels and buried sensors, increase the uncertainty in the measurement of stage and discharge in streams in arid areas. However, techniques and instruments such as the ultrasonic ranger and the scour meter have been developed to reduce these uncertainties.

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Basic Characteristics for Regression Analysis in Arid Areas

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Multiple-regression techniques are commonly used to transfer flood characteristics from gaged to ungaged watersheds. In the arid or semiarid areas of the United States, the standard errors of these regression relations often are quite large. One way to reduce the standard error is to identify basin characteristics that are significant for predicting *T*-year flood discharges such as 50- or 100-year floods. Recent investigations have identified new characteristics that appear to be promising. Examples of these are main channel sinuosity, hydraulic radius, bank-full channel conveyance, basin shape, time-to-peak of the flood hydrograph, effective drainage area, and percent of the basin in a given hydrologic soil group. The appropriateness of their use and the application of these basin characteristics are discussed. In addition, a few new basin characteristics are suggested that have not yet been investigated. Examples include channel infiltration losses, ratio of main channel width to floodplain width, stream-network magnitude, channel storage indices, and drainage density.

Planning and designing highway bridges and culverts requires knowledge of the magnitude and frequency of flooding so that economical and safe designs can be obtained. At most locations where drainage structures are contemplated, no gaging station records are available. Therefore, there is a need to develop techniques for transferring flood characteristics (such as the 100-year flood-peak discharge) from gaging stations to ungaged sites. A technique commonly used to estimate flood characteristics is multiple-regression analysis. In this analysis, flood characteristics at ungaged sites are estimated by regression equations that use basin and climatic characteristics as predictor variables. For arid areas in the southwestern United States, the standard errors of estimate of the regression equations often exceed 60 percent and can be as high as 100 percent. A possible way of improving this transfer technique is to identify new basin characteristics that are significant in predicting flood characteristics.

The purposes of this paper are to (1) describe basin characteristics presently being used, (2) document the need for new basin characteristics, (3) describe some new basin characteristics that have been recently evaluated and show promise, and (4) suggest some new basin characteristics that might be useful for estimating flood characteristics. Several of the new basin characteristics have been used in the more humid areas in the east, but they may have applicability in the arid west as well. Basin characteristics, as described in this paper, can be (1) channel characteristics, such as active channel width, hydraulic radius, and sinuosity ratio; (2) topological characteristics, such as stream-network magnitude, link-length distribution parameters, and drainage density; (3) hydrograph characteristics, such as time-to-peak and basin lag time; and (4) the more conventional basin characteristics, such as drainage area, channel slope, percent of forestation, and soil characteristics.

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PRESENTLY USED BASIN CHARACTERISTICS

The U.S. Geological Survey has made extensive use of regression equations based on basin and climatic characteristics to estimate flood characteristics. Since 1973, every Geological Survey District Office has published at least one report on estimating flood-peak discharges at ungaged sites. A list of these reports by state is provided in (*1*). The regression equations in these reports are based on basin and climatic characteristics that can be readily determined from topographic maps and climatic reports of the National Weather Service or state agencies. Table 1 summarizes the frequency of use of these basin and climatic characteristics in the statewide regional flood-frequency analyses. The data in table 1 are based on statewide reports that were published before December 31, 1986. Drainage area was used in regression equations in reports prepared for all 50 states and Puerto Rico; it is usually the most significant variable in accounting for variability in flood estimates. Main channel slope and some index of precipitation (such as annual precipitation or the 2-year 24-hour precipitation) were variables used in regression equations for 24 and 33 states, respectively. Basin storage and forest cover are expressed as percentages of the total drainage area covered with lakes, swamps and ponds (for basin storage), or forests. Mean basin elevation is a popular variable in the western states where flood characteristics for watersheds at higher elevations are much different from those at lower elevations. Main channel length is occasionally used in conjunction with drainage area to describe basin shape. Normally, however, channel length is highly correlated with drainage area and does not explain any additional variation in the flood characteristics. This partially explains why channel length is used in only five states. Minimum January temperature is sometimes used to help explain the difference in flood peaks caused predominantly by snowmelt and those caused by rainfall. Soil characteristics are not used very frequently, possibly because the infiltration values readily accessible for these regional flood studies are not well defined. Soil characteristics may be a variable that warrants further evaluation in future studies in arid areas. There were a few other variables, such as basin shape (basin length squared divided by drainage area) and

TABLE 1 SUMMARY OF BASIN AND CLIMATIC CHARACTERISTICS USED IN U.S. GEOLOGICAL SURVEY REGRESSION EQUATIONS

Independent variable	Number of States (including Puerto Rico)
Drainage area	51
Channel slope	24
Annual precipitation	22
Basin storage (%)	18
Mean basin elevation	12
Precipitation intensity	11
Forest cover (%)	9
Channel length	5
Minimum January temperature	5
Soils characteristics	4

seasonal snowfall, that were used in only one state each and are not shown in table 1.

Many of the basin and climatic characteristics shown in table 1 have been used frequently because of their availability, ease of computation, and no requirement for a site visit. Future research and evaluation should be oriented towards identifying new predictor variables that might provide a more accurate estimating relation even though additional effort may be needed to determine their values.

NEED FOR NEW BASIN CHARACTERISTICS

One useful aspect of using regression equations to estimate flood characteristics is that the accuracy of these equations is easily determined. Accuracy as used in this paper is defined as the standard error of estimate of the regression analysis; it is the error to expect two-thirds of the time. This standard error is a measure of the accuracy of the regression equations when compared to the gaging station data used to develop the equations. It may or may not reflect the true predictive accuracy of the regression equations.

Table 2 summarizes the distribution of standard errors of estimate for the 50- and 100-year flood discharges for the 50 states and Puerto Rico. The table is based on the maximum values for the different hydrologic regions within each state, and it shows that all 50 states and Puerto Rico have at least one hydrologic region where the standard error of estimate exceeds 30 percent. However, there are some hydrologic regions in Alabama, Georgia, Wisconsin, Pennsylvania, and Ohio where the standard error is less than 30 percent. The data in table 2 can be used to determine the number of states having standard errors in a certain range. For example, 6 states have maximum standard errors in the range of 30 to 40 percent, and 12 states have maximum standard errors in the range 50 to 60 percent. Keep in mind, however, that the standard errors often vary considerably among hydrologic regions within a given state, and the distribution of standard errors shown in table 2 is based on the maximum value for any hydrologic region in that state.

It is interesting to note the geographical distribution of the standard errors. States where the maximum standard error exceeds 60 percent are shown in figure 1. The states are predominantly in the West with the exception of Minnesota, Maryland, and Florida. Each of these three states has one or more low-lying hydrologic region where basin storage, channel storage, or non-contributing areas of the watershed are significant. The standard error tends to be higher in the western states due to the greater variability in annual flood-peak data (time-sampling errors) and the less dense gaging station network (space-sampling errors). The problem is compounded by the fact the watershed and climatic characteristics are more variable in the arid areas of the western states. In addition, the watershed and climatic characteristics frequently used (table 1) do not sufficiently account for the variation in flood characteristics among watersheds. These are some of the reasons why the states with standard errors exceeding 60 percent are mostly in the more arid West. It is possible that this high standard error could be reduced by using different basin characteristics that explain more of the variation in flood characteristics from site to site. It should be noted, however, that the standard errors for basins at higher elevations and the more humid areas in the west tend to be less than 60 percent, primarily because of the reduced variability in the annual flood-peak data. Therefore the reader should not infer from figure 1 that all hydrologic regions within the shaded states have standard errors exceeding 60 percent.

DESCRIPTION OF NEW BASIN CHARACTERISTICS

In this section, promising new basin characteristics recently used by various analysts and a few new basin characteristics that might be useful for estimating flood characteristics are discussed. The discussion of these basin characteristics will be grouped into the following categories: channel characteristics, elevation-oriented approach, topological characteristics, and hydrograph characteristics. These categories are defined below.

Channel Characteristics

In many arid or semiarid areas, *T*-year flood discharges decrease (attenuate) as basin size increases. This is due, in part, to

TABLE 2 DISTRIBUTION OF STANDARD ERRORS FOR 50- AND 100-YEAR FLOODS ESTIMATED FROM U.S. GEOLOGICAL SURVEY REGRESSION EQUATIONS

Standard error (in percent)	Number of States (including Puerto Rico)
>30*	51
>40	45
>50	31
>60	19
>80	10
>100	2

*Certain hydrologic regions in Alabama, Georgia, Wisconsin, Pennsylvania, and Ohio had standard errors less than 30 percent

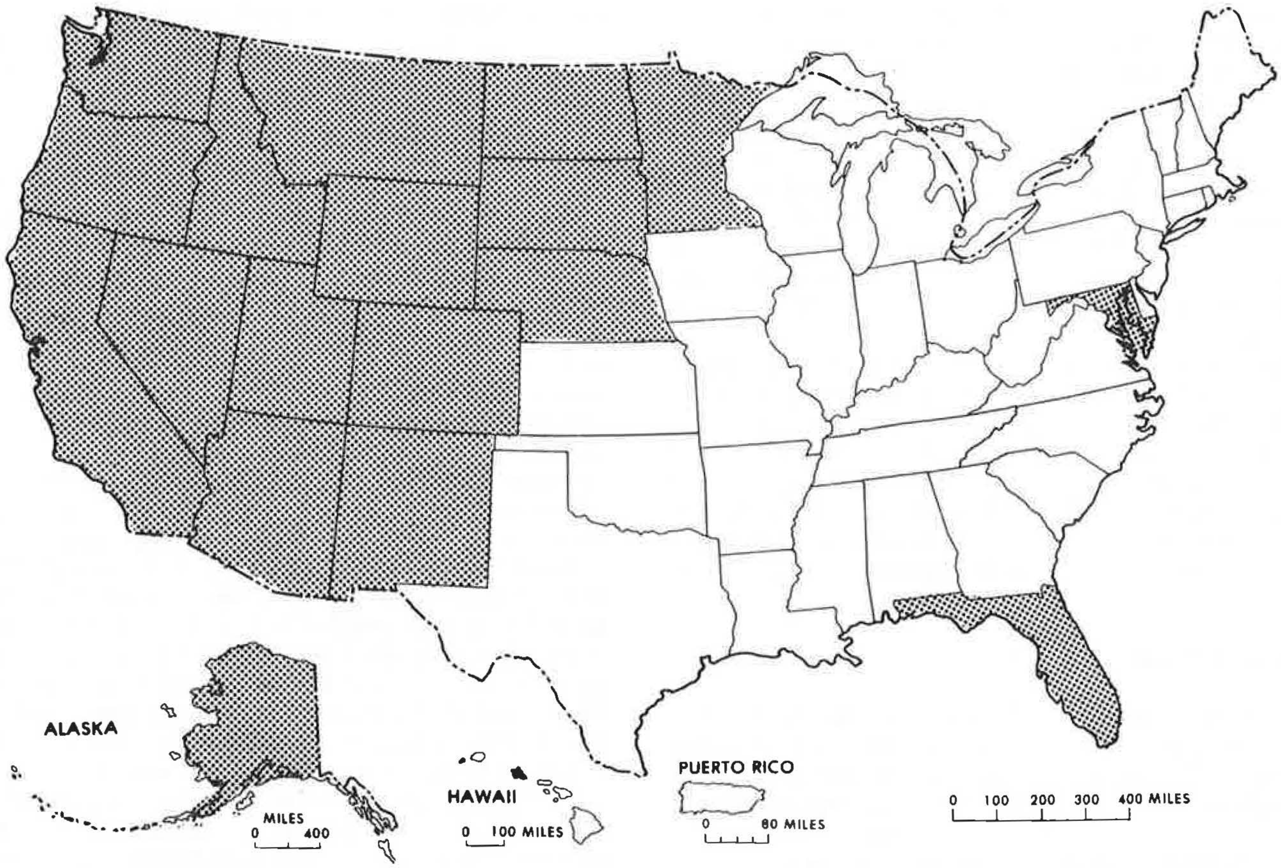

FIGURE 1 Identification of states where standard errors of estimate for the 50- and 100-year floods exceed 60 percent in at least one hydrologic region.

channel storage and channel losses. Since drainage area is usually the most important independent variable, this explains why conventional basin characteristics often do not adequately explain the variation in flood characteristics. As an alternative, many analysts (*2, 3, 4*) have used channel-geometry characteristics, primarily active channel width, to estimate flood characteristics. Active channel width is briefly defined as the width of main channel measured between the permanent vegetation on each bank. The application and accuracy of the channel-geometry technique and a more complete definition of the required channel characteristics are given in (*2, 3, 4*). The method does require a field visit to the ungaged site to measure channel width. Regression equations based on active channel width have been developed by the Geological Survey in eight western states and Ohio and are listed in (*1*). These nine states also have regression equations based on basin and climatic characteristics.

Channel storage often causes flood discharges to decrease in a downstream direction. In a recent study in southern Arizona (*5*), attenuation adjustment factors were computed by dividing at-site *T*-year flood estimates by the regional regression estimates. These adjustment factors were then multiplied by the regional regression estimates when applied to ungaged sites with significant channel storage. Another approach is to define channel characteristics that reflect these storage characteristics and use them in the regression analysis. One such study (*6*) in 1964 examined channel width, channel width/depth ratio, channel cross-sectional area, and channel cross-sectional area times length of the main channel as indices of channel storage. None of these characteristics was found to be significant in estimating flood discharges, given that drainage area and channel length were already included in the equation. However, these indices should be reevaluated in future studies. Another measure that is closely related and that might be useful as an index of channel storage, is the main channel width relative to floodplain width. This value could be measured from topographic maps at several locations throughout the basin and averaged to obtain a single index.

In arid areas in the west, floods usually occur in dry stream channels. The volume and peak discharge of the flood is reduced by infiltration into the stream bed, stream banks, and possibly the floodplain. The U.S. Soil Conservation Service (*7*) has defined ways of estimating these reductions in flood volumes and peak discharges using characteristics of the channel reach such as effective hydraulic conductivity, a decay factor, average width and length of the reach. Procedures are given for estimating the reduction in flow with and without using observed inflow-outflow data. A table is provided for relating streambed material characteristics to the above-mentioned channel characteristics so that the method is applicable to ungaged sites (i.e., no observed inflow-outflow data).

In a recent study in Kentucky (*8*), main channel sinuosity was found to be significant for estimating flood discharges. The main channel sinuosity was defined as the ratio of the main channel length divided by the basin length (straight-line distance from outlet to basin divide) and is another measure of channel storage. In the Kentucky study, the main channel sinuosity was inversely related to flood discharges.

In a recent study in Arkansas (*9*), hydraulic radius or mean channel depth was found to be a significant predictor variable. It was assumed that the hydraulic radius used in conjunction with channel slope was a better index of velocity of the flood wave than channel slope alone. A method for determining hydraulic radius without visiting the ungaged site was given in the report (*9*). The standard errors of the regression were reduced an average of 9 percent when hydraulic radius was used in conjunction with conventional watershed characteristics.

Bank-full channel conveyance was found to be a significant factor for estimating urban flood discharges in Houston, Texas (*10*). In this study, bank-full channel conveyance was defined as the conveyance at a controlling section downstream from the gage when the water-surface elevation was equal to that of the lower stream bank. The conveyance was computed using Manning's equation. Although this variable was used for urban streams, it may also be applicable to rural streams.

Elevation-Oriented Approach

In arid or semiarid areas, flood characteristics tend to vary as a function of elevation of the watershed. In the eastern foothills of the Rocky Mountains in Colorado, the drainage area below 8,000 feet was shown to be a better predictor of rainfall floods than the entire drainage area (*11*). This is because extreme rainfall events generally occur at elevations below 8,000 feet. Above 8,000 feet, precipitation generally occurs as snowfall, and unit runoff is not as great. In the Colorado study (*11*), the standard error of the 100-year rainfall-related flood was reduced significantly by using drainage area only below 8,000 feet rather than the entire drainage area of the watershed.

In Nevada, unit-flood-discharge values (discharge per square mile) were determined for each of several elevation zones (*12*). The total *T*-year flood discharge for a given watershed was determined by the sum of the products of the area of each elevation zone and its respective unit-flood-discharge values. The unit values were adjusted by trial-and-error procedures until a good agreement was reached with station data. This approach may be applicable to other areas of the southwest and deserves further evaluation.

Topological Characteristics

Topological characteristics are defined as those characteristics describing the geometry or geomorphology of the channel network and the basin. A recent report using data for small streams in Wyoming has indicated that stream-network magnitude (number of first order streams) and link-length distribution parameters are useful for estimating the shape of unit hydrographs (*13*). The link-length distribution parameters are actually the scale and shape parameters of the two-parameter gamma distribution. This distribution is fitted to the sample of internal link lengths, defined as the distance between junctions (confluences) of streams of order of 2 and 3, 3 and 4, etc. The distribution of internal link lengths essentially describes the density of the stream channel network. If the stream-network magnitude and link-length distribution parameters are important in estimating unit hydrographs, then they may have application in estimating flood discharges of a given return period.

In a study in Kentucky, basin shape, defined as the basin length squared divided by drainage area, was shown to be a significant variable for estimating *T*-year flood discharges (*8*). In this case, basin length was measured as a straight-line distance from the gage to the point on the basin divide used to determine main channel length. The shape of the basin is indicative of how fast the flood waters run off the basin. A long narrow basin tends to store water and attenuate flood peaks. On the other hand, runoff tends to be more rapid from circular basins since the travel time of the flood wave for the major tributaries tends to be more nearly equal. Occasionally, drainage area and main-channel length are used independently in the regression equations and are, in effect, a measure of basin shape (*9, 14*). In these instances, main-channel length has a negative exponent in the regression equation.

Another variable closely related to the link-length distribution parameters is drainage density. Drainage density is defined as the total length of all streams per unit of area. It is a measure of the development of the drainage system and should be an indicator of how fast the surface runoff occurs. To the author's knowledge, no one has demonstrated that this variable is significant in predicting flood discharges. This variable is affected by the scale of the maps used, the contour interval, and the extent to which streamlines are mapped. As an alternative to measuring the blue streamlines on the topographic map, perhaps the stream length should be computed as the total distance upstream until the swales disappear on the map. Now that topographic coverage at a scale of 1:24,000 is available nearly nationwide, this variable may prove to be significant in estimating *T*-year flood discharges.

Hydrograph Characteristics

Hydrograph characteristics are computed from basin characteristics combined with streamflow data. In actuality, they are streamflow characteristics, but in this paper they will be referred to as basin characteristics since they are indicative of basin response. Time-to-peak, time of concentration, and basin lag time are examples of hydrograph characteristics. These characteristics integrate the effects of soils, basin slope and shape, channel storage, land cover, and stream network configuration. In a recent study in Wisconsin, the time-to-peak was shown to decrease the standard error of prediction significantly over using the more conventional variables of drainage area and mean annual precipitation (*15*). Of course the reduction in standard error would have to be significant to make it worthwhile to collect the streamflow data needed to compute the time-to-peak. In Wisconsin, the standard error of prediction was reduced from about 35–38 percent to 23–32 percent, depending on the recurrence interval of the flood discharge. The analyst would have to make the decision as to whether the reduction in standard error warranted the collection of limited streamflow data.

Certain analysts have shown that basin lag time (time from centroid of rainfall excess to centroid of runoff) is significant in estimating flood discharges for urban areas (*16, 17*). This characteristic may also be useful in estimating flood discharges for rural areas. As noted above, the reduction in standard error would have to be significant to warrant collecting the streamflow and rainfall data needed.

The author's personal experience in using hydrograph characteristics to estimate flood discharges in Illinois was that the reduction in standard error was not sufficient to warrant collecting the needed data. In the Illinois study (unpublished report), the linear storage routing coefficient was also used as a predictor variable in addition to time of concentration. The linear storage coefficient is the slope of the recession hydrograph, which is indicative of how fast the flood waters drain from storage once inflow (precipitation excess) to the watershed has ceased. The time of concentration and the linear storage coefficient were statistically significant but they did not substantially reduce the standard error of estimate determined by using conventional basin characteristics. It is the author's opinion that the use of these hydrograph characteristics needs further evaluation relative to the estimation of *T*-year flood discharges.

Conventional Basin Characteristics

There are also possible improvements in the conventional basin characteristics given in table 1 that are worth considering. In a recent study in Colorado, effective drainage area proved to be a more significant predictor variable than total drainage area (*18*). Effective drainage area was computed by subtracting drainage areas upstream from all erosion-control or flood-retention structures in the basin from the total drainage area. Since the U.S. Soil Conservation Service has constructed these erosion-control and/or flood-retention structures in most arid basins, the use of effective drainage area should be applicable in areas outside of Colorado. These control structures are generally small, uncontrolled reservoirs designed to retain about a 25-year flood. This concept of effective drainage area is not intended for use in basins with large flood-control reservoirs with controlled outflow. Effective drainage area is most applicable for small drainage areas, since this is where the majority of the erosion-control and flood-retention structures have been constructed. It may be necessary to determine effective drainage area from field reconnaissance rather than maps if the reproducibility of computing this variable proves to be low.

Soil characteristics have occasionally proven to be significant in estimating flood discharges (see table 1). One possible reason that soils characteristics have not been used more is that the infiltration values are not well defined. The infiltration values available in the Geological Survey Streamflow and Basin Characteristics File were provided to the Geological Survey by the U.S. Soil Conservation Service during a nationwide surface-water network analysis study in 1969–70. It is the author's opinion that these infiltration values should be reviewed and possibly recomputed based on the latest soils maps.

Studies in Maryland and Delaware have indicated that the percentage of the basin in each of two (A and D) of the four hydrologic soil groups (A, B, C, D) of the U.S. Soil Conservation Service is a significant predictor variable for *T*-year flood discharges (*19, 20*). The use of these soil characteristics greatly reduced the standard error of estimate in the Coastal Plain region of Maryland and Delaware, and these characteristics should be investigated in arid areas of the western United States. The percent of the basin having a certain soil type proved to be more significant than an average infiltration value for the entire basin.

SUMMARY AND CONCLUSION

The standard errors of regression equations for estimating *T*-year flood discharges in arid areas are often quite high. A possible solution is to identify new basin characteristics that are significant in explaining the variation in flood discharges from site to site. Several types of basin characteristics that have recently been evaluated were discussed. The following basin characteristics may be useful for estimating flood discharges in the arid West.

Channel characteristics—

1. Channel-geometry characteristics, primarily active channel width, have been used by the Geological Survey in eight western states and Ohio to provide reliable estimates of *T*-year flood discharges.
2. A study in the southwestern United States in 1964 investigated four indices of channel storage and did not find any of them significant for estimating *T*-year flood discharges (*6*). However, these characteristics should be reevaluated in future studies.
3. A possible new indicator of channel storage is main channel width relative to floodplain width.
4. Channel infiltration losses as defined by the U.S. Soil Conservation Service may be useful in estimating *T*-year flood discharges (*7*).
5. Main channel sinuosity was found to be significant in estimating *T*-year floods in Kentucky and may have applicability in the arid West (*8*).
6. Hydraulic radius was found to be significant in Arkansas, where the inclusion of this characteristic reduced the standard error of estimate by an average of 9 percent for several recurrence interval floods (*9*).
7. Bank-full channel conveyance as used in an urban study in Houston should be investigated in studies of rural flood characteristics in arid areas (*10*).

Elevation-oriented approach—

1. The drainage area of basins below 8,000 feet was shown to be more significant than total drainage area when estimating rainfall related flood discharges in Colorado (*11*).
2. In Nevada, unit flood runoff per elevation zone was summed to obtain an estimate of the total flood discharge (*12*). This approach may be applicable to other areas in the Southwest.

Topological characteristics—

1. Stream-network magnitude and link-length distribution parameters were shown to be useful in Wyoming for estimating unit hydrographs (*13*) and may have applicability for estimating *T*-year flood discharges.
2. Basin shape was found to be significant in Kentucky in

estimating flood discharges (*8*) and may have applicability in the arid West.

3. Drainage density could be a useful characteristic for estimating *T*-year floods.

Hydrograph characteristics—

1. The time-to-peak hydrograph characteristic reduced standard errors of prediction 6 to 12 percent in Wisconsin over use of conventional basin characteristics (*15*).

2. Basin lag time was shown to be significant in urban areas (*16, 17*) and may have application to rural streams in arid areas if streamflow and rainfall data can be obtained in a cost-effective manner.

Conventional basin characteristics—

1. Effective drainage area was shown to be significant in estimating *T*-year flood discharges for small streams in eastern Colorado (*18*) and may have applications in other areas of the Southwest.

2. In studies in Maryland and Delaware, the percent of the basin in a given hydrologic soil group proved to be a more significant variable than the average infiltration value for the entire basin (*19, 20*). This approach needs to be evaluated in the more arid areas in the West.

The investigation and use of the basin characteristics discussed in this report may lead to development of improved regression models for streams in arid or semiarid areas of the United States. Understanding the physical processes that cause floods is necessary to develop improved predictive models.

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Bridges Are Expensive—Bridge Failures Are More Expensive

EMMETT M. LAURSEN

The failure of the New York State Thruway Bridge over Schoharie Creek demonstrated once again that bridge failures are more expensive than just the bridge itself, and that it would be prudent to assess the vulnerability to floods of all existing bridges over alluvial rivers. All such bridges are vulnerable to some degree unless they have been built by design or by engineering judgment for the maximum flood to be expected and the worst geometry and flow conditions that may come into being during the life of the bridge—and with proper evaluation of all factors. The remedial measures needed to make an existing vulnerable bridge virtually invulnerable will probably cost more than comparable measures would have cost at the time it was built; but, nevertheless, the cost can probably be justified when life and limb and a year or two of traffic delay are considered. That nothing has ever happened to a bridge that is 25 or 50 years old may be just a matter of luck, and it is not a sufficient reason to not assess the vulnerability of the bridge. Luck can run out, and the failure of an old bridge can still be very costly.

Bridges are expensive. Therefore, one might expect a prudent man (or organization) to build bridges so there will be only a minimal chance of failure during their anticipated useful life. More precisely, the added costs of building a more secure bridge should be balanced against the product of the total cost of a failure times the probability of that failure. The fact that none of the three factors is known with complete confidence is not a reason to do nothing; engineers should not shirk the responsibility of making an explicit, judicious assessment of the risk to be taken. The world is a dangerous place and man's knowledge is limited: therefore, some risk must be taken. However, bridge failures are more expensive than the bridge itself, as evidenced by the total cost of the failure of the New York Thruway Bridge over Schoharie Creek. The small risks of unusual events should be assessed and the cost of building to stand, even if an unusual event occurs, should be thought of as an insurance policy.

THE PROBLEM

Bridges can fail in any of several ways:

1. The live loads imposed on a bridge—legally or illegally—can be much greater than anticipated in the design.
2. The materials of which the bridge is made can deteriorate so their strength is reduced.
3. An earthquake greater than considered in the design can occur.
4. The earth on which the bridge is built can sink or slide.
5. If the bridge crosses a stream and is not founded on bedrock, the stream bed around the piers and abutments can be scoured out, destroying the ability of the foundations to support the bridge.

The first of these reasons for failure can only be reduced by the education of legislators and the enforcement of laws. The second, and possibly the fourth, hopefully should be noticeable during the bridge inspections that have been conducted regularly since the tragic failure of the Silver Bridge over the Ohio River. The third and the fourth can be the subject of re-analysis when and if additional information on earthquakes or landslides become available. Few bridges fail for these first four reasons, and those that do are usually old and are likely to be obviously vulnerable.

It is the fifth reason for bridge failures—floods—that is at issue here. More bridges fail in floods than in any other way, and their vulnerability is not apparent in routine inspections. The vulnerability of a bridge to floods can only be assessed by (1) determining the magnitudes of the floods that could occur, (2) imagining the changes that might occur in the channel reach sometime in the future, (3) delineating the flow pattern through the bridge, and (4) estimating the scour and lateral forces that could result in the failure of the bridge. In the case of a new bridge, there is little doubt that the bridge should be built so it will not fail, although the best way to build the bridge may not be clear. In the case of an old bridge, a greater risk might have to be accepted because of the limited further useful life of the bridge and because of the difficulty and cost of making the old bridge secure.

THE DESIGN FLOOD

One thing that is clear is that the 100-year flood rule for design is simply not good enough. The 25 (or 50) year nominal life of a new bridge means that there is a 25 percent (or 50 percent) chance the bridge will fail in a flood of greater magnitude than the 100-year flood. Making the bridge invulnerable to the maximum flood that can be expected will not increase the cost by 25 percent or 50 percent. The lesser useful life of the old bridge may result in the 100-year flood being the flood to be resisted—if the value of the bridge is the only loss considered. But if life and limb and a year of traffic delay are included in the losses considered, remedial work to enable the bridge to resist a rarer flood can probably be justified.

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It should be noted here that the first and most unrealistic guess would be the infinite flood as posited by almost all mathematical expressions used in hydrologic flood-frequency analysis. A bridge cannot be designed to withstand an infinite flood. However, there can always be a question whether or not some large, but finite, flood could be exceeded. The best evidence of realistic maximum floods that can be expected, but probably will not be exceeded, is being obtained by the geomorphologists who have been studying paleofloods.

INSURING THE INVESTMENT IN BRIDGES

The total investment in bridges in any state is a truly large sum. Some of these are not over waterways, and some of those over waterways are not by any stretch of the imagination vulnerable to floods. But bridges over alluvial streams which are founded on the alluvium are vulnerable to some unknown degree. It would be prudent to assess their vulnerability, and it would be wise to decrease that vulnerability if possible and justifiable.

Can one be absolutely certain that a bridge has been made invulnerable to floods? Not if mere best guesses are made as to the maximum expected flood flow, the future character of the river channel, the hydraulics of the flow, the amount of debris, and the predicted scour. Only if unrealistic guesses are made of all these factors could one feel *absolutely* certain that the bridge has been made invulnerable. That certainty, however, would have been achieved at a cost that may be unreasonable. It should be possible to compare the cost associated with the best guesses and the cost associated with the unrealistic guesses. And then consider whether there is something better to do with the extra money—say, some other safety measures in the transportation system that would save lives.

NEW BRIDGES

For new bridges the best solution will usually be to deepen the foundations to accommodate the scour predicted in the worst case because it is only the extra length of pile or caisson, or deeper spread footing, that results in added costs—the construction plant is in place. However, this is not to imply that alternative (and innovative) designs need not be considered; there will be opportunities to save by optimizing the foundation (and perhaps bridge) design.

OLD BRIDGES

For old bridges, a number of solutions are possible, including:

1. Riprap at the level of the bottom of the deepest (future) scour hole for which the present foundation is adequate.
2. Spur dikes to move the scour hole away from vulnerable abutments.
3. Channel improvements to improve the hydraulics of the bridge opening.
4. Additions to the present foundation (such as a sheet pile ring).
5. A new foundation.
6. A low dam or drop structure downstream of the bridge to raise the stream bed under the bridge.
7. Adding spans to the bridge.

These solutions are listed roughly in order of cost, and it is readily apparent that the last solutions can cost as much as the bridge is worth. Therefore, it is also obvious that the first solutions would be preferred if spur dikes or other remedial work would stay during a big flood and would function properly. However, enough is known now to go ahead with assessments and remedial work—there will always be a need to know more.

OTHER FLOOD PROBLEMS OF HIGHWAYS

It should be noted that the discussion here has referred only to the bridge, not to approach embankments or roads parallel to the river, or culverts. These are separate problems—similar, but different. They should also be investigated, but the urgency for remedial measures is less for one simple reason. Bridges, when they fail in a flood, are likely to fail quite suddenly with little or no advance notice because the scour hole cannot be seen through the muddy flood waters. Approach embankments, parallel roads, and culverts should usually give evidence of impending failure—if someone will only look. Therefore, since everything cannot be done at once, the priority should be given to the bridge problem, and when that is in hand, attention can also be given to the other parts of the transportation system which may be vulnerable to floods.

A PROGRAM TO ASSESS THE VULNERABILITY OF BRIDGES TO FLOODS

A program to assess the vulnerability of bridges to floods needs to address several issues more or less simultaneously—some of which may have been studied by someone in the past:

1. Development of one or more flood magnitude-frequency-watershed area relations with special emphasis on the maximum expected flood magnitudes.
2. Accumulation of evidence of changes in channels in regard to plan form and aggradation and degradation.
3. A tentative, quick examination of the bridges over alluvial rivers to separate them into (a) those hopefully not vulnerable to floods, (b) those probably vulnerable to floods, and (c) those that may or may not vulnerable to floods.
4. A careful examination, first, of the bridges probably vulnerable to floods and the recommendation of measures that should be taken to make them less vulnerable (or invulnerable) to floods; eventually, all bridges should be checked.
5. Adding to the routine bridge inspection program, observations and standard photographs of the channel characteristics upstream, through, and downstream of the bridge.

Once the bridges have been assessed, it should not be necessary to repeat the assessments unless (a) new evidence of flood magnitudes are obtained, (b) new evidence of possible channel changes are obtained, (c) improved methods of predicting the hydraulics of flow or the expected scour become available, or (d) the routine bridge inspections provide evidence of channel changes not already considered.

The assessment program would be expensive; remedial measures would cost more. However, the losses which would not occur should result in an overall savings to the transportation system in the long run. In addition, the traveling public and the responsible officials would have peace of mind.

Prediction Methods for Local Scour at Intermediate Bridge Piers

Howard D. Copp, Jeffrey P. Johnson, and Jack L. McIntosh

The ability to establish foundation elevations for intermediate bridge piers that will provide a reasonable degree of assurance that the pier will not be undermined by the flowing stream and to rate existing intermediate bridge pier foundations relative to their risk of being undermined has become a matter of national concern. This paper will document the results of a study that presents and recommends formulae that can be used to predict the anticipated depth of local scour in both uniform-particle, cohesionless streambeds, and graded, armored streambeds. The study incorporated both a literature search and a field verification of the results of the literature search. The Laursen and Toch formula will be recommended to predict anticipated local scour depths at intermediate bridge piers in uniform-particle, cohesionless streambeds, and the University of Aukland formula will be recommended for consideration for graded, armored streambeds. This document will show that these scour prediction formulae, in conjunction with other engineering data, can be a valuable tool to aid the engineer in economically and safely establishing an intermediate bridge pier foundation elevation, or rating the safety of an existing foundation.

For over 100 years, engineers have noted that the intrusion of intermediate bridge piers into a flowing stream causes eddy currents, which in turn may scour and undermine the bridge foundation. Researchers have proposed over 35 different formulae for local scour prediction since 1949. Almost all of this research has focused on streambed materials that are uniform in size and cohesionless; however, many streams in Washington and other states have beds of graded material with some degree of armoring.

An extensive literature search that uncovered over 50 publications dealing with the prediction of local scour at intermediate bridge piers and an investigation of 28 bridge sites in Washington indicate that the use of prediction formulae based on uniform-particle, cohesionless streambeds for estimating scour in graded, armored streams may produce excessive scour depths. Under certain circumstances, though, armored beds may exhibit scour greater than that found in uniform-particle, cohesionless beds.

Scour was predicted and compared for six bridge sites in the state of Washington using four methods of scour prediction formula for uniform-particle, cohesionless streambeds, and one method for graded, armored streams. The method for graded, armored streams indicated anticipated scour depths of about one quarter that of the uniform-particle methods. All these structures had experienced meaningful floods. In-depth investigations of these foundations indicated no significant scour problems.

This paper presents and suggests consideration of a procedure developed by Raudkivi and Ettema at the University of Aukland, with a safety factor suggested by the authors, for estimating local scour at intermediate bridge piers in graded, armored streams. The formula developed by Laursen and Toch is recommended for streams with uniform-particle, cohesionless beds. Caution, and the application of engineering judgment, is suggested in the evaluation of the results of either method. More research is required to further confirm or deny the validity of these formulae.

BACKGROUND

The Washington State Department of Transportation (WSDOT) has traditionally protected both its bridge approach abutments and intermediate bridge piers against erosion with riprap. Riprap has been placed on the end abutments as shown in Figure 1, and over the intermediate piers as shown in Figure 2.

At least 20 percent and not more than 90 percent of stones weighing 300 pounds to 1 ton, at least 80 percent of stones weighing 50 pounds to 1 ton, and at least 10 percent and not more than 20 percent of stones weighing 50 pounds or less are the riprap for both bridge abutments and intermediate piers. Periodic observations by inspection crews indicate that this type of protection has performed adequately on structures experiencing flows as large as the 50-year mean recurrence interval. This apparent adequate performance applies to both end abutments and intermediate bridge piers.

WSDOT has long recognized that the thalweg of the stream can meander across the floodplain; thus, the tops of all foundations in the floodplain were set a minimum of 2 feet below the thalweg. This, with the use of riprap for all erosion, appears to have provided adequate countermeasures for general, constriction, and local scour for structures that have experienced flows as large as the 50-year mean recurrence interval. A quantitative estimate of each type of scour has not been required. This observation, however, is not intended to suggest or recommend continuation of this practice.

In the early 1970s, environmental requirements precluded WSDOT's practical use of riprap at intermediate bridge piers. To excavate for the riprap, cofferdams were needed to prevent the accommodation of silt and the resulting adverse effect on fish. Riprap could still be used to protect the bridge abut-

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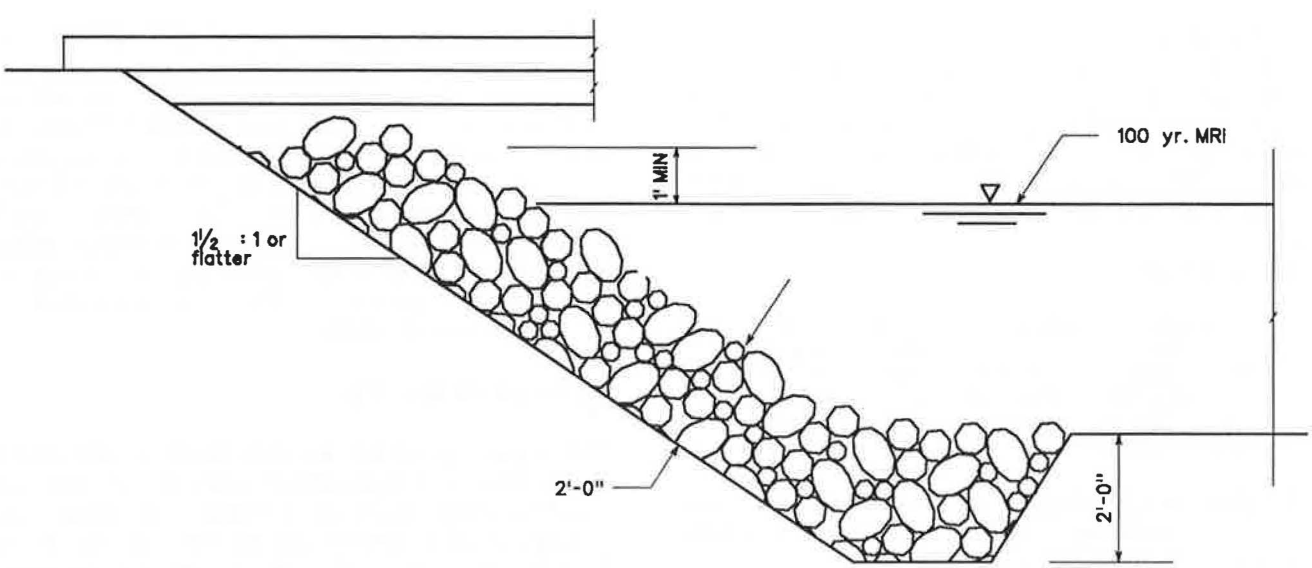

FIGURE 1 Riprap at end abutments.

ments, as the needed excavation normally could be done in the dry. WSDOT then searched for another way to protect the intermediate bridge foundations from undermining.

WSDOT realized that the practical solution was to set the foundations of the intermediate bridge piers at an elevation that would not be undermined. General and constriction scour at most Washington sites were subjectively considered negligible. WSDOT determined that the key to adequately setting the foundation to prevent undermining centered on the knowledge of the meandering thalweg and the ability to predict scour. Foundation elevations could then be established to offer a reasonable assurance that the pier would not be undermined by scour.

Intermediate bridge pier foundations are very costly. As the foundation is lowered to mitigate against scour, the head on the bottom of the cofferdam seal is increased. This increased head requires a thicker seal, which increases the amount of excavation, sheet piling, and concrete required for the foundation. WSDOT recognized that while overly conservative methods of scour prediction should be avoided, loss of the structure because of an inadequate pier foundation is even more costly.

SCOUR PREDICTION RESEARCH

Highway agencies are concerned with the loss of any bridge attributed to undermining of a pier foundation by local scour

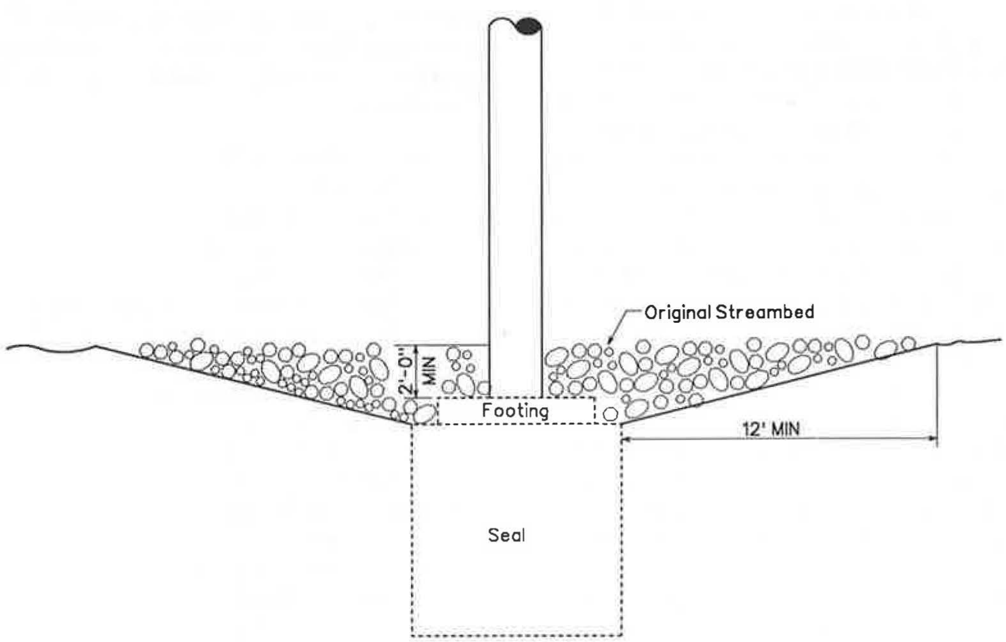

FIGURE 2 Riprap at intermediate bridge piers.

and will sponsor research leading to a more precise ability to predict the depth of anticipated local scour at intermediate bridge piers. For example, considerable bridge losses in Iowa in 1947 resulted in the intensive study by Laursen and Toch (*4*). This was the forerunner of many modern research projects that addressed the issue of predicting local scour depth an intermediate bridge piers in uniform-particle, cohesionless soil.

Hopkins et al. (*3*) stated:

> Over the past century many investigators have attempted to develop a simple scour prediction formula. . . . It appears that a set of variables were arbitrarily selected and data collected over a limited range to determine their relationship to scour depth. . . . This approach has left us with a large number of sometimes conflicting formulas to predict scour.

Hopkins's statement suggests a reason for the many scour prediction formulae uncovered in the literature and the diverse scour depths that they produce.

WSDOT engineers recognized that the majority of the scour research had produced formulae that predicted anticipated local scour at intermediate bridge piers for uniform-particle, cohesionless streambeds. WSDOT also realized that most of the Washington bridge sites consisted of graded, armored material. In most situations, graded, armored material resists erosion much better than uniform-particle, cohesionless soil. Recent work by Raudkivi and Ettema (*6*) indicates that under certain conditions this may not be true. Further research is needed to better describe and quantify this issue. A reliable equation to predict local scour based on graded, armored streams could produce significant cost savings while maintaining a reasonable degree of confidence.

SCOUR PREDICTION IN GRADED, ARMORED STREAMS

WSDOT, in cooperation with the Federal Highway Administration (FHWA), issued a request for a prospectus for a research program dealing with determining the estimated scour depth for intermediate bridge piers in graded, armored streams. Different prediction methodology is required for local scour at end abutments, local scour at intermediate piers, general scour, and constriction scour. Recognizing a need for the ability to predict all these types of scour, it was arbitrarily decided that to narrow the scope of research, only local scour at intermediate piers would be investigated. This decision resulted in a research project award to Washington State University (WSU) in 1986. The principal investigator was Howard D. Copp, who was assisted by Jeffrey Johnson. Their work culminated in the report *Riverbed Scour at Bridge Piers* (*2*).

The study had a single objective. WSDOT was using the Laursen Toch formula to estimate the depth of local scour at intermediate bridge piers under all streambed conditions. WSU was to determine, within the specified limitations, the most appropriate methods for predicting local scour depth at intermediate bridge piers for both uniform-particle, cohesionless and graded, armored streambeds.

The study team was to concentrate on keeping recommended methods practical. Methods of scour prediction that required extensive collection of data or observations of stream characteristics over a period of time were not to be considered. Absolute and rigorous research methods were to be subordinated to practicality. Known methods of scour prediction were to be uncovered by a literature search, and these methods were to be compared and evaluated. Comparisons of known methods of scour prediction with a field investigation were included in the scope of the research. Field investigations, made after a flood, cannot be relied on to show the maximum scour occurring during the flood peak. Combined with historical records of flood flows, they do give a general indication of the present condition of the foundation, i.e., whether it is safe or unsafe.

LITERATURE SEARCH

The literature search revealed 38 formulae developed to predict the anticipated depth of local scour at intermediate bridge piers. Some were based on laboratory experiments, others were developed through field investigations, and some involved both laboratory and field work. All but one pertained to uniform-particle, cohesionless streambeds.

Only the scour prediction formula based on research conducted by Raudkivi and Ettema (*5*) at the University of Aukland incorporates a parameter that recognizes a nonuniform or graded streambed material. It will be referred to as the UAK formula.

The UAK formula includes a geometric standard deviation of size distribution. All other parameters being equal, this one difference predicts an estimated scour depth significantly less than any of the other 37 equations that were developed for uniform-particle, cohesionless beds.

A complete listing of these expressions is contained in *Riverbed Scour at Bridge Piers*.

COMPARISON OF SCOUR PREDICTION METHODS

To compare the many different scour prediction formulae, they first must be rearranged so that the variables are identified and classified in a common manner. The many parameters that influence scour around intermediate bridge piers have been arranged by Breussers (*2*, p. 276) into the following four groups:

1. Stream fluid variables
   a. Density of fluid, p
   b. Viscosity of fluid, v
2. Stream flow variables
   a. Depth of flow, y_0
   b. Velocity of the flow approaching the pier, U
   c. Magnitude of stream discharge, Q
3. Streambed materials
   a. Grain size distribution
   b. Grain diameter
   c. Sediment density, P_s
   d. Cohesive properties
4. Pier size and shape
   a. Pier dimensions
   b. Pier shape in plan view
   c. Surface roughness
   d. Number and spacing of piers
   e. Orientation of piers to approach flow direction
   f. Pier protection (fenders, for example)

In attempting to make this comparison, it was found that because of the complexities and attendant costs of measuring, analyzing, and evaluating all of the above-mentioned variables, many investigators deliberately

1. assumed that the differences between the laboratory and field values for density, viscosity, and the acceleration due to gravity can be neglected;
2. restricted the study to steady, uniform flow fields unconstricted by bridge approach fills;
3. considered only alluvial, noncohesive, uniform particle-sized bed materials; or
4. considered only perfectly smooth, single piers that are perfectly aligned with the approach flow and do not have scour protection systems, such as riprap.

These assumptions and restrictions reduce a long list of variables that affect scour depth to the following eight:

1. Fluid density
2. Kinematic viscosity of fluid
3. Gravitational acceleration constant
4. Sediment grain size diameter
5. Bed sediment density
6. Approach flow depth
7. Mean approach flow velocity
8. Pier width

Many researchers have compared these different categorical arrangements and have determined that under certain conditions several equations would give comparable results. Several of the prediction equations give comparable results and reasonable estimates of scour; however, they are not necessarily valid. This was the conclusion of Raudkivi and Sutherland (*7*) after they had compared 17 prediction equations to actual scour depths measured at four New Zealand bridge sites. A field investigation, while it cannot absolutely verify the validity of a scour prediction equation, can certainly nullify it.

FIELD STUDY

Twenty-eight bridge sites on state highway routes in Washington State were investigated for evidence and magnitude of scour at intermediate bridge piers. The exposed streambed and bank materials at most of these locations were nonuniform in size (graded), and consisted of fines to large gravel, and, in some locations, small to medium boulders. Significant armoring was observed in most cases.

It was recognized that the field measurements would not indicate the maximum scour that had occurred but rather that they would show the general condition that exists. These measurements were not intended to be a verification of any given equation.

Table 1 lists the six sites, gives the date of construction, shows the magnitude of the flood of record, compares the different prediction methods, and gives a general indication of the condition of the foundation with the field measurements.

The field procedure at each of the six sites consisted of

1. documenting the channel geometry, including the identification of the channel pattern and measuring the bridge waterway cross-sectional dimensions;
2. evaluating the type and characteristics of the streambed bank and bed materials; and
3. measuring the apparent depth of local scour at various

TABLE 1 SUMMARY OF SCOUR DEPTHS AT SIX WASHINGTON STATE BRIDGE SITES

	Study Site					
Bridge Site Equation	5/216E Newaukum (1)	507/102 Skookumchuck (2)	507/128 Nisqually (3)	90/82S S. Fk. Snoq. (4)	12/706 Touchet (5)	12/725 Tucannon (6)
Year Built	1952	1971	1917	1975	1966	1967
Flood of Record	50 Year MRI	10 Year MRI	50 Year MRI	15 Year MRI	35 Year MRI	7 Year MRI
C.S.U.	19.6	5.5	24.9 [a]	17.3	11.7	12.7 [b]
Laursen-Toch 1	25.8	6.5	25.1	13.8	9.3	14.7
Shen II	15.7	6.4	34.0	27.0	16.5	15.7
Neill	17.2	4.5	31.4	14.0	5.7	20.0
UAK	5.2	1.4	8.0	4.3	2.1	5.1
Field Measurements	6.1	1.7	8.0	2.8	1.7	3.3

Note: Units in feet; 1 ft = 0.305 m

[a] Computed using foundation width, 15.7 ft

[b] Computed using pedestal width, 10.0 ft

locations around the piers, which is not intended to represent the maximum depth of scour but to give an indication of the overall condition of the foundation.

WSDOT "as-built" drawings were obtained for each site, and the design flow, from U.S. Geological Survey streamflow records, was listed for each of the six sites. The historical flood of record was determined and expressed in terms of the mean recurrence interval. All sites had experienced meaningful flood flows in their lifetime. Work by Laursen (*4*) and others suggests that a significant parameter influencing depth of scour is the depth of the approach flow. The historical floods at the locations in question were sufficient to produce an approach flow depth that would have the capability to generate meaningful scour. It is not possible to determine, by direct measurements, the amount of deposition, if any, that occurred during the recession of the flood. Further research should center on developing an indirect method based on various field measurements combined with known principles of sediment transport to estimate the maximum depth of scour that has occurred.

CONCLUSIONS AND RECOMMENDATIONS

The comparisons in Table 1 between the four different scour prediction formulae for uniform-particle, cohesionless stream beds at the six structures investigated gave comparable results. As stated elsewhere in this paper, that is no guarantee of their validity. No scour prediction formula has been completely validated by objective, measured means. All the bridges studied had experienced meaningful historical floods, and all were in locations where the beds were graded, armored material. None of the bridge foundations appeared to be in any immediate danger from undermining.

WSDOT has been using the Laursen and Toch equation to predict anticipated scour at all intermediate bridge piers since the early 1970s. This is one of the earliest scour prediction methods, and it has received historical acceptance. This paper recommends the continued use of the method to predict local scour depths in uniform-particle, cohesionless streambeds. The recommendation is not founded on the field investigations made for this study but on the simplicity of the method and its wide historical acceptance.

It has been the subjective opinion of many engineers that the scour prediction formulae for use in uniform-particle, cohesionless streambeds give overly conservative results when applied to graded, armored streams. The work done by Raudkivi and Ettema, under normal conditions, tends to support this subjective opinion. Although their work has not been rigorously substantiated, it is recommended that the UAK formula, with adequate consideration for streambed layering and with a factor of safety, be considered for predicting local scour in graded, armored streams. Again, it should be noted that this recommendation is not founded on any field investigation made for this study but rather the literature review that indicates Raudkivi and Ettema are alone in their studies of graded, armored streambeds.

USE OF THE LAURSEN AND TOCH EQUATION

The Laursen and Toch equation is

$$d_s/b = 1.5\ (y_0/b)^{0.3}$$

where d_s is the anticipated depth of local scour, b is the width of the pier, and y_0 is the depth of flow approaching the pier.

The right-hand side of the above equation is multiplied by a design factor, K_σ, that ranges from 1 to 7 for the angle of attack of the stream to the pier, and by a shape coefficient, K_s, that varies from 1.00 to 0.70, depending on the nose shape of the pier.

The design factor, K_σ, for angle of attack of the stream to the pier, can be found in Figure 3. Table 2 lists the shape coefficient, K_s, for various nose shapes and pier configurations. These same modifiers are applicable to the UAK equation and to all the other scour prediction formulae referenced in this report.

USE OF THE UAK EQUATION

The initial step in using the UAK formula for estimating the scour depth in graded, armored streambed material is to

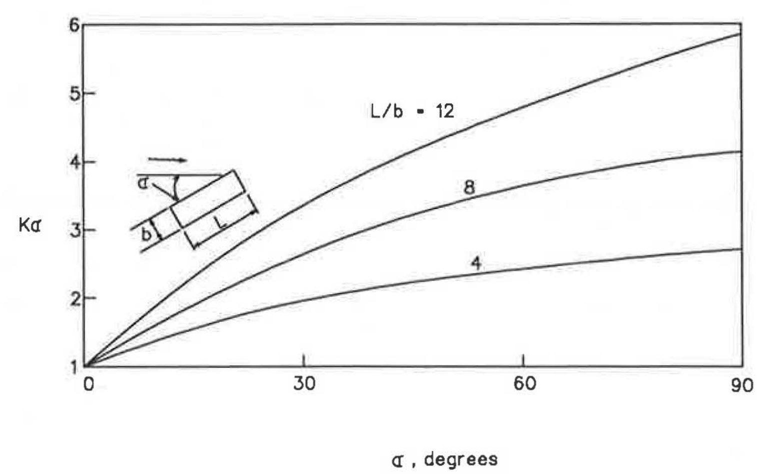

FIGURE 3 Design factor K_α for angle of attack of stream to pier.

TABLE 2 SHAPE COEFFICIENT K_s FOR VARIOUS NOSE FORMS

Nose Form	Length - Width	Shape	Ks
Rectangular		▭	1.00
Semicircular		(	0.90
Elliptic	2:1	<	0.80
	3:1	<	0.75
Lenticular	2:1	<	0.80
	3:1	<	0.70
Square		▭	1.0
Round		▭	0.9
Cylinder		o	0.9
Sharp		⬭	0.8
Group of Cylinders		o o	0.9

determine the characteristics of the riverbed material near the planned bridge. At a minimum, two samples of the streambed material should be obtained from each pier location. The sampling should range from 30 feet upstream to 30 feet downstream of each pier.

Samples should be obtained in a way that will not lose the fines. Special core drill apparatus are available, and they should penetrate the streambed about as deep as a "guess" of the estimated scour depth (about 6 to 10 feet). In some instances, a back hoe may be used.

When obtaining streambed samples, the engineer must ascertain whether the streambed is layered with interstices of fine sand or clay. If layering is present, actual scour may be deeper than predicted by the UAK formula because of stepwise failure in the layers. Predicted scour depths should be increased by 15 percent to 20 percent, depending on the engineer's judgment. This increase for layering is above the recommended safety factor that will be applied later.

The sample of the bed material from each location should represent material for the surface to the maximum depth. These samples should be carefully marked, taken to the laboratory, and analyzed with a sieve. A gradation curve can then be prepared for each sample obtained. A single "site" gradation curve is an average of all the curves of the site. A single average curve may be used if the variation of samples does not exceed 20 percent.

From this "site" curve, the size of sample that corresponds to the 16, 50, and 84 percentiles is determined. These are d_{16}, d_{50}, and d_{84}.

Next, b/d_{50} should be calculated, where b is the anticipated pier width in the direction of the streamflow. With this value of b/d_{50}, use Figure 4 with d_{50} greater than 0.7 mm and find the corresponding value of d_s/b. The mean value of scour depth can now be calculated as $d_{sm} = d_s/b \times b$. This is the

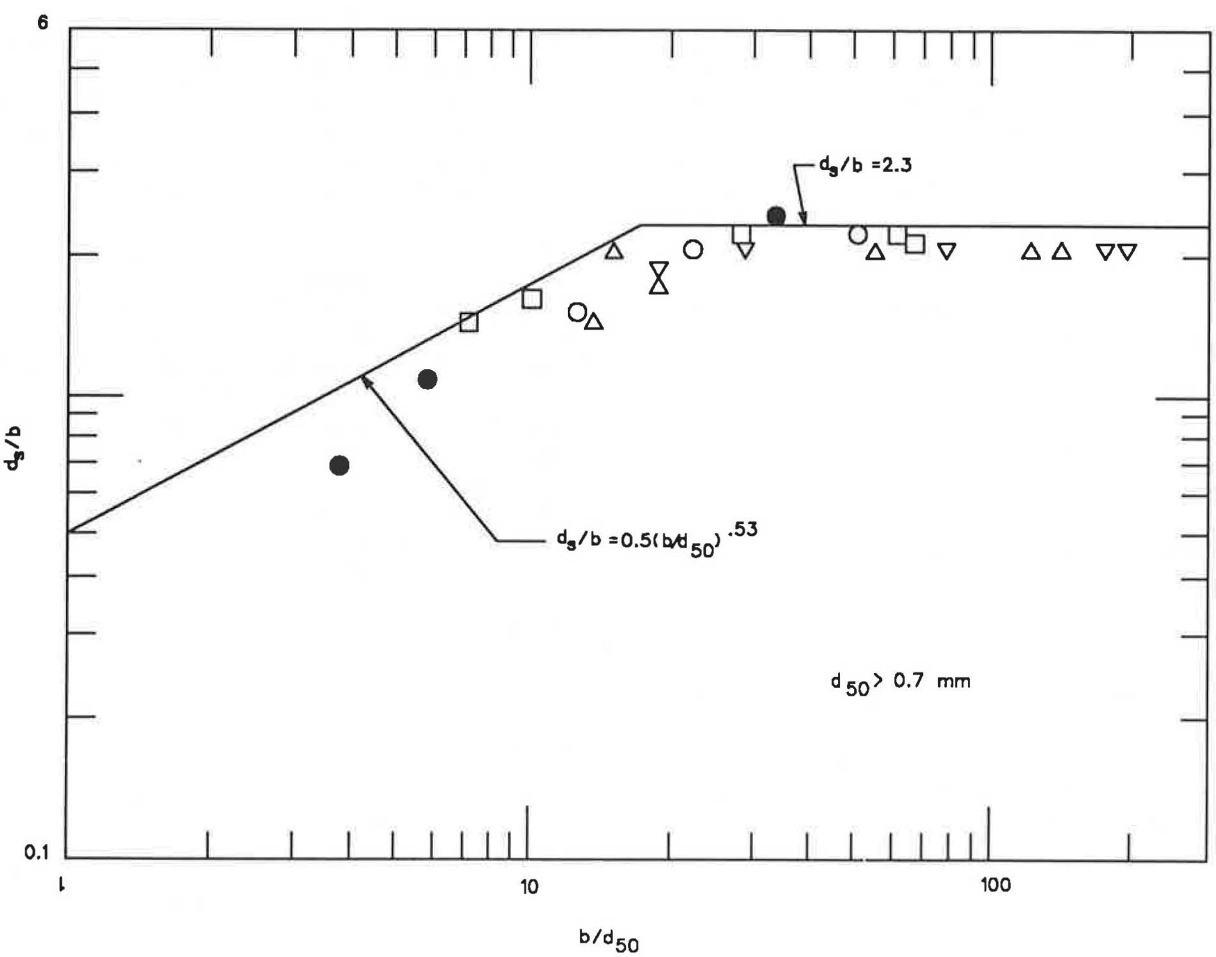

FIGURE 4 Scour-depth-to-pier-diameter ratio as a function of pier-diameter-to-sediment-size ratio.

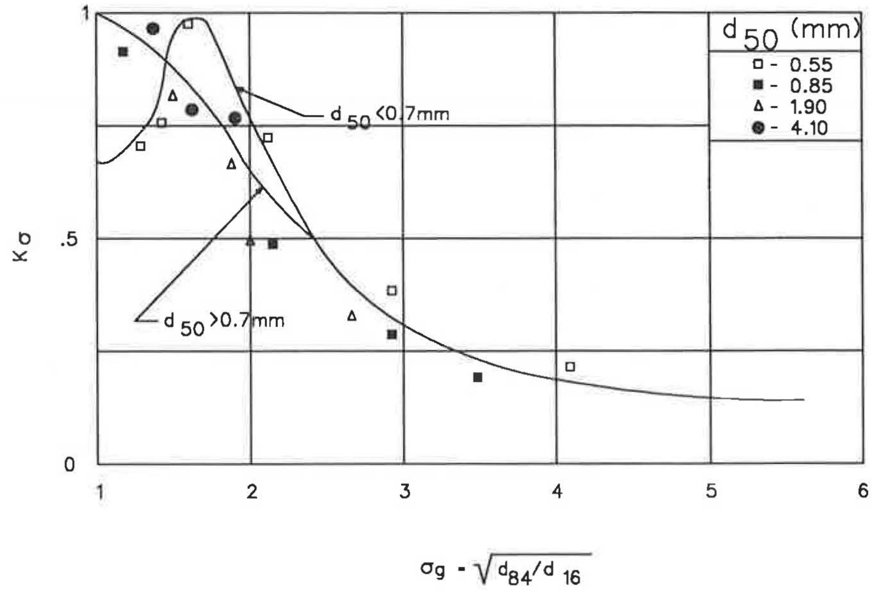

FIGURE 5 Particle size coefficient K_σ related to geometric deviation σg.

predicted scour depth if a rectangular-shaped pier is built and it will be oriented perfectly with the stream flow paths. Adjustments may be necessary.

K_σ is then determined. With the previously determined values of d_{16} and d_{84}, $\sigma g = (d_{84}/d_{16})^{1/2}$ can be computed. Use Figure 5 with this value of 5g and find K_σ. As in Figure 4, the $d \rangle 0.7$ mm curve is used.

Next, K_α is determined. L/b is calculated (L is the pier length) and used in Figure 3 with the angle α at which the pier will be oriented with the streamflow. These two values will permit the determination of K_α. K_s is then determined from Table 2.

Next, a factor of safety is established, K_{fs}. K_{fs} equal to $1/K_\sigma$ is selected whenever σg is less than 2, and 1.5 when K_σ is greater than 2. The final step is to compute the estimated scour depth as

$$d(\text{est}) = d_{sm}\, K_\sigma\, K_\alpha\, K_s\, K_{fs}$$

USE OF ESTIMATED SCOUR DEPTH

The use of either formula recommended in this report to predict anticipated local scour is only one of many tools that can determine the risk factors associated with the potential undermining of an intermediate bridge pier foundation. In addition to predicting local scour, the engineer must predict and quantify the effect of general and constriction scour and the meandering thalweg. It must also be kept in mind that the validity of all scour prediction formulae has not been conclusively demonstrated.

The degree that other information, such as underwater investigation of nearby bridge foundations on the same stream, soils investigation relating to the nature of the streambed, and the knowledge that can be obtained applying accepted principles of sediment transport to the stream, should be combined and evaluated with the results of the scour prediction formula is a matter of subjective engineering judgement.

Ignoring the potential for scour or relying solely on some form of artificial armoring as protection against undermining is a situation that can no longer be accepted. The engineer's goal is the knowledge that the existing or proposed bridge pier foundation is reasonably safe from undermining by the flow of the stream with the attendant loss of the structure. Scour prediction formulae, properly applied, are a way to help attain this goal.

An initial prediction of anticipated scour depth is made using the formula appropriate for the type of streambed material and the width of the pier, b, that protrudes into the flowing stream. If this predicted scour depth lies above the top of footing or pedestal, the foundation is safe. If this predicted scour depth lies below the top of the footing or pedestal, the predicted scour depth should be recalculated using the width of the seal for b. In a spread footing, if this new predicted scour depth is above the bottom of the seal, the foundation is safe. In a pile-supported footing, the predicted scour depth using the width of the seal for b can be below the bottom of the seal and still result in a safe foundation, provided that sufficient embedment of the piling exists below this predicted depth to fully develop the horizontal and vertical loads that are transmitted to the foundation.

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Inflow Seepage Influence on Pier Scour

STEVEN R. ABT, JERRY R. RICHARDSON, AND RODNEY J. WITTLERS

A flume study was conducted to investigate the influence of inflow seepage on localized pier scour through an alluvial channel bed. Eight tests were performed in which seepage inflow ranged from zero to three times the liquefaction velocity of the bed material. Channel Froude numbers ranged from approximately 0.40 to 0.80. The results indicated that for channel Froude numbers less than 0.70 and seepage velocity less than critical, the scour hole degraded by 20 percent. For channel Froude numbers less than 0.70 and seepage velocity greater than critical, the scour hole aggraded while the scour hole width increased as much as 2.25 times wider than scour hole widths observed for the no-inflow conditions. When the channel Froude number exceeded 0.70, scour hole depths were similar to the no-inflow conditions while the bed elevations adjacent to the scour hole significantly degraded.

Localized scour at bridge piers, abutments, and channel banks results in millions of dollars in damage to river-crossing structures. In general, scour is the erosive action of flowing water in streams and rivers that excavates and transports material from channel beds and banks (*10*). Localized erosion often causes a change in the channel bed elevation and/or lateral channel migration into the banks at or near the crossing structure. Improper accounting of many of the site specific parameters affecting local scour may result in a catastrophic failure of the structure.

Several types of local scour can be identified at most river crossings, including constriction scour, abutment scour, aggradation, and degradation. Numerous parameters exist at each river crossing that affects site stability. These parameters may include soil characteristics, hydrologic and hydraulic characteristics, seepage/groundwater conditions, and methods of construction.

A flume study was conducted that measured local scour resulting from flow in a straight alluvial channel around a single circular pier with and without inflow seepage. The objective was to investigate the influence of inflow seepage on localized pier scour and areas adjacent to the pier. Inflow seepage conditions exist when the water table elevation is high enough to contribute flow into the stream through the channel bed and/or banks. It was not the purpose of the study to generate another pier scour prediction equation, but rather to indicate how inflow seepage from the channel bed may affect pier scour prediction. The results are presented in this paper.

BACKGROUND

The attempts to predict the extent of pier scour and enhance bridge design procedures have been studied since the 1940s. One of the most comprehensive literature reviews was conducted by Jones (*10*) in which he cited 12 bridge pier scour prediction equations. Jones categorized the pier scour efforts as (a) pier scour formulas based on foreign research to include Ahmad (*1*), Bruesers (*2*), Chitale (*3*), and Inglis (*8*); (b) pier scour formulas patterned after University of Iowa research to include Laursen (*6–8*) and Jain (*9*); and (c) pier scour formulas patterned after Colorado State University Research to include Shen (*10*) and Richardson, et al. (*11*). All the equations presented were to predict the maximum depth of scour in the area adjacent to the pier. The primary independent variables cited included approach flow depth, projected pier width, approach velocity and Froude number.

Raudkivi performed a comprehensive analysis of flow around a circular pier, concentrating on flow patterns, velocity distributions, variation of scour depths around the pier, scour as a function of sediment gradation, and scour as a function of sediment size (*12*). He concluded that the pier width, type and gradation of sediment, flow depth, sediment size-pier width ratio, and pier alignment control the depth and pattern of scour.

The study of how inflow seepage affects an alluvial channel system has not yielded results as numerically sound as scour prediction. In 1966, Simons and Richardson stated that seepage force could change the bed form and, therefore, the resistance to flow (*13, 14*). However, this was not experimentally or analytically documented in their study. In a qualitative sense, Simons and Richardson concluded that seepage of water into the bed (outflow seepage) would tend to increase the effective weight of the bed particles and, therefore, increase the stability of the bed. Conversely for a gaining stream (groundwater flow into the channel or inflow seepage), the effective weight of the bed particles decreases and thereby decreases the bed stability. The inflow seepage could result in an increase in the sediment transport and change in the predicted bed form. This conclusion was cited in publications by Simons and Richardson (*13*) and Simons and Senturk (*14*).

Martin addressed the influence of inflow and outflow seepage on incipient motion of uniform bed materials (*15, 16*). Martin concluded that inflow seepage does not aid incipient motion of the sediment particles.

Harrison tested the effects of groundwater seepage (inflow, outflow, and zero seepage) forces on sediment transport for both lower and upper regime flows (*17, 18*). Harrison's study indicated that the upward (inflow) seepage had a limited effect on the stream sediment transport rate, even when the bed was quick. He concluded that the decrease in effective grain density brought about by inflow seepage might be offset by a decrease in surface drag on the individual grains and an increase in form drag. Harrison also noted that when bed forms were present the angle of repose of the downstream face of the bed forms increased by 10° for outflow seepage and decreased by 9° for inflow seepage.

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Harrison (*18*) and Martin (*15*) did not observe a significant increase in sediment transport or scour when inflow seepage occurred. Harrison did note that the bed form could be altered by inflow seepage. However, Harrison did not show that inflow seepage could change the bed form (i.e., from ripples to dunes, dunes to plane, etc.) which was hypothesized by Simons and Richardson (*13*).

The interaction between the channel flow and seepage flow interface and the subsequent response were studied by Watters and Rao (*19*). In their investigation, geometrically packed spheres were analyzed for lift and drag forces under inflow and outflow seepage conditions. Quantitatively, Watters and Rao concluded that inflow seepage tended to reduce the drag on bed particles whether they were on top of or in the soil matrix. The lift on bed particles under inflow seepage increased for particles in the soil matrix, but the lift on particles resting on the bed was reduced.

Watters and Rao concluded that

1. Inflow seepage increased the sublayer thickness.
2. The hydraulic roughness and consequently the drag on the plane bed decreased.
3. Inflow seepage increased the lift on particles within the bed and decreased lift for particles resting on the bed.
4. Turbulent fluctuations were more intense with inflow seepage than for outflow seepage or no seepage.
5. Inflow seepage increased the momentum transfer between fluid particles.

Nezu (*20*) performed a rigorous mathematical analysis quantifying the existence of a matched or exaggerated boundary layer along the bed. Nezu's analysis incorporated turbulence, induced stresses, velocity profiles, and backwater effects on flow over a permeable bed with seepage.

Nezu showed that seepage flow near the porous surface becomes turbulent as a result of the pressure fluctuations of the main flow. When this occurs, the seepage flow cannot remain laminar, and Darcy's law for flow in a permeable medium cannot be applied. This results in an additional shear stress induced in the main flow by the turbulent seepage flow. Nezu identified this additional stress as induced stress.

Richardson et al. conducted a series of tests in which a straight channel with alluvial material was subject to inflow seepage ranging from zero to where the material liquified (*21*). They determined that the bed forms could potentially be altered by inflow seepage. For example, a change from plane bed to ripples to dunes could result from introducing inflow seepage.

Richardson et al. concluded that inflow seepage through a porous bed directly affects many aspects of alluvial channel flow. Inflow seepage influence on the alluvial system is summarized as follows:

1. The interaction between the main and seepage flows causes a boundary layer or wedge to form near the bed. The layer influences the channel hydraulics, bed forms, stream power, and sediment transport in the zone of inflow seepage.
2. The development of the layer or wedge near the bed results in an effective increase in the bed elevation. The water depth decreased by as much as 15 percent for subcritical flow and remained constant for supercritical flow through the inflow seepage zone.
3. The stream power, in the reach where infiltration occurs, increases primarily due to an increase in the water surface slope. The mean velocity was observed to increase as much as 23 percent.
4. Inflow seepage caused significant changes in bed forms and subsequently in the resistance to flow.
5. Fluid shear and fluid particle to particle momentum transfer between the main and seepage flows increase turbulence along the interface.

FACILITIES

The investigation was conducted at the Engineering Research Center at Colorado State University. A 1.5-ft wide (45.7 cm), 2.0-ft (61 cm) deep, and 32-ft (9.8 m) long flume capable of recirculating water and sediment was used for this study. The flume was divided into three reaches. The upstream reach provided a 10-ft (3.0 m) length of channel for the flow, bed forms, surface waves, and sediment transport to stabilize. The upstream reach contained the head box, rock baffle, flow straightener, and wave suppressor. An inflow gallery was located in the mid one-third of the flume. The downstream reach served as an outrun section to minimize the effect of the backwater in the testing zone.

The inflow gallery consisted of perforated pipes installed in the bed of the flume perpendicular to the flow. The length of the inflow gallery was approximately 10 feet (3 m). The perforated pipes, spaced at one foot intervals, were overlain by a one-inch thick layer of one-fourth-inch gravel and a permeable geomembrane to diffuse the seepage flow. A piece of wire mesh was placed on top of the geomembrane and attached the gallery to the flume. Each of the perforated pipes was valved for flow regulation and connected to a common supply conduit.

A single circular pier was used in this study. The pier was 0.104 feet (3.2 cm) in diameter and two feet in length. The pier was placed in the center of the flume 0.75 feet (22.9 cm) from each side wall, and in the center of the inflow gallery, 15 feet (4.6 m) from the flume entrance. The Plexiglas pier extended from the top of the flume, through the flow and bed material, to the top of the wire fabric of the inflow gallery. Channel discharge was measured using a calibrated segmented orifice. Inflow seepage discharge was determined volumetrically. Twelve liters of water were diverted from the inflow discharge supply conduit and the elapsed time recorded. Several measurements of the inflow discharge were made for each test and averaged. The average inflow velocity was determined by dividing the inflow discharge by the area of the inflow gallery.

Water surface slope was measured by the use of three piezometer taps spaced 10 feet (3 m) apart and located on the left wall of the flume. The upstream tap was located in the approach section upstream of the inflow zone. The middle pressure tap was located in the zone of infiltration adjacent to the pier. The third tap was located downstream of the inflow zone.

The mapping of the bed was performed by using a point gauge and a movable carriage on rails mounted on the flume walls. Mapping was performed in cartesian coordinates. Distances in the longitudinal direction are stationed increasing upstream, while the lateral direction represented the distance from the left wall of the flume. The point gauge was capable

TABLE 1 SUMMARY OF DATA

Run	Channel* Discharge Q_c cfs	Mean Channel Velocity V_c fps	Channel Approach Depth d_c ft	Maximum Scour Depth d_{sm} ft	Maximum Scour Width W_{sm} ft	Channel Froude F_c
4-0	1.71	2.02	0.565	0.161	0.53	0.47
1		1.93	0.565	0.183	0.67	0.44
2		1.93	0.608	0.166	1.05	0.44
3		1.93	0.606	0.162	1.17	0.44
5-0	1.50	1.97	0.510	0.200	0.65	0.49
1		1.90	0.502	0.208	0.72	0.46
2		1.90	0.529	0.153	1.11	0.46
3		1.90	0.560	0.105	0.75	0.46
6-0	1.77	2.56	0.461	0.187	0.92	0.66
1		2.33	0.504	0.168	0.91	0.58
2		2.33	0.485	0.159	0.85	0.58
3		2.33	0.531	0.139	1.18	0.58
7-0	1.97	2.45	0.537	0.144	0.98	0.59
1		2.43	0.531	0.162	0.99	0.58
2		2.43	0.538	0.184	0.87	0.58
3		2.43	0.560	0.142	1.06	0.58
8-0	2.10	2.35	0.596	0.178	0.91	0.54
1		2.43	0.605	0.185	0.80	0.57
2		2.43	0.560	0.162	1.03	0.57
3		2.43	0.563	0.137	0.96	0.57
9-0	2.11	2.57	0.550	0.203	0.68	0.61
1		2.21	0.616	0.166	0.79	0.50
2		2.21	0.633	0.166	0.85	0.50
3		2.21	0.671	0.077	0.75	7.50
10-0	1.90	2.36	0.536	0.307	1.00	0.57
1		2.63	0.470	0.142	0.75	0.67
2		2.63	0.489	0.139	0.97	0.67
3		2.63	0.480	0.125	1.06	0.67
11-0	2.11	2.97	0.474	0.138	1.03	0.76
1		3.04	0.463	0.124	0.82	0.79
2		3.04	0.465	0.130	0.86	0.79
3		3.04	0.460	0.125	0.92	0.79

*Water temperature for all tests: 68°F ± 2°F.

of measuring the distance in the vertical direction from a fixed datum to a resolution of 0.001 feet (0.3 mm).

TESTING PROGRAM

A series of eight flume tests were conducted, in which the channel Froude number and the inflow seepage velocity were varied. In each of the eight tests, the channel was subjected to three inflow seepage velocities as well as a no-inflow seepage condition. Table 1 is a tabulation of the channel discharge, average channel velocity, maximum depth of scour, maximum width of scour, and channel Froude number. The run numbers refer to the test and the inflow rate. The inflow rate of zero indicates a no-inflow condition. The inflow rates of 1–3 indicate the range of inflow velocities tested.

The channel discharge varied from 1.504 cubic feet per second (cfs) (0.04 m^3/s) to 2.113 cfs (0.06 m^3/s). The channel Froude number varied from 0.44 to 0.79. The depth of flow ranged from 0.46 feet (11.3 cm) to 0.67 feet (20.4 cm). The seepage velocities for each test, tabulated in table 2, varied from zero to 2.59×10^{-3} fps (0.08 cm/s). The liquefaction critical seepage velocity for the bed material was 5.4×10^{-4}

TABLE 2 SEEPAGE INFLOW VELOCITIES

Test Number \ Inflow Condition	0	1	2	3
4	0	5.00	9.00	11.13
5	0	1.40	10.60	15.73
6	0	2.87	5.40	10.80
7	0	0.60	3.47	9.07
8	0	2.40	4.40	9.33
9	0	3.67	8.13	14.87
10	0	6.33	8.47	15.33
11	0	6.87	15.53	25.87

$v_i * 10^4$ fps

$v_{i_{CR}} = 5.4 \times 10^{-4}$ fps

fps (0.016 cm/s) and shall be referenced as the critical or threshold inflow velocity.

MATERIAL

The alluvial bed material was a silica river sand. A visual accumulation tube analysis (VA) was performed to determine the fall diameter of the material. The VA analysis determined the material to have a fall diameter of 0.27 mm. A sieve analysis indicated that the median grain size of the material was 0.33 mm. The VA and sieve analysis were performed after the material had been washed.

TESTING PROCEDURE

At the beginning of each test run, the channel discharge was established and the flume adjusted to produce a flow depth of approximately 0.5 ft (15.2 cm). The sediment depth was approximately 0.7 ft. The flume was then allowed to run a minimum of 12 hours to stabilize flow conditions relative to discharge and sediment recirculation in the channel. Stability of flow was subjectively judged to have been established when the bed elevation was steady for a period of at least one hour.

Data were collected after flow and sediment stabilization. Sediment transport rates were not measured. These data included the channel discharge, inflow discharge, water surface slope, and the mapping of the bed. After initial data collection, the inflow seepage was either initiated or increased. The channel was allowed to restabilize for a period of two to six hours and data were again collected.

The first data collection was conducted without inflow seepage. Subsequent subtests were conducted with inflow seepage as shown in table 2. The zero inflow test data were used to form a baseline to compare the influence of inflow seepage on scour around the pier. The higher seepage rates were sufficient to cause localized liquefaction of the bed material.

Flow depth was measured during the mapping of the bed. Cross sections were located upstream and downstream of the pier at 0.10-ft (3 cm) intervals. In each cross section, vertical elevations of the bed were measured every 0.2 feet (6.1 cm) or at each break in grade. At each cross section, water level readings were taken along with the measurements of the bed elevation. The difference between these measurements yielded the flow depth at each cross section. The water temperature for all tests was 68° ± 2°.

RESULTS

The initial step in the analysis was to compare the equilibrium scour depths obtained in this study with composite data presented by Jones (*10*). Figure 1 portrays the scour depths versus pier Reynolds number relationship. The scour depths resulting from the eight record tests for the no inflow condition are shown in figure 1. Although the maximum scour depths plot below the prediction relationship, the no-inflow scour depth data fall well within the data scatter of previous studies. Therefore, these test results compare favorably with accepted procedures for estimating pier scour depth.

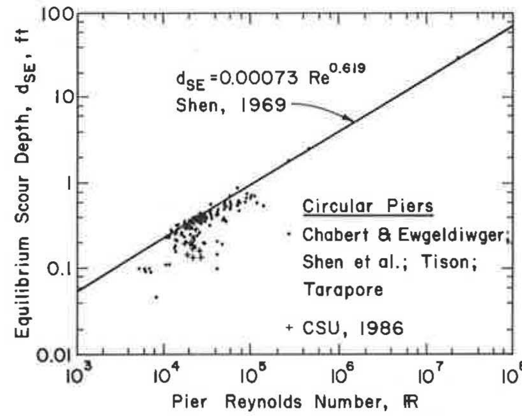

FIGURE 1 Scour depth versus pier Reynolds number (after Shen).

Scour Depth

During each test, the depth of scour was carefully monitored for each no-inflow and inflow seepage condition. The maximum scour depths, d_{sm}, adjacent to the circular pier without seepage inflow are presented in table 1. The maximum scour depths recorded without seepage inflow were compared to the maximum scour depths recorded with varying inflow seepage conditions. A typical longitudinal section relating the maximum scour depths of the no-inflow condition to the maximum inflow seepage condition is presented in figure 2 for test 10. It is observed in figure 2 that the presence of inflow seepage resulted in a decrease in the maximum depth of the scour hole. The decrease in scour hole depth in the presence of inflow seepage was observed for tests where the Froude number ranged from 0.4 to 0.7. In general, as the inflow seepage velocity rate increased, the maximum depth of scour became shallower. The data indicate that when the measured inflow seepage rate, v_i, exceeded the critical inflow seepage, v_{iCR}, rate by a factor of 3, the maximum scour hole depth decreased by 62 percent.

In order to trace the development of the pier scour depth, the ratio of the maximum scour depth with inflow to the

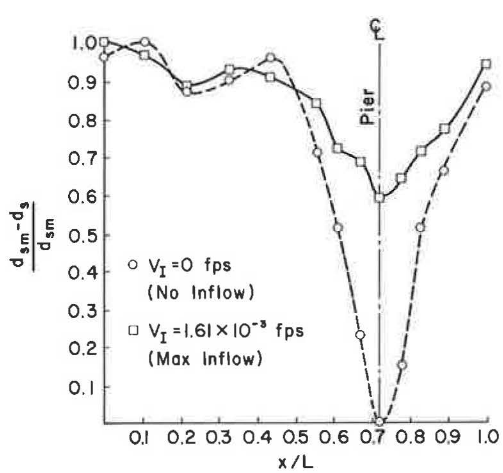

FIGURE 2 Longitudinal section for test run No. 10, L = 2.3 ft.

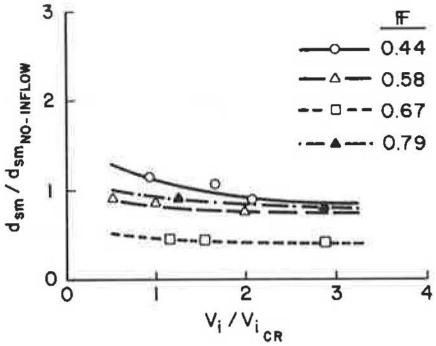

FIGURE 3 Pier scour depth as a function of inflow seepage and channel Froude number.

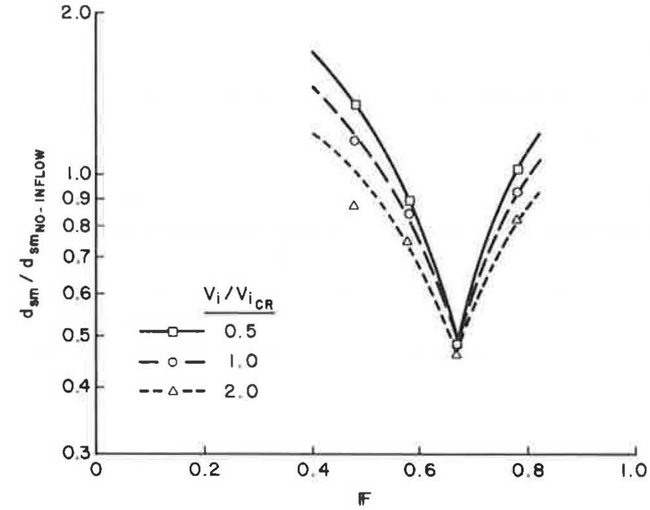

FIGURE 4 Pier scour depth versus channel Froude number.

maximum scour depth without inflow was related to the ratio of the inflow seepage velocity to the critical inflow seepage velocity as presented in figure 3. It is observed that when the Froude number ranges from 0.44 to 0.67, the pier scour depth decreases as the inflow seepage velocity increases. The maximum depth of scour with inflow seepage exceeded the maximum scour depth without inflow seepage by 20 percent when the channel Froude number was less than approximately 0.5 and the inflow seepage velocity was less than liquefaction. However, the trend is reversed when the Froude number exceeded 0.70. The trend reversal becomes evident where the depth of pier scour is related to the channel Froude number, as shown in figure 4. When the channel Froude number approached 0.7, the scour depth ratio converged at approximately 0.48, independent of inflow seepage velocity. The maximum scour depth then increased as the Froude number and the inflow seepage velocity increased. The scour hole depth degraded toward the no-inflow scour depth level.

Scour Width

The width of scour adjacent to the circular pier was measured and documented as shown in table 1 for each test. It is observed that for the tests in which the channel Froude number is less than 0.70, the maximum width of scour, W_{sm}, with inflow seepage exceeds the maximum width of scour without inflow seepage. An example of how inflow seepage affects the maximum width of scour is illustrated in figure 5. Figure 5 shows the bed and scour hole elevations relative to the maximum scour depth without inflow seepage at a cross section immediately downstream of the pier for test 9. Without inflow seepage, the scour hole was localized and well defined. When inflow seepage was introduced, the scour hole filled and the

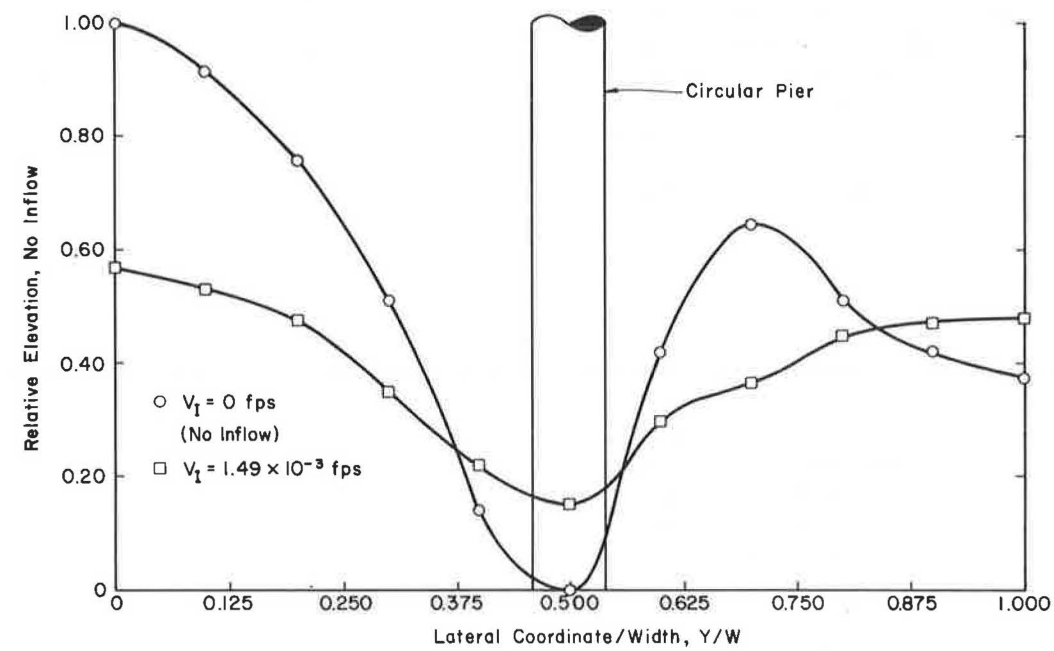

FIGURE 5 Inflow seepage influence on scour width.

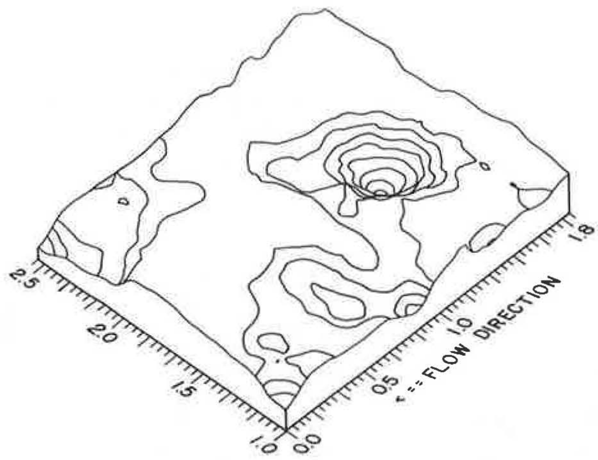

FIGURE 6 Alluvial bed topography for test 5, no inflow.

hole side slopes flattened. As the seepage rate increased, the width of scour increased. The presence of inflow seepage reduced the bed elevation relative to the maximum depth of scour by as much as 50 percent at a lateral distance of 8 pier diameters away from the pier. The flattened bed resembled a general scour condition between piers.

A series of topographic plots were constructed to illustrate the widening of the scour hole as shown in figures 6–9. The alluvial bed topography for test 5 indicates the general scour conditions that resulted from seepage inflow when inflow ranged from zero, Figure 6, to 290 percent of the liquefaction velocity, Figure 9.

In an attempt to correlate the maximum scour hole width, W_{sm}, to the inflow seepage velocity, v_i, and channel Froude numbers, the ratio of the maximum scour width with inflow seepage divided by the maximum scour width without inflow seepage (W_{sm} no-inflow) was related to the ratio of inflow seepage velocity divided by the critical inflow seepage velocity, as presented in figure 10. It is observed that for channel Froude numbers less than or equal to 0.50, inflow seepage increases scour hole widths by as much as 2.25 times the no inflow seepage condition. However, as the inflow velocity increases and the channel Froude number increases, the scour hole width decreases, as shown in figure 11.

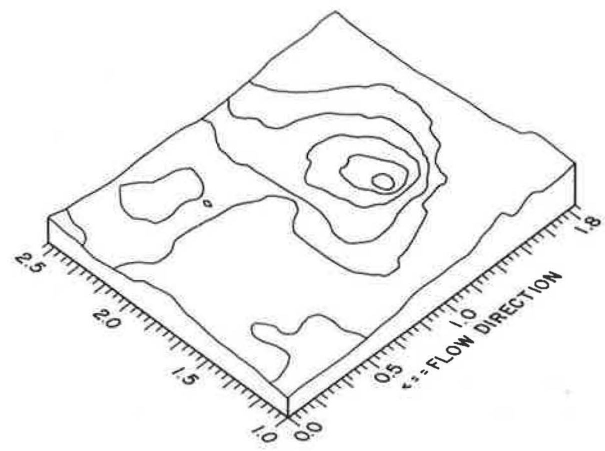

FIGURE 8 Alluvial bed topography for test 5, inflow 200 percent of liquefaction velocity.

GUIDELINES

Based on the test results and analysis, it appears that the presence of inflow seepage alters the extent of localized pier scour in an alluvial channel. The following guidelines are presented in accordance with these findings:

Seepage Influence on Scour Depth

F < 0.70 scour hole may aggregate up to 60 percent of the no-inflow condition if $V_i < V_{CR}$

F < 0.70 scour hole may degrade 20 percent if $V_i < V_{CR}$

F ≥ 0.70 scour hole degradates toward no-inflow level

Seepage Influence on Scour Width

F < 0.5 scour width may increase by 2.25 times over the no seepage condition

F < 0.7 adjacent bed elevations may degradate to 50 percent of the difference between the maximum scour depth and bed elevation for the non-inflow condition

F < 0.7 little difference between the inflow and the no-inflow conditions.

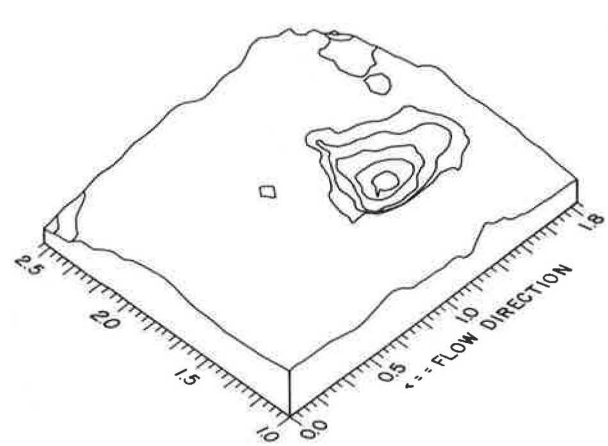

FIGURE 7 Alluvial bed topography for test 5, inflow at 25 percent of liquefaction velocity.

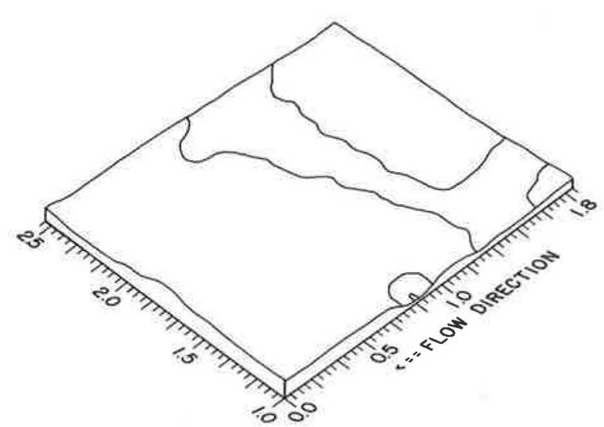

FIGURE 9 Alluvial bed topography for test 5, inflow at 290 percent of liquefaction velocity.

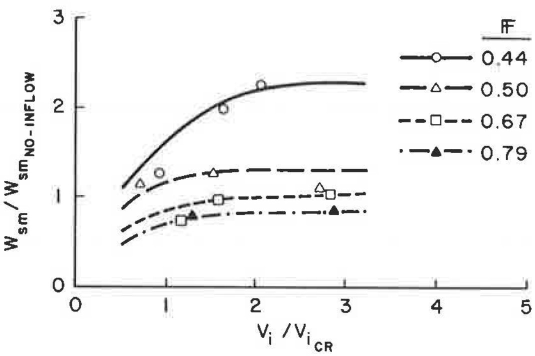

FIGURE 10 Relative scour width as a function of inflow seepage and channel Froude number.

These guidelines are based on tests in which channel Froude numbers range from 0.4 to 0.8 and inflow seepage velocities range from 0.5 to 3.0 times the critical inflow seepage velocity. Further, only a single circular pier was tested.

APPLICATIONS

It is recommended that all bridge sites be evaluated for the presence of inflow seepage. If inflow seepage is found or suspected, it would be prudent to increase the predicted pier scour hole depth independent of scour estimation procedure by approximately 20 percent. The presence of inflow seepage escalates the risk of increased scour around the pier. Also, the presence of inflow seepage can potentially increase the width of pier scour by 225 percent over no-inflow conditions. Although these guidelines reflect worst-case scenarios, they represent a real risk to existing and planned structures.

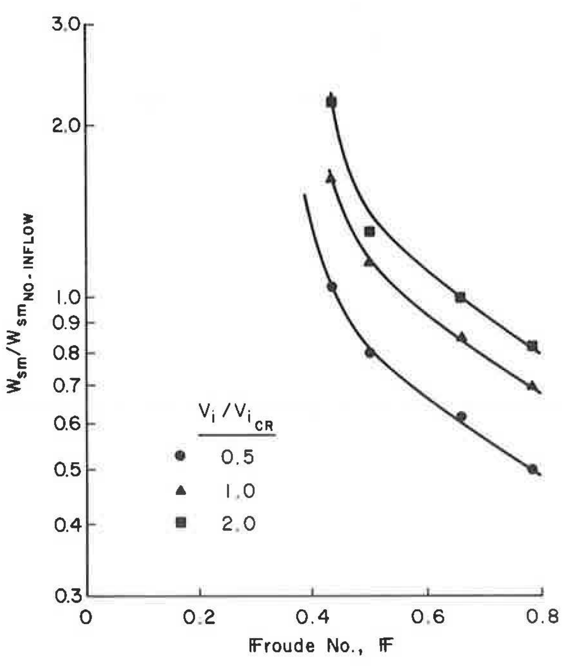

FIGURE 11 Scour width versus channel Froude number with varying inflow seepage conditions.

CONCLUSIONS

Inflow seepage significantly affects scour hole depth at piers. When the channel Froude number is less than 0.7 and the inflow seepage velocity is less than critical, the scour hole depth may increase by 20 percent over the no-inflow seepage scour depth. When the Froude number is less than 0.7 and the seepage velocity is greater than the critical seepage velocity, the scour hole depth decreases by 60 percent over the no-inflow seepage scour depth. When channel Froude numbers exceed 0.7 and inflow seepage is present, the scour depth decreases by 40 to 60 percent of the no-inflow maximum scour depth. General bad degradation occurs as seepage velocity increases.

The scour hole width may increase by 225 percent over the no-inflow seepage condition for Froude numbers of less than or equal to 0.5. The increase in scour hole width is attributed to the general bed degradation as the bed washes out.

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Detention Basins for Water Quality Improvement at a High Mountain Maintenance Station

JAMES A. RACIN AND RICHARD B. HOWELL

An evaluation of a detention basin system at a snow removal maintenance station is documented. In response to concerns by the US Forest Service, the basins were built as a mitigation measure to clarify storm runoff and snowmelt from the maintenance station before it entered Benwood Creek. The creek is a tributary of the headwaters of the South Fork of the American River. A portion of the creek was realigned. The three-basin system, completed in September 1981, is above elevation 7000 feet at the Echo Summit Maintenance Station in California. Sediment and dissolved materials in storm runoff and snowmelt from the maintenance yard were reduced. The capacity of the basins was approximately 10,000 cubic feet as measured in 1982. The basin riser outlets were fitted with grease rings, which retain most oil, grease, and floatables. Snowmelt was sampled and tested in spring 1982. Samples were tested for turbidity, chloride, specific conductance, and oil and grease. Storm runoff in fall 1982, was sampled and tested for turbidity, nonfilterable residue, specific conductance, filterable residue, and chloride. Sediment accumulation in the basins was measured, and a biological assessment of the construction impacts on Benwood Creek was made.

This paper documents an evaluation of a detention basin system for water quality improvement at a snow removal maintenance station. The station is a satellite facility that is used for stockpiling sand, cinders, and deicing salts to keep US 50 open to traffic during the winter. It is located at Echo Summit above elevation 7000 feet in the El Dorado National Forest, Sierra Nevada mountains of California. See figure 1.

Caltrans uses the land under the conditions of a permit issued by the US Forest Service. The Forest Service was concerned that runoff from the maintenance yard was adversely affecting the stream habitat in Benwood Creek, a tributary of the headwaters of the South Fork of the American River. Their measurements showed elevated values of turbidity and specific conductance in the creek. In response to this concern, Caltrans District 3 designed a channel realignment of East Benwood Creek and a detention basin system. The objective was to reduce turbidity and high concentrations of sediment and dissolved materials in the creek due to uncontrolled storm runoff and snowmelt.

Detention basins temporarily hold storm runoff or snowmelt and provide time for the water to clarify before returning to surface or ground water systems. The design criterion for locating the basins was to use existing areas without interfering with established operations. They were built by contract between the maintenance activity areas and nearby receiving waters.

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The Transportation Laboratory (TransLab) of Caltrans evaluated the detention basins in 1982, roughly one year after they were built. There were three phases of evaluation: sampling runoff, measuring and identifying the accumulated sediment, and making a biological assessment of Benwood Creek. The objective of sampling runoff was to compare concentrations of water quality parameters during snowmelt in spring and storms in fall, both upstream and downstream of the basins. Water samples were also collected in the basins. Besides sampling runoff, flow rate and precipitation were also observed and recorded. The sediment measurements were used to estimate basin rates-of-filling and to propose a cleanout schedule. The biological assessment was done to determine the biological potential of the creek and to estimate the effect of constructing the detention basins and channel realignment.

A prelude to any water quality study includes knowledge of the site hydrology and hydraulics. Field trips were made well in advance of the runoff events to design a sampling plan. Using automatic samplers was considered but was not done due to field conditions. A manual sampling plan was adopted and modified as required during runoff events. Enough sampling was done to demonstrate the mitigation effect of the basins. A mass balance was attempted but was not possible because there were no continuous flow records for all drainage areas. Conclusions and findings were based on graphical analyses and recorded observations. Sediment measurements and the biological assessments were done at times other than runoff sampling.

HYDROLOGY—SITE DESCRIPTION

The average annual precipitation measured as rain at Echo Summit is approximately 40 inches. Temperature extremes can range from 2 degrees Fahrenheit in the winter to 80 degrees Fahrenheit in the summer (*1*). Since the elevation at the maintenance station is above 7000 feet, most of the precipitation is in the form of snow and normally occurs from October through April. Most of the surface runoff is from the spring snowmelt in May and June.

The drainage areas at the maintenance station and nearby, the detention basins, and the sampling locations are shown in figure 2. Benwood Creek originates along the westerly side of the crest of the Sierra Nevada mountains and ultimately

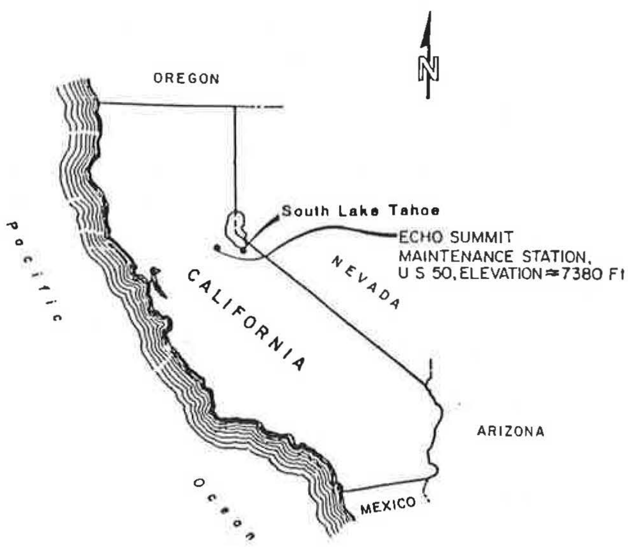

FIGURE 1 Location map.

joins the South Fork of the American River. Observed flows in the creek varied from >6 cubic feet per second (CFS) during snowmelt in spring to <1 CFS in fall toward the end of the dry season. During a dry year the creek may have no flow. The headwaters of Benwood Creek are spring-fed bogs immediately south of US 50 near the maintenance station. Multiple channels merge to form the east and west branches of Benwood Creek before passing under US 50 in culverts. The east and west branches of Benwood Creek receive runoff from areas IV and V. Area IV receives runoff from the headwaters of West Benwood Creek and US 50, while Area V receives runoff from the headwaters of East Benwood Creek and US 50.

The maintenance area, which contributes runoff to the creek, is approximately 5 acres, of which approximately 3 acres are paved with asphalt concrete. Before the detention basins were built, areas I and III drained to Benwood Creek via overland flow on bare ground. Area II drained to East Benwood Creek, which was connected to Benwood Creek via two 24-inch diameter corrugated steel pipe (CSP) culverts, which are under the maintenance station access road and a Forest Service road.

The detention basins were built to intercept all the runoff from areas I, II, and III. The two basins labeled Upper and Lower are new construction, while the third basin, labeled Old Channel, was the previous alignment of East Benwood Creek. The upper basin intercepts area I (approximately 50 percent), the old channel basin intercepts area II (approximately 45 percent), and the lower basin intercepts area III (approximately 5 percent) of the contributing area. The makeup of the drainage areas is discussed below.

Figure 3 shows a view of the access road from US 50, the salt hopper, sand, and the sand storage shed and a view of the upper and lower basins from the salt hopper. Figures 4 and 5 are close-ups of the upper, lower, and old channel basin and the realigned section of East Benwood Creek. The sides and bottoms of the basins are the local untreated, weathered, granitic soil. Asphalt concrete ramps were constructed in the new basins to allow equipment access for removing sediment. Imported borrow and the on-site excavated material were used to construct the berms that separated the upper and lower basins from each other and from the Forest Service access road and Benwood Creek. A berm was also constructed to separate the old channel basin from the realigned section of East Benwood Creek. It screens the maintenance yard from motorists on US 50. For aesthetic reasons, the basins and berms were contour-graded. Vegetation was left undisturbed where possible. The berms were seeded and containerized native trees and shrubs were planted. Table 1 shows the overall basin dimensions.

If the three basins were initially empty, a rainfall of approximately 0.70 inch would fill them. When the basins were evaluated in 1982, their capacity to detain runoff was approximately 10,000 cubic feet, excluding sediment already accumulated. Runoff that exceeds the capacity of the system

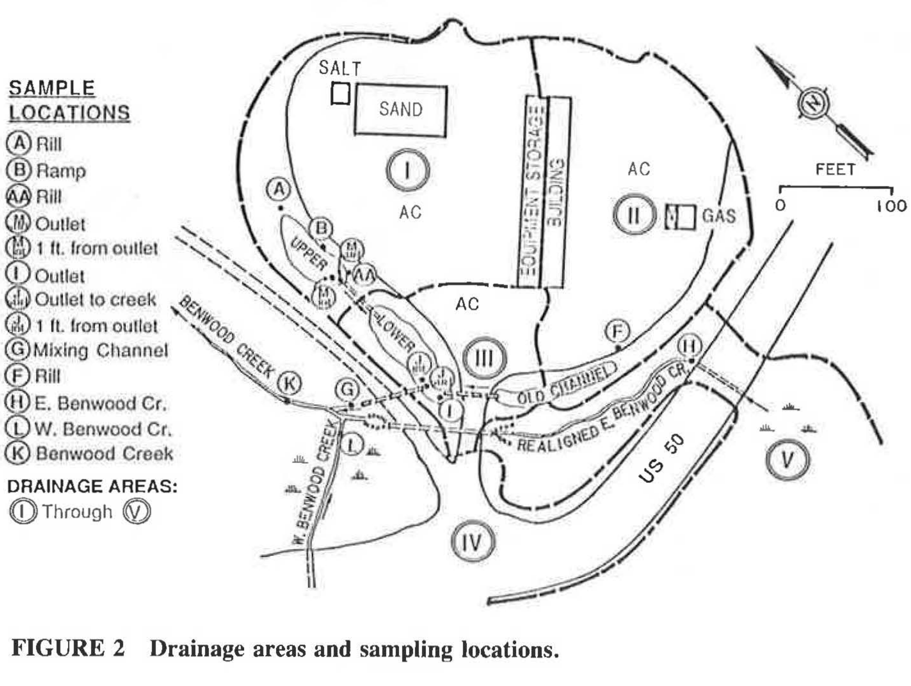

FIGURE 2 Drainage areas and sampling locations.

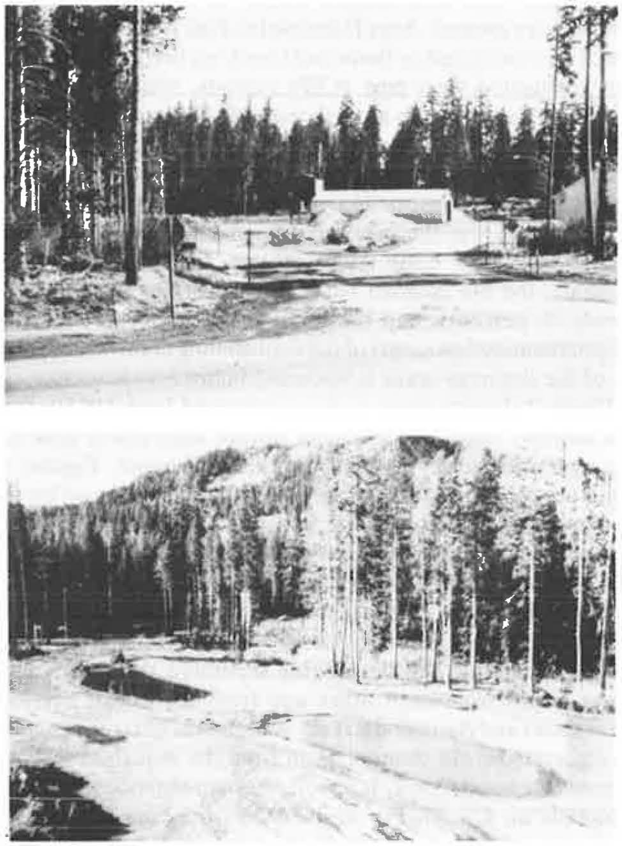

FIGURE 3 ***(Top)*** **Entrance to Echo Summit Maintenance Station.** ***(Bottom)*** **Upper and lower detention basins.**

is discharged from the lower basin to Benwood Creek. Detained water exits the basins by infiltration into the ground and by evaporation.

HYDRAULIC OPERATION

A schematic diagram of the three detention basins is shown in figure 6. The water surface elevations in the lower and old channel basins are the same, because the basin bottoms are connected by a CSP culvert (40-feet long by 24-inches in diameter) with a 0.1-foot elevation difference. When the basins are full, the upper basin water surface elevation is about 0.5 foot higher than the lower basin. The lower basin water surface elevation is about 2.60 feet higher than Benwood Creek, when the creek is 1.2 feet deep at the confluence with the mixing channel (see below). When the basins overflow excess runoff flows from the upper basin through a CSP outlet riser (2-feet high by 12-inches in diameter) and a CSP culvert (40-feet long by 12-inches in diameter with a 0.2-foot elevation difference) to the lower basin. The outlet riser operates like a circular weir, not a submerged orifice. A grease ring (24-inch diameter perforated CSP) was fitted around the riser. The top of the grease ring is 0.5 foot higher than the top of the outlet riser. Runoff flows through perforations in the grease ring and empties through the outlet riser (not perforated). There are two rows of 3-inch diameter perforations staggered and spaced approximately 8 inches on-center both horizontally and vertically. When the lower basin overflows, water passes through a CSP outlet riser (3.4 feet high by 24-inches in diameter) into a CSP culvert (56-feet long by 24-inches in diameter with a 0.2-foot elevation difference) and then to a mixing channel (28-feet long by 6-feet wide), which is part of the old alignment of East Benwood Creek. The end of the mixing channel is the confluence with Benwood Creek. The lower basin outlet riser also operates as a circular weir and is fitted with a grease ring made from a 36-inch diameter CSP with 3-inch perforations, like the upper basin grease ring. Submerged orifice flow never occurred at either the upper or lower basin outlet risers.

Figure 7 shows the grease rings in the upper and lower basins empty and partially full, respectively. Figure 8 shows a top view of the outlet riser, grease ring, and trash rack in the upper basin and the outlet riser and grease ring in the lower basin retaining some floatables (conifer pollens).

RUNOFF SAMPLING LOCATIONS AND PROCEDURES

Locations

Figure 2 shows where water samples were collected. Sample locations were chosen in the maintenance yard upstream of the basins to characterize the uncontrolled runoff that previously entered Benwood Creek. To characterize dilution water,

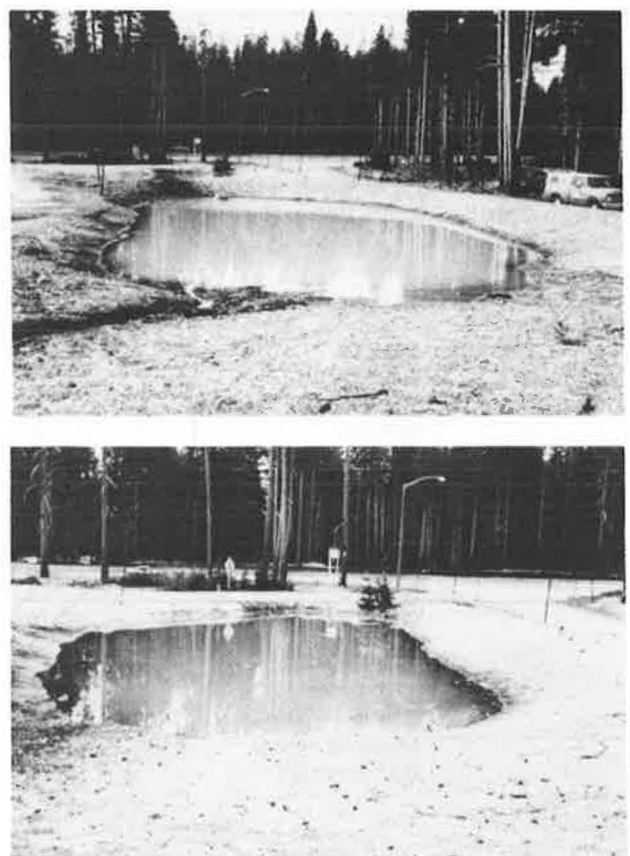

FIGURE 4 ***(Top)*** **Upper basin.** ***(Bottom)*** **Lower basin.**

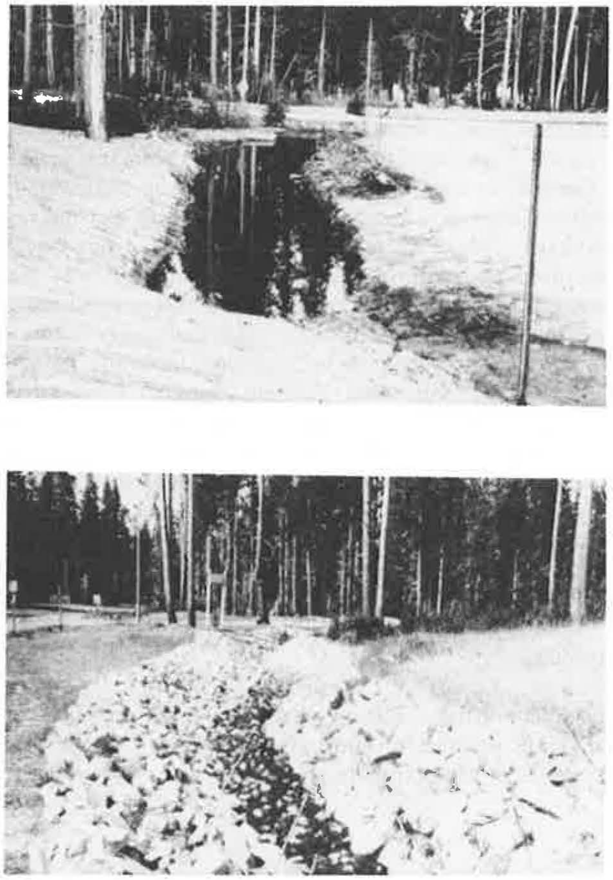

FIGURE 5 ***(Top)*** **Old channel basin.** ***(Bottom)*** **Realigned section of East Benwood Creek.**

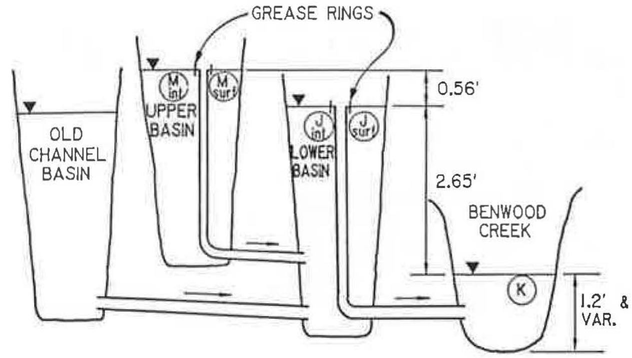

FIGURE 6 Schematic of detention basins.

samples were collected upstream in the east and west branches of Benwood Creek. To determine the effect of the basins, samples were collected in Benwood Creek downstream of the lower basin. Sampling was also done to characterize concentrations of water quality parameters in the basins.

Drainage area I consists of asphalt concrete pavement, a salt hopper, a sand storage shed, truck loading areas, and the upper basin. Locations A, B, and AA are just upstream of the upper basin. A and AA are rills just beyond the limits of paving, while B is a sheet flow area on the pavement above the cleanout ramp. (M-surf) is in the overflow jets at the upper basin outlet riser. (M-int) is in the upper basin one foot from the grease ring.

Drainage area II consists of asphalt concrete pavement, are fueling station, and an equipment storage building. Refueling and steam cleaning are occasionally done in area II; however, these activities normally take place at the base maintenance station in South Lake Tahoe. Location F is where runoff from drainage area II concentrates before it enters the old channel basin.

Drainage area III consists of the access road to the maintenance station and the lower basin. Location I is in the lower basin near the outlet of the 24-inch diameter CSP culvert, the connection to the Old Channel basin. (J-surf) is in the overflow jets at the lower basin outlet riser. (J-int) is in the lower basin one-foot from the grease ring.

Drainage area V consists of wetlands, forest, a small portion of US 50, and realigned East Benwood Creek. Location H is upstream of the maintenance station at the outlet of a 24-inch diameter CSP culvert that conveys East Benwood Creek under US 50.

Drainage area IV consists of wetlands, forest, approximately 0.5 acre of US 50, Benwood Creek, and the confluence of East and West Benwood Creek. Location L is just upstream of the maintenance station in West Benwood Creek, approximately 20-feet upstream of the confluence with East Benwood Creek. Location G is at the outlet of the 24-inch diameter CSP culvert from the lower basin. The waters from Benwood Creek and the lower basin are mixed near G in a transitional mixing channel 28-feet long by 6-feet wide, before the actual confluence with Benwood Creek. The mixing channel is part of the old alignment of East Benwood Creek. Location K is in Benwood Creek, 58-feet downstream of the outlet of the lower basin, 30-feet from the end of the transitional channel, and 42-feet downstream of the confluence of the east (realigned) and west branches of Benwood Creek.

Procedures

All water samples were collected in one-pint glass bottles. A DH-48 depth integrating sampler (*2*) was used at most locations except at rills, sheet flow areas, and outlet risers. Rills A, AA, and F were sampled manually by immersing the sample bottle in the runoff, specifically avoiding a scooping action.

TABLE 1 DETENTION BASIN DIMENSIONS

Basin	Maximum Length (ft)	Maximum Width (ft)	Maximum Depth (ft)	Side Slopes[a]	Surface Area (ft^2)	Capacity (ft^3)
Upper	83	34	3.5	5:1 (var)	2,360	4,320
Old Channel	112	11	2.3	3:1 (var)	1,070	1,350
Lower	99	40	3.5	5:1 (var)	2,670	6,475

[a]Horizontal:vertical.

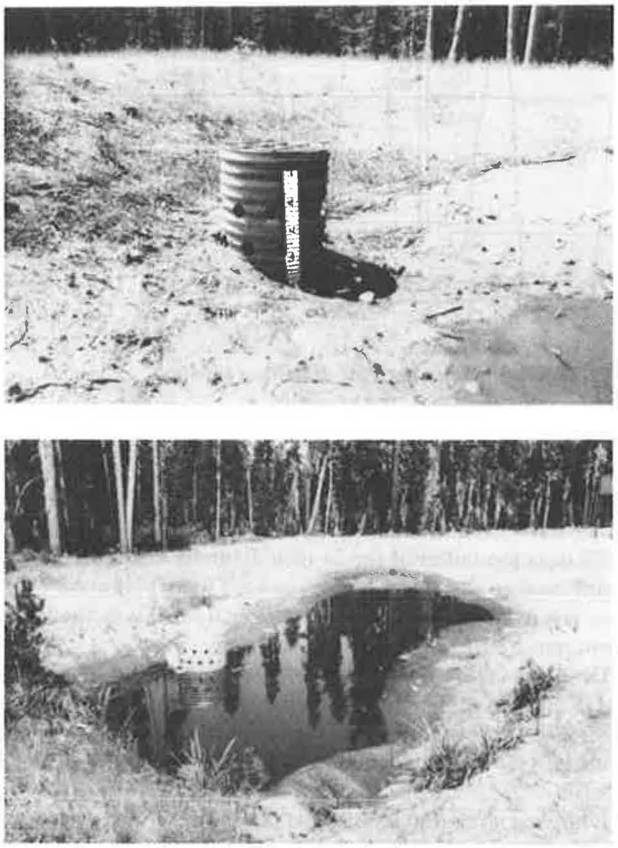

FIGURE 7 ***(Top)*** **Outlet pipe riser in upper basin.** ***(Bottom)*** **Outline pipe riser in lower basin.**

Sheet flow samples were collected at B by using a flexible piece of Plexiglas shaped as a funnel, which intercepted a one-foot wide section of runoff. The samples at (M-surf) and (J-surf) were collected manually by immersing the sample bottles in the overflow jets. All samples were stored in ice chests and brought back to TransLab, where they were analyzed according to test methods in Standard Methods (*3*).

Instantaneous flow rates were measured using one of the following techniques: velocity-area or time to fill a known volume. Flow rates over the circular weirs at (M-surf) and (J-surf) could not be computed with any of the standard weir formulae, because surface tension effects were observed and the heads never exceeded 0.10 foot.

A sample set consisted of taking at least one pint of runoff (when there was runoff) from each of the 13 locations shown in figure 2, and measuring flow rates. It required less than one hour to traverse the entire site and collect one set of data. The pint samples were discrete, because each of them was collected in about one minute or less. Composite samples (amounts smaller than one pint collected over a longer time until there was a pint of runoff) were not collected, because there were 13 locations, and there were only two people sampling.

Rainfall accumulation was measured with a post-mounted rain gage and was recorded periodically during storms. The gage was installed between the upper and lower basins.

SAMPLING RUNOFF—SPRING SNOWMELT

The 1981–82 water year was wet. The snow depth at Echo Summit was 136 inches on April 8, 1982, approximately 180 percent of normal. Snowmelt occurred gradually because temperatures were lower than normal in May and June (*4*).

The effectiveness of the detention basins in improving the quality of snowmelt was determined by plotting and analyzing the values of water quality parameters and reviewing the flow rate observations during the spring snowmelt in May 1982. Table 2 is a diary of the sampling dates and comments regarding maintenance activities or other factors that may have affected the water quality. At 9 of the 13 sample locations, normally three (but sometimes two) discrete, one-pint samples were collected at approximately one-minute intervals. Each discrete sample was tested for turbidity, specific conductance, chloride, and oil and grease. Because there were no large differences in values among the one-minute discrete samples, averaged values for each location were plotted (figures 9 through 12) for each sample set.

Turbidity

The detention basins effectively reduced turbidity values. Turbid runoff was due to fine particles being eroded from the stockpiles of sand, cinders, and roadway base material and

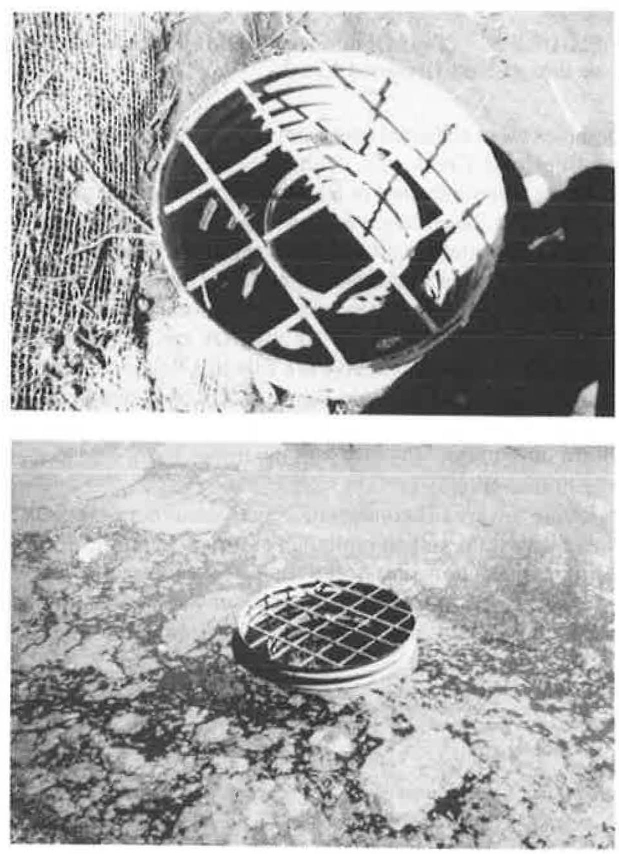

FIGURE 8 ***(Top)*** **Grease ring, trash rack, pipe riser outlet.** ***(Bottom)*** **Floatables retained by grease ring.**

TABLE 2 SPRING, 1982, SNOWMELT DIARY

Sampling Date	Comments
May 3 (AM)	Sunny, no maintenance activity No ice on basins Both basins overflowing Runoff was not measured, but was steady
May 5 (AM)	Five trucks loaded with sand near B (0920 to 1130) Thin ice on lower basin (greenish-colored water) No ice on upper basin (brownish-colored water) Both basins overflowing Silt plume visible in mixing zone G, but not at K Sunny with clear skies Flow rate at K is approximately 6.0 cfs
May 5 (PM)	No ice on any basins No changes in flow rates from AM Scum on lower basin retained by grease ring No maintenance activity during PM
May 10 (AM)	Snow showers previous night, partly cloudy, light snow shower from 1035 to 1045 Three trucks loaded with sand near B Most of snowpack on maintenance yard melted Both basins overflowing Flow rate at K is approximately 6.1 cfs, steady; slight increase over May 5 due to snow showers Silt plume visible at G, but not at K
May 17 (AM)	Partly cloudy No overflow from upper basin to lower basin Lower basin overflowing Thin oil film on old channel and lower basins Flow rate at K is approximately 5.3 cfs, steady
May 24 (AM)	Sunny with clear skies Flow rate at K is approximately 5.8 cfs, steady No runoff from rills at A or AA No overflow from upper basin to lower basin Lower basin overflowing Trout were seen 50 feet downstream of K

from bare soil in drainage area I. Turbidity values at the upper basin inlets (locations A and B) were several orders of magnitude higher than those measured in the lower basin outlet (J-surf) on May 5 and 10. The highest turbidity values occurred at locations A and B when trucks were being loaded with base material. See figure 9.

Fine particles can remain in suspension because there are no baffles or chambers in the basins and because there are relatively short distances from the basin inlets to the outlet risers. The suspended particles are 74 microns (No. 200 sieve) and smaller. The particle size was determined visually by inspecting the water samples in the field and the dried residue in the laboratory using the criterion of the Unified Soil Classification System (*5*). The values of turbidity were between 20 and 30 nephelometric turbidity units (NTU) at the lower basin outlet (J-surf).

Movement of a silt plume indicated that mixing was occurring in the channel near G. Samples were collected at G during spring snowmelt but not during the fall storms because concentrations in the mixing zone are transitional (*6*). The focus of this evaluation was to characterize concentrations downstream after mixing with the upstream dilution waters. The east and west branches of Benwood Creek diluted the effluent from the lower basin so that turbidity values downstream at K did not exceed 3 NTU.

Chloride and Specific Conductance

Before the detention basins were built the Forest Service measured chloride concentrations downstream of K in Benwood Creek. Values were greater than 800 mg/1 (milligrams per liter).

The source of chloride is dissolved deicing salt, NaCl (sodium chloride), from spillage around the salt hopper in drainage

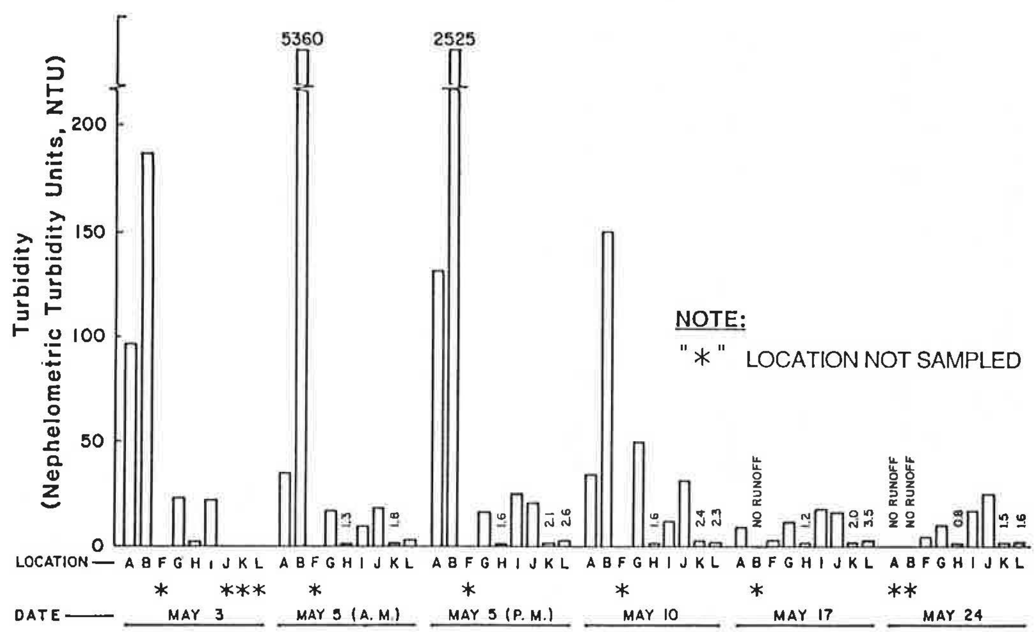

FIGURE 9 Turbidity: Spring snowmelt, 1982.

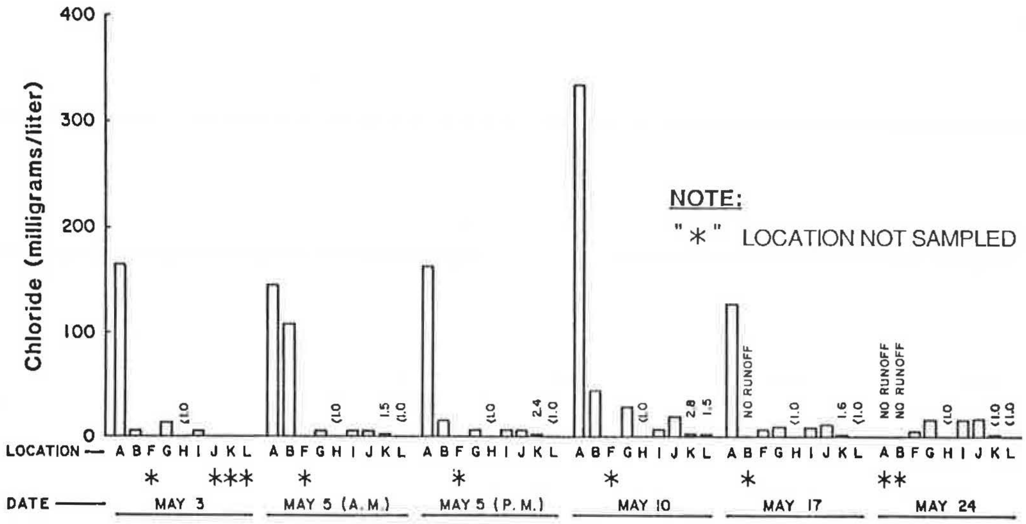

FIGURE 10 Chloride: Spring snowmelt, 1982.

area I. Specific conductance is a measure of the dissolved solids. In figures 10 and 11, when the chloride concentrations were high, so were the values of specific conductance. This indicates that chloride is a substantial fraction of the dissolved solids. Correlations between chloride and specific conductance at locations A and (J-surf) were 0.99 and 0.98 respectively.

The highest values of specific conductance and chloride concentration were at location A, downstream of the salt hopper. Chloride concentrations did not exceed 20 mg/l at the lower basin outlet (J-surf) due to the dilution water from drainage areas II and III. Chloride concentrations were less than 3 mg/l in Benwood Creek at K due to the additional dilution waters from drainage areas IV and V. Chloride concentrations were always less than 2 mg/l upstream in the east and west branches of Benwood Creek (at H and L).

The effluent concentrations at the lower basin outlet (J-surf) are considered not harmful or excessively high. The chloride standard for drinking water is 250 mg/l (*7*), and the LC50, the lethal concentration at which 50 per cent of rainbow trout die within 96 hours, is 12,200 mg/l (*8*).

Oil and Grease

The entire maintenance yard (drainage areas I, II, and III) and US 50 (drainage areas IV and V) have potential for oil accumulation and spills. No oil spills were observed during the dates of sampling. Normal maintenance and other vehicular activity was observed at the maintenance station and on the highway.

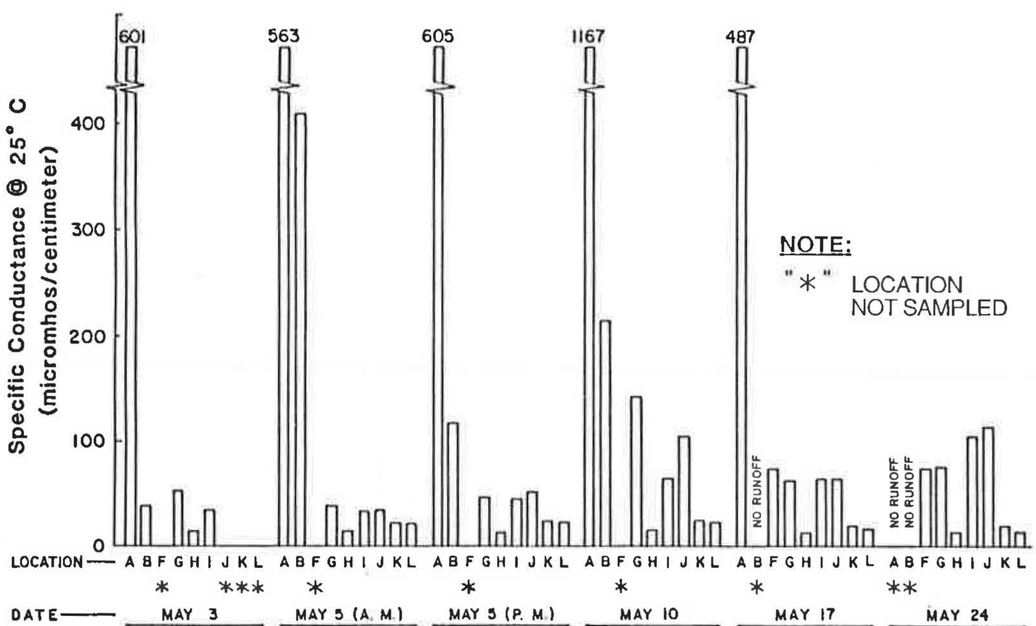

FIGURE 11 Specific conductance: Spring snowmelt, 1982.

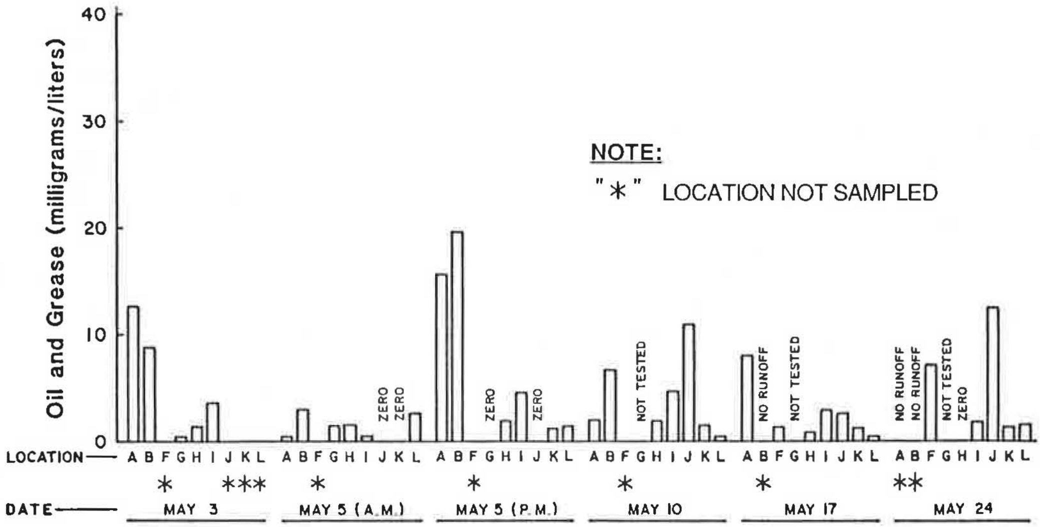

FIGURE 12 Oil and grease: Spring snowmelt, 1982.

The highest oil and grease concentration was 20 mg/1. It was collected at B above the cleanout ramp by the upper basin on May 5 in the afternoon after, not during, the truck loading operation. Concentrations were less than 15 mg/1 at the lower basin outlet (J-surf). Concentrations did not exceed 3 mg/1 in Benwood Creek at K. See figure 12.

The grease rings appear to retain most of the oil, grease, and other floatables in the lower and upper basins, except during transitional periods. During filling, or as the water level recedes due to infiltration into the ground, some oil and grease or floatables near the risers can flow through the 3-inch diameter perforations and become trapped in the concentric space between the grease ring and the riser. During the next period of overflow trapped substances will be discharged to the creek.

SAMPLING RUNOFF—FALL STORMS

Turbidity, specific conductance, chloride, filterable residue and nonfilterable residue, were measured for samples taken during two storms in October 1982. Oil and grease was not measured for these samples because there was no visible sheen on the water surface. The mitigation effect of the detention basins during storm runoff was determined the same way as for snowmelt, i.e., by plotting and analyzing the values of water quality parameters and reviewing the observations of flow rates, rainfall, and activities during sampling. That information is omitted from this paper for brevity; it is reported in (*9*). Only one, discrete pint sample was taken for each sample set, because of the small differences found among the one-minute discretes collected during snowmelt and because it took about 45 minutes to traverse the entire site.

There was not enough runoff on October 21 and 22 for the basins to overflow. The basins were about half-full initially, and after the storm they were just short of overflowing. Four sets of runoff samples were collected. A set sometimes consisted of less than thirteen samples when there was no runoff at certain locations. Two sets were collected on the 21st, from the beginning until the end of rainfall (approximately 0.56 inch of rain in 2.5 hours). Two more sets were collected on the 22nd, at which time there were sporadic drizzles that were not measurable.

Both the upper and lower basins were overflowing upon arrival at the site and continued to overflow during the October 25 rainstorm (1.13 inches), which changed to snow (5 inches) during the early hours of October 26. Sampling of runoff (4 sample sets) was done during the first 0.43 inches of rainfall (4.5 hours) on the 25th. The first sample set represented the tail of a rainfall/runoff event that occurred earlier, while the next three sets represented the 0.43 inches of rain. Four more sample sets were collected on the 26th, when there was no measurable snow or rain.

Turbidity and Nonfilterable Residue

Storm runoff samples were analyzed for both turbidity and nonfilterable residue. Values of turbidity and nonfilterable residue were higher on October 21 during the first hour at locations B and H (respectively 39, 29 NTU and 352, 35 mg/1) than during the second hour at B and H (respectively 37, 6 NTU and 42, 8 mg/1). This is called the first flush phenomenon.

It was only drizzling sporadically during the sampling periods on October 22, and there was little or no runoff entering the basins. Turbidity and nonfilterable residue values decreased at H ((2 NTU and 1 mg/1) as compared to the values on October 21 during the first hour of rainfall. The basins were not overflowing. Runoff in the three-basin system was infiltrating into the ground, as evidenced by the decrease in water surface elevations from morning to afternoon.

Analysis of the data from the storm on October 25 and 26 showed that values of turbidity and nonfilterable residue were lower at the basin outlets as compared to the basin inflows. At the lower basin outlet (J-surf) values of turbidity and nonfilterable residue were respectively, less than 18 NTU and 19 mg/1 and ranged from 5–10 NTU and 3–9 mg/1 less than the upper basin outlet (M-surf). Values of turbidity and nonfilterable residue at K (downstream, Benwood Creek) were less than 9 NTU and 18 mg/1.

During the latter two sampling periods on the 25th, the rainfall intensity was greater than 0.1 inch per hour. The nonfilterable residue values at K (12 and 17 mg/l) were slightly higher than at the lower basin outlet (J-surf) (10 and 12 mg/l). Turbidity values did not show this trend. Comparisons of nonfilterable residue at the two other upstream sampling locations, H (<2 and <2 mg/l) and L (<7 and <7 mg/l), could indicate that soil particles between the sample points in drainage area IV were loosened by the intense rain. Alternatively, the sampling technique and the laboratory method may not have been sensitive enough to positively explain these slight differences. The history of concentrations for all other storm runoff sample sets showed lower concentrations of nonfilterable residue at K than at (J-surf).

Specific Conductance, Filterable Residue, and Chloride

The interrelationship of specific conductance, filterable residue, and chloride appears to be consistent. Recall that the upper basin was approximately half-full at the start of the October 21 storm and was just short of overflowing on the 22nd. Thus, the volume of runoff collected in the basins was roughly doubled. The highest value of each parameter occurred in the upper basin at (M-int) during the first hour of sampling. Specific conductance was 1332 micromhos per centimeter, filterable residue was 704 mg/l, and chloride was 389 mg/l. As more runoff entered the upper basin, the concentrations decreased, and by the last sampling period on the 22nd the values of these parameters were respectively, 684 micromhos per centimeter, 340 mg/l, and 191 mg/l, roughly half their initial values.

On October 25 and 26 the chloride concentration did not exceed 40 mg/l at the lower basin outlet (J-surf). The highest observed chloride concentration did not exceed 25 mg/l in Benwood Creek, downstream at K. The corresponding highest values of filterable residue and specific conductance were respectively <67 mg/l and 109 micromhos per centimeter. Values of these parameters at K were correspondingly lower when the basins were not overflowing, indicating that there was not a large amount of dissolved materials generated in areas IV and V during these particular storms.

The relatively large volume of the three detention basins helps to dilute dissolved materials from the maintenance yard. Most of the NaCl came from area I. The data indicated that drainage areas II and III did not produce significant amounts of chloride or other dissolved materials. Dissolved solids in the overflow from the lower basin are further diluted with runoff from areas IV and V.

SEDIMENT ACCUMULATION

The second phase of evaluating the detention basin system consisted of measuring the amount of sediment that accumulated in the upper and lower basins. No sediment accumulation measurements were made in the old channel basin, because drainage area II did not contain any significant sources of sediment. The measurements were performed in September 1982, approximately one year after the basins were built. Sediment sources were stockpiles of sand and excavated materials in drainage area I, spillage from maintenance trucks in drainage area III, and, predominantly, unstable soil from the sides of the recently constructed basins.

Samples were collected for analysis of grain sizes at various locations in the basins and the creek. The material in the basins was mostly inorganic. Grain size curves are reported in (*9*). All samples were well-graded and ranged from sandy gravel to silty sand (upper basin), from gravelly sand to silty sand (lower basin), and from sandy gravel to gravelly sand (Benwood Creek). Grain sizes of the sediment samples were compared to particle sizes of snowmelt and storm runoff samples. The basins retain most of the sediment conveyed to them via runoff. Dried samples of nonfilterable residue showed that only particles smaller than 74 microns (200 sieve) escaped from the lower basin.

The amount of sediment retained in the basins was measured by probing through soft sediment to firmer material with a steel rod at several cross sections, when there was approximately 2 feet of water in the basins. Sediment volumes were computed using the average end area method. The amounts of sediment accumulated in one year were

upper basin: 535 cubic feet (12.4 percent full)
lower basin: 1285 cubic feet (19.8 percent full)

The amounts of sediment shown above reduced the design capacities listed in table 1 for temporary water storage. The capacities for water storage before the Fall 1982 storms were 3785 cubic feet for the upper basin, 1350 cubic feet for the old channel basin, and 5190 cubic feet for the lower basin. Thus before the October storms were sampled for runoff, the capacity of the three-basin system was 10,325 cubic feet.

In (*9*) the same sediment accumulation rates were assumed for future years. It was estimated that the upper and lower basins should be cleaned out every two years. A field inspection and discussion between the maintenance foreman and the author in June 1987 revealed that the basins had not been cleaned out since they were built. There was water in all three basins. No measurements were taken; however it was estimated that the basins were not half-full of sediment. The culvert outlet from the old channel to the lower basin was completely plugged with sediment, and the outlet of the 12-inch culvert from the upper to the lower was 9 inches full of sediment. A South Lake Tahoe maintenance crew cleaned the culverts in early November 1987.

BIOLOGICAL REVIEW OF BENWOOD CREEK

The third phase of evaluating the detention basin system consisted of assessing the aquatic habitat of Benwood Creek. The objective was to determine the biological potential of the creek and to estimate the effect of constructing the detention basins and channel realignment. The assessment was made in September 1982, one year after construction by a TransLab aquatic biologist and a US Forest Service hydrologist. At that time the flow in Benwood Creek appeared normal, approximately 1 cfs, and no snow was present.

The bottom of the realigned section of East Benwood Creek is rock rubble (average size from 2 to 4 inches) and is covered with fine silt. Unlike the bog, which has a dense cover, the rechanneled section is exposed. At the end of the rechanneled section, the stream passes through a second steel culvert before entering the old streambed. The old bed is composed of an alternating series of runs and deep pools. Vegetation covers the banks, but some stretches of the stream are exposed during midday. Bottom sediments were rock cobble in the runs and silt and sand in the pools.

The water leaving the bog is acidic and brown, which indicates large amounts of organic acids. There was no change in temperature within the bog. If there is any aquatic life in this area none was observed, due to the small channels and dense vegetation.

In the rechanneled section, especially near the culvert under US 50 and sampling location H, there was reddish brown floc on the cobbles. The floc was caused by the precipitation of iron, probably as ferric hydroxide in association with iron bacteria. Several different types of aquatic insects were observed; however, a single species of chironomid larva was more prevalent than any other species. The water temperature increased several degrees in the rechanneled section.

In the old streambed the organic acid content was sufficient to color the water; however iron precipitation was not evident. Many different species of aquatic insects were observed in the streambed and leaf litter.

Benwood Creek seems typical of streams originating in spring-fed bogs in noncalcareous soils. The precipitation of iron and the low pH are characteristic of such streams. The increase in temperature and presence of large numbers of single species in the rechanneled area is due to the construction, not to pollution from the maintenance yard. The condition of the aquatic habitat in the rechanneled area is highly disturbed due to the construction and is just beginning to stabilize. The condition of the aquatic habitat above and below the rechanneled area is not significantly different from any other mountain stream originating in a similar bog. By November 1987 the surviving trees and shrubs that were planted on the berms and along the rechanneled section appeared to be growing slowly.

CONCLUSIONS AND FINDINGS

The detention basins effectively reduced turbidity values from the maintenance yard runoff during conditions of snowmelt and storm water runoff. Turbidity values measured at eroded areas and paved ramps leading into the basins from the maintenance yard were greater than those in Benwood Creek by two, and sometimes three, orders of magnitude. Downstream values in Benwood Creek did not exceed 10 nephelometric turbidity units (NTU). Turbidity values measured downstream of the maintenance yard in Benwood Creek were only slightly greater than those measured upstream of the detention basins.

Shock loadings of nonfilterable residue in Benwood Creek from maintenance yard runoff will be mitigated, as long as there is capacity for runoff and as long as accumulated sediment is periodically removed from the basins and the interconnecting culverts. Concentrations of nonfilterable residue did not exceed 20 milligrams per liter (mg/1). Particles of sediment smaller than 74 microns (No. 200 sieve) can remain in suspension and can flow through the lower basin into Benwood Creek.

The old channel and lower detention basins did not produce large amounts of chlorides, thus the runoff collected by them diluted dissolved materials and NaCl deicing salts from the upper basin. Chloride concentrations in Benwood Creek were below 30 mg/1 as compared to approximately 800 mg/1 before the basins were built.

The grease rings, which were fitted around the outlet risers, appeared to prevent most of the oil, grease, and floatables from escaping the basins. During the transitional period (filling or emptying), oil and grease or floatables near the risers can flow through the 3-inch diameter perforations and become trapped between the grease ring and outlet. The trapped substances will flow through the outlet during the next overflow event. Oil and grease concentrations from the maintenance yard during snowmelt were approximately 20 mg/1, while values in Benwood Creek upstream and downstream of the basins did not exceed 3 mg/1.

Measured snowmelt rates were 2 to 5 times greater than storm runoff rates. Snowmelt produced steady, gradually changing flows as compared to unsteady, rapidly changing flows from rainstorms.

The runoff detained in the three basins ultimately infiltrates into the ground and/or evaporates.

The basins are expected to remain aerobic, since they are shallow (less than 3.5 feet deep when full), and they will remain aerobic as long as they do not receive large amounts of decaying leaves or any human or animal waste.

Basin cleanout schedules are not easily forecasted. Original estimates showed that the basins would need to be cleaned out approximately every two years. After almost six years of operation there appears to be adequate volume for retaining sediment in the basins; however, the interconnecting culverts needed to be cleaned so the basins could function as they were designed.

The biological assessment indicated that the aquatic habitat in the realigned channel was highly disturbed but was beginning to stabilize. The condition of the aquatic habitat upstream and downstream of the realigned channel was not significantly different than any other mountain stream originating in a similar bog.

RECOMMENDATIONS AND APPLICATIONS

Maintenance yards should be inspected to determine if drainage from the yard is affecting nearby streams, lakes, or other bodies of water. It is usually not a trivial task to assign a dollar value to mountain streams and their supporting watersheds. Building and operating a maintenance station without consideration or protection of nearby water resources can destroy the valuable uses of the water and may lead to regulatory agency enforcement actions. When impacts are found or potential impacts are identified, a detention basin system should be considered as a mitigation measure.

A properly designed, constructed, and maintained detention basin system can effectively reduce the turbidity levels and sediment loads from maintenance yard runoff. Drainage areas, levels and kinds of maintenance activities, and the uses of nearby receiving waters must be studied before sizing and locating basins. The distance between the outlet of the basins and the major points of inflow should be as long as possible. Baffles should be considered for situations where berms cannot be used to prevent inflows from going directly to the outlet. Baffles reroute primary inflows, reduce turbulence, and inhibit resuspension of fine particles already deposited in the basins.

If oil and grease or other floatables are not wanted or allowed in the effluent, a floating, sorbent boom can be installed around grease rings. The floating boom will absorb and block oil and grease and other floatables from escaping the basins during the periods when the water level in the basins is fluctuating and near the level of the perforations in a grease ring. Floating sorbent booms must be changed periodically.

The materials used to construct the outlet structures of basins, e.g., weirs or spillway aprons, should be nonerosive to prevent nonfilterable residue from entering the effluent.

Generally, when a basin becomes half-full of sediment, it should be cleaned out to provide more volume for dilution and to maintain trap efficiency. The outlet risers should be banded or clearly marked at the half-full level when the detention basins are first built, so that a simple visual inspection will show how much sediment has accumulated. A minimum of one inspection per year is suggested, when the basins are mostly dry.

Culverts that interconnect basins and outlets should also be inspected annually and cleaned when they are more than 50 percent plugged. When it is feasible, the outlet elevations of the interconnecting culverts should be constructed such that the flow lines are at the half-full level of the basin.

Maintenance yards should be kept clean. Spills of salt, sand, and cinders around bunkers and in the yard should be cleaned-up before storms. Where it is feasible, diked and/or covered areas can be used to contain materials in the yard. Frequent cleanups will help prevent these materials from entering the runoff in the first place.

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