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61

Bridge Hydraulics

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61.1 Introduction

This chapter presents bridge engineers basic concepts, methods, and procedures used in bridge hydraulic analysis and design. It involves hydrology study, hydraulic analysis, on-site drainage design, and bridge scour evaluation.

Hydrology study for bridge design mainly deals with the properties, distribution, and circulation of water on and above the land surface. The primary objective is to determine either the peak discharge or the flood hydrograph, in some cases both, at the highway stream crossings. Hydraulic analysis provides essential methods to determine runoff discharges, water profiles, and velocity distribution. The on-site drainage design part of this chapter is presented with the basic procedures and references for bridge engineers to design bridge drainage.

Bridge scour is a big part of this chapter. Bridge engineers are systematically introduced to concepts of various scour types, presented with procedures and methodology to calculate and evaluate bridge scour depths, provided with guidelines to conduct bridge scour investigation and to design scour preventive measures.

61.2 Bridge Hydrology and Hydraulics

61.2.1 Hydrology

61.2.1.1 Collection of Data

Hydraulic data for the hydrology study may be obtained from the following sources: as-built plans, site investigations and field surveys, bridge maintenance books, hydraulic files from experienced report writers, files of government agencies such as the U.S. Corps of Engineers studies, U.S. Geological Survey (USGS), Soil Conservation Service, and FEMA studies, rainfall data from local water agencies, stream gauge data, USGS and state water agency reservoir regulation, aerial photographs, and floodways, etc.

Site investigations should always be conducted except in the simplest cases. Field surveys are very important because they can reveal conditions that are not readily apparent from maps, aerial photographs and previous studies. The typical data collected during a field survey include high water marks, scour potential, stream stability, nearby drainage structures, changes in land use not indicated on maps, debris potential, and nearby physical features. See HEC-19, Attachment D [16] for a typical Survey Data Report Form.

61.2.1.2 Drainage Basin

The area of the drainage basin above a given point on a stream is a major contributing factor to the amount of flow past that point. For given conditions, the peak flow at the proposed site is approximately proportional to the drainage area.

The shape of a basin affects the peak discharge. Long, narrow basins generally give lower peak discharges than pear-shaped basins. The slope of the basin is a major factor in the calculation of the time of concentration of a basin. Steep slopes tend to result in shorter times of concentration and flatter slopes tend to increase the time of concentration. The mean elevation of a drainage basin is an important characteristic affecting runoff. Higher elevation basins can receive a significant amount of precipitation as snow. A basin orientation with respect to the direction of storm movement can affect peak discharge. Storms moving upstream tend to produce lower peaks than those moving downstream.

61.2.1.3 Discharge

There are several hydrologic methods to determine discharge. Most of the methods for estimating flood flows are based on statistical analyses of rainfall and runoff records and involve preliminary or trial selections of alternative designs that are judged to meet the site conditions and to accommodate the flood flows selected for analysis.

Flood flow frequencies are usually calculated for discharges of 2.33 years through the overtopping flood. The frequency flow of 2.33 years is considered to be the mean annual discharge. The base flood is the 100-year discharge (1% frequency). The design discharge is the 50-year discharge (2% frequency) or the greatest of record, if practical. Many times, the historical flood is so large that a structure to handle the flow becomes uneconomical and is not warranted. It is the engineer's responsibility to determine the design discharge. The overtopping discharge is calculated at the site, but may overtop the roadway some distance away from the site.

Changes in land use can increase the surface water runoff. Future land-use changes that can be reasonably anticipated to occur in the design life should be used in the hydrology study. The type of surface soil is a major factor in the peak discharge calculation. Rock formations underlying the surface and other geophysical characteristics such as volcanic, glacial, and river deposits can have a significant effect on runoff. In the United States, the major source of soil information is the Soil Conservation Service (SCS). Detention storage can have a significant effect on reducing the peak discharge from a basin, depending upon its size and location in the basin.

The most commonly used methods to determine discharges are

1. Rational method
2. Statistical Gauge Analysis Methods
3. Discharge comparison of adjacent basins from gauge analysis
4. Regional flood-frequency equations
5. Design hydrograph

The results from various methods of determining discharge should be compared, not averaged.

61.2.1.3.1 Rational Method

The rational method is one of the oldest flood calculation methods and was first employed in Ireland in urban engineering in 1847. This method is based on the following assumptions:

TABLE 61.1 Runoff Coefficients for Developed Areas

Type of Drainage Area	Runoff Coefficient
Business	
Downtown areas	0.70–0.95
Neighborhood areas	0.50–0.70
Residential areas	
Single-family areas	0.30–0.50
Multiunits, detached	0.40–0.60
Multiunits, attached	0.60–0.75
Suburban	0.25–0.40
Apartment dwelling areas	0.50–0.70
Industrial	
Light areas	0.50–0.80
Heavy areas	0.60–0.90
Parks, cemeteries	0.10–0.25
Playgrounds	0.20–0.40
Railroad yard areas	0.20–0.40
Unimproved areas	0.10–0.30
Lawns	
Sandy soil, flat, 2%	0.05–0.10
Sandy soil, average, 2–7%	0.10–0.15
Sandy soil, steep, 7%	0.15–0.20
Heavy soil, flat, 2%	0.13–0.17
Heavy soil, average, 2–7%	0.18–0.25
Heavy soil, steep, 7%	0.25–0.35
Streets	
Asphaltic	0.70–0.95
Concrete	0.80–0.95
Brick	0.70–0.85
Drives and walks	0.75–0.85
Roofs	0.75–0.95

1. Drainage area is smaller than 300 acres.
2. Peak flow occurs when all of the watershed is contributing.
3. The rainfall intensity is uniform over a duration equal to or greater than the time of concentration, T_c.
4. The frequency of the peak flow is equal to the frequency of the rainfall intensity.

$$Q = C\,i\,A \tag{61.1}$$

where

Q = discharge, in cubic foot per second

C = runoff coefficient (in %) can be determined in the field and from Tables 61.1 and 61.2 [5,16] or a weighted C value is used when the basin has varying amounts of different cover. The weighted C value is determined as follows:

$$C = \frac{\sum C_j A_j}{\sum A_j} \tag{61.2}$$

i = rainfall intensity (in inches per hour) can be determined from either regional IDF maps or individual IDF curves

A = drainage basin area (in acres) is determined from topographic map

(*Note*: 1 sq. mile = 640 acres = 0.386 sq. kilometer)

TABLE 61.2 Runoff Coefficients for Undeveloped Area Watershed Types

Soil	0.12–0.16 No effective soil cover, either rock or thin soil mantle of negligible infiltration capacity	0.08–0.12 Slow to take up water, clay or shallow loam soils of low infiltration capacity, imperfectly or poorly drained	0.06–0.08 Normal, well-drained light or medium-textured soils, sandy loams, silt and silt loams	0.04–0.06 High, deep sand or other soil that takes up water readily, very light well-drained soils
Vegetal Cover	0.12–0.16 No effective plant cover, bare or very sparse cover	0.08–0.12 Poor to fair; clean cultivation crops, or poor natural cover, less than 20% of drainage area over good cover	0.06–0.08 Fair to good; about 50% of area in good grassland or woodland, not more than 50% of area in cultivated crops	0.04–0.06 Good to excellent; about 90% of drainage area in good grassland, woodland or equivalent cover
Surface Storage	0.10–0.12 Negligible surface depression few and shallow, drainageways steep and small, no marshes	0.08–0.10 Low, well-defined system of small drainageways; no ponds or marshes	0.06–0.08 Normal; considerable surface depression storage; lakes and pond marshes	0.04–0.06 High; surface storage, high; drainage system not sharply defined; large floodplain storage or large number of ponds or marshes

The time of concentration for a pear-shaped drainage basin can be determined using a combined overland and channel flow equation, the Kirpich equation:

$$T_c = 0.0195(L / S^{0.5})^{0.77} \tag{61.3}$$

where
T_c = Time of concentration in minutes
L = Horizontally projected length of watershed in meters
S = H/L (H = difference in elevation between the most remote point in the basin and the outlet in meters)

61.2.1.3.2 Statistical Gauge Analysis Methods

The following two methods are the major statistical analysis methods which are used with stream gauge records in the hydrological analysis.

1. Log Pearson Type III method
2. Gumbel extreme value method

The use of stream gauge records is a preferred method of estimating discharge/frequencies since they reflect actual climatology and runoff. Discharge records, if available, may be obtained from a state department of water resources in the United States. A good record set should contain at least 25 years of continuous records.

It is important, however, to review each individual stream gauge record carefully to ensure that the database is consistent with good statistical analysis practice. For example, a drainage basin with a large storage facility will result in a skewed or inconsistent database since smaller basin discharges will be influenced to a much greater extent than large discharges.

The most current published stream gauge description page should be reviewed to obtain a complete idea of the background for that record. A note should be given to changes in basin area over time, diversions, revisions, etc. All reliable historical data outside of the recorded period should be included. The adjacent gauge records for supplemental information should be checked and utilized to extend the record if it is possible. Natural runoff data should be separated from later controlled data. It is known that high-altitude basin snowmelt discharges are not compatible with rain flood discharges. The zero years must also be accounted for by adjusting the final plot positions, not by inclusion as minor flows. The generalized skew number can be obtained from the chart in Bulletin No.17 B [8].

Quite often the database requires modification for use in a Log Pearson III analysis. Occasionally, a high outlier, but more often low outliers, will need to be removed from the database to avoid skewing results. This need is determined for high outliers by using $Q_H = \bar{Q}_H + K\,S_H$, and low outliers by using $Q_L = \bar{Q}_L + K\,S_L$, where K is a factor determined by the sample size, $\bar{Q}_H$ and $\bar{Q}_L$ are the high and low mean logarithm of systematic peaks, Q_H and Q_L are the high and low outlier thresholds in log units, S_H and S_L are the high and low standard deviations of the logarithmic distribution. Refer to FHWA HEC-19, Hydrology [16] or USGS Bulletin 17B [8] for this method and to find the values of K.

The data to be plotted are "PEAK DISCHARGE, Q (CFS)" vs. "PROBABILITY, Pr" as shown in the example in Figure 61.1. This plot usually results in a very flat curve with a reasonably straight center portion. An extension of this center portion gives a line for interpolation of the various needed discharges and frequencies.

The engineer should use an adjusted skew, which is calculated from the generalized and station skews. Generalized skews should be developed from at least 40 stations with each station having at least 25 years of record.

The equation for the adjusted skew is

$$G_w = \frac{MSE_{G_S}(G_L) + MSE_{G_L}(G_S)}{MSE_{G_S} + MSE_{G_L}} \tag{61.4}$$

where

G_w = weighted skew coefficient
G_S = station skew
G_L = generalized skew
MSE_{G_S} = mean square error of station skew
MSE_{G_L} = mean square error of generalized skew

The entire Log Pearson type III procedure is covered by Bulletin No. 17B, "Guidelines for Determining Flood Flow Frequency" [8].

The Gumbel extreme value method, sometimes called the double-exponential distribution of extreme values, has also been used to describe the distribution of hydrological variables, especially the peak discharges. It is based on the assumption that the cumulative frequency distribution of the largest values of samples drawn from a large population can be described by the following equation:

$$f(Q) = e^{-e^{a(Q-b)}} \tag{61.5}$$

where

$$a = \frac{1.281}{S}$$

$$b = \bar{Q} - 0.450\,S$$

S = standard deviation
$\bar{Q}$ = mean annual flow

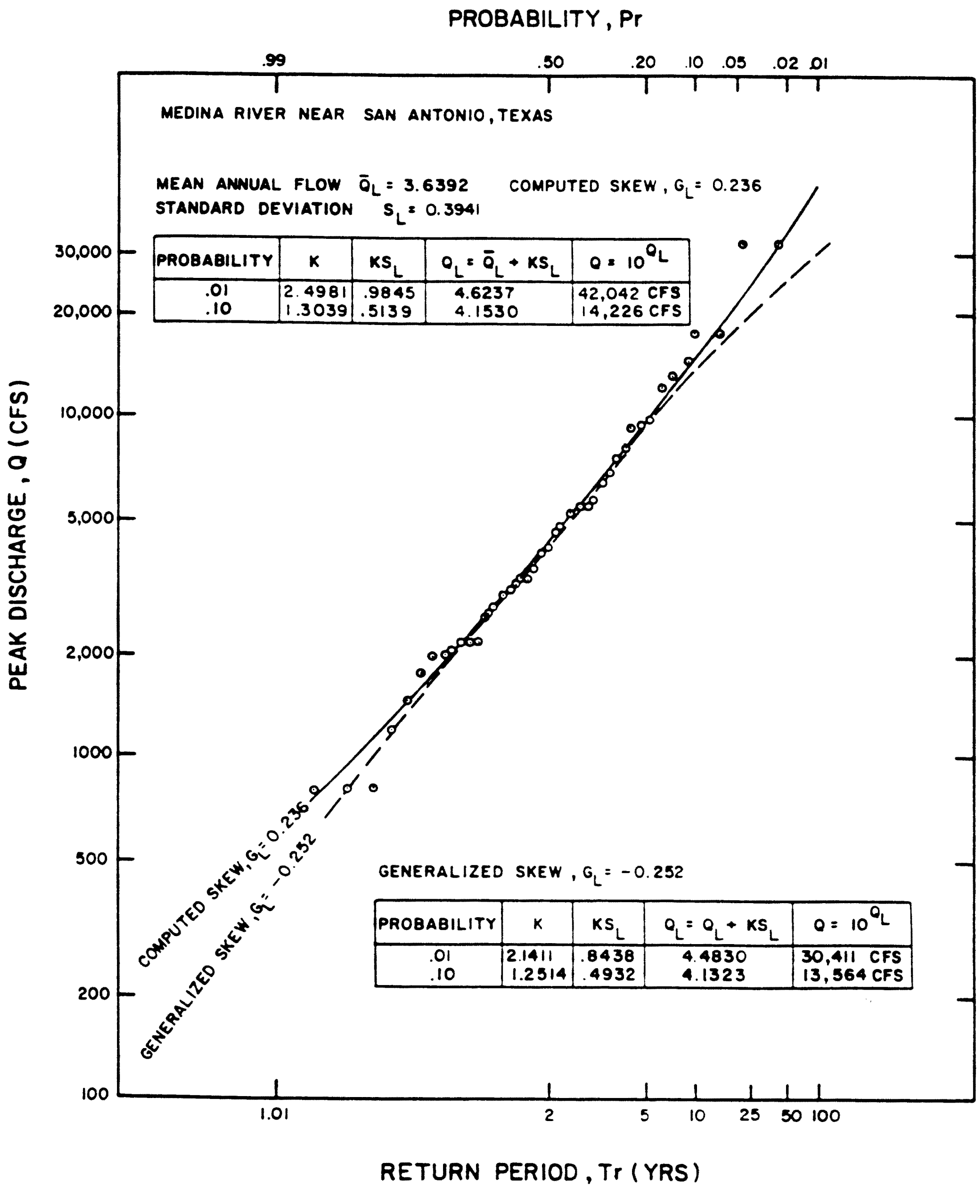

FIGURE 61.1 Log Pearson type III distribution analysis, Medina River, TX.

Values of this distribution function can be computed from Eq. (61.5). Characteristics of the Gumbel extreme value distribution are that the mean flow, $\bar{Q}$, occurs at the return period of T_r = 2.33 years and that it is skewed toward the high flows or extreme values as shown in the example of Figure 61.2. Even though it does not account directly for the computed skew of the data, it does predict the high flows reasonably well. For this method and additional techniques, please refer to USGS Water Supply Paper 1543-A, Flood-Frequency Analysis, and Manual of Hydrology Part 3.

The Gumbel extreme value distribution is given in "Statistics of Extremes" by E.J. Gumbel and is also found in HEC-19, p.73. Results from this method should be plotted on special Gumbel paper as shown in Figure 61.2.

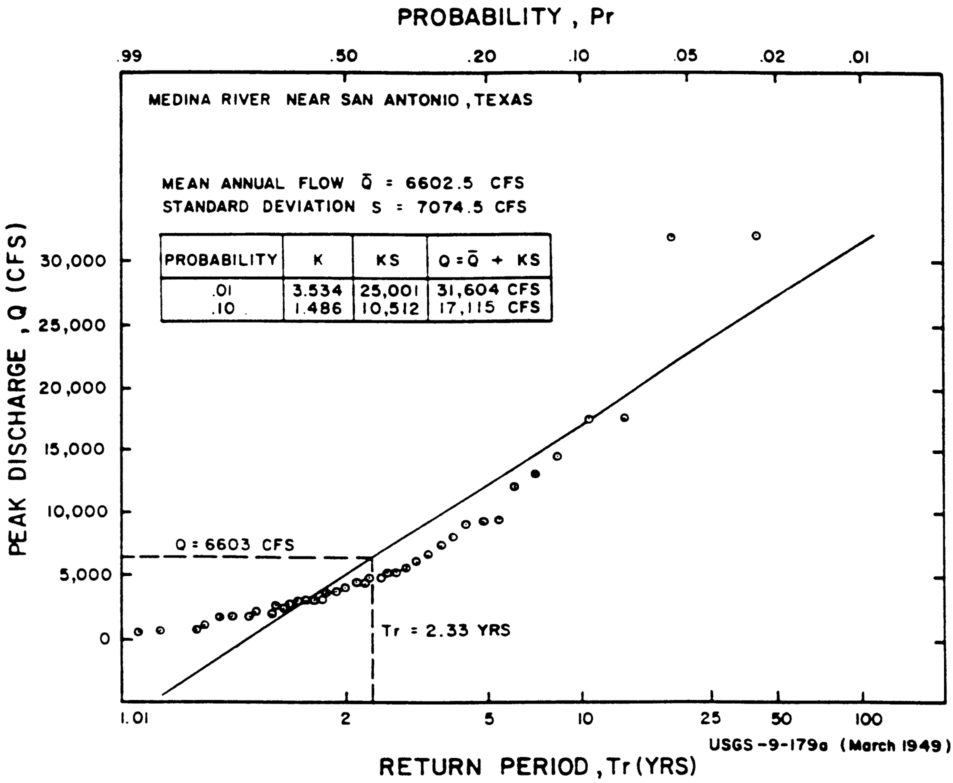

FIGURE 61.2 Gumbel extreme value frequency distribution analysis, Medina River, TX.

61.2.1.3.3 Discharge Comparison of Adjacent Basins

HEC 19, Appendix D [16] contains a list of reports for various states in the United States that have discharges at gauges that have been determined for frequencies from 2-year through 100-year frequencies. The discharges were determined by the Log Pearson III method. The discharge frequency at the gauges should be updated by the engineer using Log Pearson III and the Gumbel extreme value method.

The gauge data can be used directly as equivalent if the drainage areas are about the same (within less than 5%). Otherwise, the discharge determination can be obtained by the formula:

$$Q_u = Q_g (A_u / A_g)^b \tag{61.6}$$

where

Q_u = discharge at ungauged site

Q_g = discharge at gauged site

A_u = area of ungauged site

A_g = area of gauged site

b = exponent of drainage area

61.2.1.3.4 Regional Flood-Frequency Equations

If no gauged site is reasonably nearby, or if the record for the gauge is too short, then the discharge can be computed using the applicable regional flood-frequency equations. Statewide regional regression equations have been established in the United States. These equations permit peak flows to be estimated for return periods varying between 2 and 100 years. The discharges were determined by the Log Pearson III method. See HEC-19, Appendix D [16] for references to the studies that were conducted for the various states.

61.2.1.3.5 Design Hydrographs

Design hydrographs [9] give a complete time history of the passage of a flood at a particular site. This would include the peak flow. A runoff hydrograph is a plot of the response of a watershed to a particular rainfall event. A unit hydrograph is defined as the direct runoff hydrograph resulting from a rainfall event that lasts for a unit duration of time. The ordinates of the unit hydrograph are such that the volume of direct runoff represented by the area under the hydrograph is equal to 1 in. of runoff from the drainage area. Data on low water discharges and dates should be given as it will control methods and procedures of pier excavation and construction. The low water discharges and dates can be found in the USGS Water Resources Data Reports published each year. One procedure is to review the past 5 or 6 years of records to determine this.

61.2.1.4 Remarks

Before arriving at a final discharge, the existing channel capacity should be checked using the velocity as calculated times the channel waterway area. It may be that a portion of the discharge overflows the banks and never reaches the site.

The proposed design discharge should also be checked to see that it is reasonable and practicable. As a rule of thumb, the unit runoff should be 300 to 600 s-ft per square mile for small basins (to 20 square miles), 100 to 300 s-ft per square mile for median areas (to 50 square miles) and 25 to 150 s-ft for large basins (above 50 square miles). The best results will depend on rational engineering judgment.

61.2.2 Bridge Deck Drainage Design (On-Site Drainage Design)

61.2.2.1 Runoff and Capacity Analysis

The preferred on-site hydrology method is the rational method. The rational method, as discussed in Section 61.2.1.3.1, for on-site hydrology has a minimum time of concentration of 10 min. Many times, the time of concentration for the contributing on-site pavement runoff is less than 10 min. The initial time of concentration can be determined using an *overland flow* method until the runoff is concentrated in a curbed section. Channel flow using the roadway-curb cross section should be used to determine velocity and subsequently the time of flow to the first inlet. The channel flow velocity and flooded width is calculated using Manning's formula:

$$V = \frac{1.486}{n} A R^{2/3} S_f^{1/2} \tag{61.7}$$

where

V = velocity
A = cross-sectional area of flow
R = hydraulic radius
S_f = slope of channel
n = Manning's roughness value [11]

The intercepted flow is subtracted from the initial flow and the bypass is combined with runoff from the subsequent drainage area to determine the placement of the next inlet. The placement of inlets is determined by the allowable flooded width on the roadway.

Oftentimes, bridges are in sump areas, or the lowest spot on the roadway profile. This necessitates the interception of most of the flow before reaching the bridge deck. Two overland flow equations are as follows.

1. **Kinematic Wave Equation**:

$$t_o = \frac{6.92 L^{0.6} n^{0.6}}{i^{0.4} S^{0.3}} \tag{61.8}$$

2. **Overland Equation:**

$$t_o = \frac{3.3(1.1 - C)(L)^{1/2}}{(100\,S)^{1/3}} \tag{61.9}$$

where
t_o = overland flow travel time in minutes
L = length of overland flow path in meters
S = slope of overland flow in meters
n = manning's roughness coefficient [12]
i = design storm rainfall intensity in mm/h
C = runoff coefficient (Tables 61.1 and 61.2)

61.2.2.2 Select and Size Drainage Facilities

The selection of inlets is based upon the allowable flooded width. The allowable flooded width is usually outside the traveled way. The type of inlet leading up to the bridge deck can vary depending upon the flooded width and the velocity. Grate inlets are very common and, in areas with curbs, curb opening inlets are another alternative. There are various monographs associated with the type of grate and curb opening inlet. These monographs are used to determine interception and therefore the bypass [5].

61.2.3 Stage Hydraulics

High water (HW) stage is a very important item in the control of the bridge design. All available information should be obtained from the field and the Bridge Hydrology Report regarding HW marks, HW on upstream and downstream sides of the existing bridges, high drift profiles, and possible backwater due to existing or proposed construction.

Remember, observed high drift and HW marks are not always what they seem. Drift in trees and brush that could have been bent down by the flow of the water will be extremely higher than the actual conditions. In addition, drift may be pushed up on objects or slopes above actual HW elevation by the velocity of the water or wave action. Painted HW marks on the bridge should be searched carefully. Some flood insurance rate maps and flood insurance study reports may show stages for various discharges. Backwater stages caused by other structures should be included or streams should be noted.

Duration of high stages should be given, along with the base flood stage and HW for the design discharge. It should be calculated for existing and proposed conditions that may restrict the channel producing a higher stage. Elevation and season of low water should be given, as this may control design of tremie seals for foundations and other possible methods of construction. Elevation of overtopping flow and its location should be given. Normally, overtopping occurs at the bridge site, but overtopping may occur at a low sag in the roadway away from the bridge site.

61.2.3.1 Waterway Analysis

When determining the required waterway at the proposed bridge, the engineers must consider all adjacent bridges if these bridges are reasonably close. The waterway section of these bridges should be tied into the stream profile of the proposed structure. Structures that are upstream or downstream of the proposed bridge may have an impact on the water surface profile. When calculating the effective waterway area, adjustments must be made for the skew and piers and bents. The required waterway should be below the 50-year design HW stage.

If stream velocities, scour, and erosive forces are high, then abutments with wingwall construction may be necessary. Drift will affect the horizontal clearance and the minimum vertical clearance line of the proposed structure. Field surveys should note the size and type of drift found in the channel. Designs based on the 50-year design discharge will require drift clearance. On major streams and rivers, drift clearance of 2 to 5 m above the 50-year discharge is needed. On smaller streams 0.3 to 1 m may be adequate. A formula for calculating freeboard is

$$\text{Freeboard} = 0.1Q^{0.3} + 0.008V^2 \tag{61.10}$$

where
Q = discharge
V = velocity

61.2.3.2 Water Surface Profile Calculation

There are three prominent water surface profile calculation programs available [1,2]. The first one is HEC-2 which takes stream cross sections perpendicular to the flow. WSPRO is similar to HEC-2 with some improvements. SMS is a new program that uses finite-element analysis for its calculations. SMS can utilize digital elevation models to represent the streambeds.

61.2.2.3 Flow Velocity and Distribution

Mean channel, overflow velocities at peak stage, and localized velocity at obstructions such as piers should be calculated or estimated for anticipated high stages. Mean velocities may be calculated from known stream discharges at known channel section areas or known waterway areas of bridge, using the correct high water stage.

Surface water velocities should be measured roughly, by use of floats, during field surveys for sites where the stream is flowing. Stream velocities may be calculated along a uniform section of the channel using Manning's formula Eq. (61.7) if the slope, channel section (area and wetted perimeter), and roughness coefficient (n) are known.

At least three profiles should be obtained, when surveying for the channel slope, if possible. These three slopes are bottom of the channel, the existing water surface, and the HW surface based on drift or HW marks. The top of low bank, if overflow is allowed, should also be obtained. In addition, note some tops of high banks to prove flows fall within the channel. These profiles should be plotted showing existing and proposed bridges or other obstruction in the channel, the change of HW slope due to these obstructions and possible backwater slopes.

The channel section used in calculating stream velocities should be typical for a relatively long section of uniform channel. Since this theoretical condition is not always available, however, the nearest to uniform conditions should be used with any necessary adjustments made for irregularities.

Velocities may be calculated from PC programs, or calculator programs, if the hydraulic radius, roughness factor, and slope of the channel are known for a section of channel, either natural or artificial, where uniform stream flow conditions exists. The hydraulic radius is the waterway area divided by the wetted perimeter of an average section of the uniform channel. A section under a bridge whose piers, abutments, or approach fills obstruct the uniformity of the channel cannot be used as there will not be uniform flow under the structure. If no part of the bridge structure seriously obstructs or restricts the channel, however, the section at the bridge could be used in the above uniform flow calculations.

The roughness coefficient n for the channel will vary along the length of the channel for various locations and conditions. Various values for n can be found in the References [1,5,12,17].

At the time of a field survey the party chief should estimate the value of n to be used for the channel section under consideration. Experience is required for field determination of a relatively close to actual n value. In general, values for natural streams will vary between 0.030 and 0.070. Consider both low and HW n value. The water surface slope should be used in this plot and the slope should be adjusted for obstructions such as bridges, check dams, falls, turbulence, etc.

The results as obtained from this plot may be inaccurate unless considerable thought is given to the various values of slope, hydraulic radius, and n. High velocities between 15 and 20 ft/s (4.57 and 6.10 m/s through a bridge opening may be undesirable and may require special design considerations. Velocities over 20/ 6.10 m/s should not be used unless special design features are incorporated or if the stream is mostly confined in rock or an artificial channel.

61.3 Bridge Scour

61.3.1 Bridge Scour Analysis

61.3.1.1 Basic Scour Concepts

Scour is the result of the erosive action of flowing water, excavating and carrying away material from the bed and banks of streams. Determining the magnitude of scour is complicated by the cyclic nature of the scour process. Designers and inspectors need to study site-specific subsurface information carefully in evaluating scour potential at bridges. In this section, we present bridge engineers with the basic procedures and methods to analyze scour at bridges.

Scour should be investigated closely in the field when designing a bridge. The designer usually places the top of footings at or below the total potential scour depth; therefore, determining the depth of scour is very important. The total potential scour at a highway crossing usually comprises the following components [11]: aggradation and degradation, stream contraction scour, local scour, and sometimes with lateral stream migration.

61.3.1.1.1 Long-Term Aggradation and Degradation

When natural or human activities cause streambed elevation changes over a long period of time, aggradation or degradation occurs. Aggradation involves the deposition of material eroded from the channel or watershed upstream of the bridge, whereas degradation involves the lowering or scouring of the streambed due to a deficit in sediment supply from upstream.

Long-term streambed elevation changes may be caused by the changing natural trend of the stream or may be the result of some anthropogenic modification to the stream or watershed. Factors that affect long-term bed elevation changes are dams and reservoirs up- or downstream of the bridge, changes in watershed land use, channelization, cutoffs of meandering river bends, changes in the downstream channel base level, gravel mining from the streambed, diversion of water into or out of the stream, natural lowering of the fluvial system, movement of a bend, bridge location with respect to stream planform, and stream movement in relation to the crossing. Tidal ebb and flood may degrade a coastal stream, whereas littoral drift may cause aggradation. The problem for the bridge engineer is to estimate the long-term bed elevation changes that will occur during the lifetime of the bridge.

61.3.1.1.2 Stream Contraction Scour

Contraction scour usually occurs when the flow area of a stream at flood stage is reduced, either by a natural contraction or an anthropogenic contraction (like a bridge). It can also be caused by the overbank flow which is forced back by structural embankments at the approaches to a bridge. There are some other causes that can lead to a contraction scour at a bridge crossing [11]. The decreased flow area causes an increase in average velocity in the stream and bed shear stress through the contraction reach. This in turn triggers an increase in erosive forces in the contraction. Hence, more bed material is removed from the contracted reach than is transported into the reach. The natural streambed elevation is lowered by this contraction phenomenon until relative equilibrium is reached in the contracted stream reach.

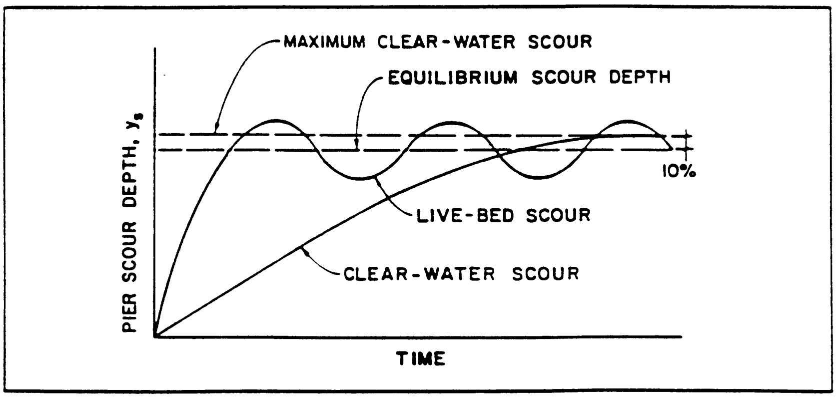

FIGURE 61.3 Illustrative pier scour depth in a sand-bed stream as a function of time.

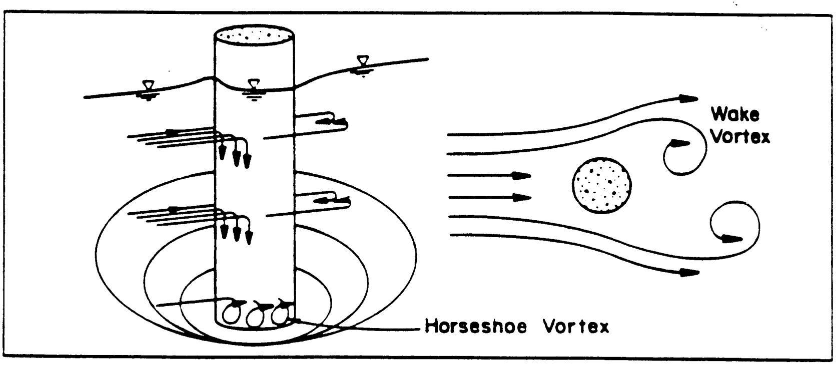

FIGURE 61.4 Schematic representation of local scour at a cylindrical pier.

There are two forms of contraction scour: live-bed and clear-water scours. Live-bed scour occurs when there is sediment being transported into the contracted reach from upstream. In this case, the equilibrium state is reached when the transported bed material out of the scour hole is equal to that transported into the scour hole from upstream. Clear-water scour occurs when the bed sediment transport in the uncontracted approach flow is negligible or the material being transported in the upstream reach is transported through the downstream at less than the capacity of the flow. The equilibrium state of the scour is reached when the average bed shear stress is less than that required for incipient motion of the bed material in this case (Figure 61.3).

61.3.1.1.3 Local Scour

When upstream flow is obstructed by obstruction such as piers, abutments, spurs, and embankments, flow vortices are formed at their base as shown in Figure 61.4 (known as horseshoe vortex). This vortex action removes bed material from around the base of the obstruction. A scour hole eventually develops around the base. Local scour can also be either clear-water or live-bed scour. In considering local scour, a bridge engineer needs to look into the following factors: flow velocity, flow depth, flow attack angle to the obstruction, obstruction width and shape, projected length of the obstruction, bed material characteristics, bed configuration of the stream channel, and also potential ice and debris effects [11, 13].

61.3.1.1.4 Lateral Stream Migration

Streams are dynamic. The lateral migration of the main channel within a floodplain may increase pier scour, embankment or approach road erosion, or change the total scour depth by altering the flow angle of attack at piers. Lateral stream movements are affected mainly by the geomorphology of the stream, location of the crossing on the stream, flood characteristics, and the characteristics of the bed and bank materials [11,13].

61.3.1.2 Designing Bridges to Resist Scour

It is obvious that all scour problems cannot be covered in this special topic section of bridge scour. A more-detailed study can be found in HEC-18, "Evaluating Scour at Bridges" and HEC-20, "Stream Stability at Highway Structures" [11,18]. As described above, the three most important components of bridge scour are long-term aggradation or degradation, contraction scour, and local scour. The total potential scour is a combination of the three components. To design a bridge to resist scour, a bridge engineer needs to follow the following observation and investigation steps in the design process.

1. **Field Observation** — Main purposes of field observation are as follows:
   - Observe conditions around piers, columns, and abutments (Is the hydraulic skew correct?),
   - Observe scour holes at bends in the stream,
   - Determine streambed material,
   - Estimate depth of scour, and
   - Complete geomorphic factor analysis.

   There is usually no fail-safe method to protect bridges from scour except possibly keeping piers and abutments out of the HW area; however, proper hydraulic bridge design can minimize bridge scour and its potential negative impacts.
2. **Historic Scour Investigation** — Structures that have experienced scour in the past are likely to continue displaying scour problems in the future. The bridges that we are most concerned with include those currently experiencing scour problems and exhibiting a history of local scour problems.
3. **Problem Location Investigation** — Problem locations include "unsteady stream" locations, such as near the confluence of two streams, at the crossing of stream bends, and at alluvial fan deposits.
4. **Problem Stream Investigation** — Problem streams are those that have the following characteristics of aggressive tendencies: indication of active degradation or aggradation; migration of the stream or lateral channel movement; streams with a steep lateral slope and/or high velocity; current, past, or potential in-stream aggregate mining operations; and loss of bank protection in the areas adjacent to the structure.
5. **Design Feature Considerations** — The following features, which increase the susceptibility to local scour, should be considered:
   - Inadequate waterway opening leads to inadequate clearance to pass large drift during heavy runoff.
   - Debris/drift problem: Light drift or debris may cause significant scour problems, moderate drift or debris may cause significant scour but will not create severe lateral forces on the structure, and heavy drift can cause strong lateral forces or impact damage as well as severe scour.
   - Lack of overtopping relief: Water may rise above deck level. This may not cause scour problems but does increase vulnerability to severe damage from impact by heavy drift.
   - Incorrect pier skew: When the bridge pier does not match the channel alignment, it may cause scour at bridge piers and abutments.
6. **Traffic Considerations** — The amount of traffic such as average daily traffic (ADT), type of traffic, the length of detour, the importance of crossings, and availability of other crossings should be taken into consideration.

7. **Potential for Unacceptable Damage** — Potential for collapse during flood, safety of traveling public and neighbors, effect on regional transportation system, and safety of other facilities (other bridges, properties) need to be evaluated.
8. **Susceptibility of Combined Hazard of Scour and Seismic** — The earthquake prioritization list and the scour-critical list are usually combined for bridge design use.

61.3.1.3 Scour Rating

In the engineering practice of the California Department of Transportation, the rating of each structure is based upon the following:

1. **Letter grading** — The letter grade is related to the potential for scour-related problems at this location.
2. **Numerical grading** — The numerical rating associated with each structure is a determination of the severity for the potential scour:

   A-1 No problem anticipated
   A-2 No problem anticipated/new bridge — no history
   A-3 Very remote possibility of problems
   B-1 Slight possibility of problems
   B-2 Moderate possibility of problems
   B-3 Strong possibility of problems
   C-1 Some probability of problems
   C-2 Moderate probability of problems
   C-3 Very strong probability of problems

Scour effect of storms is usually greater than design frequency, say, 500-year frequency. FHWA specifies 500-year frequency as 1.7 times 100-year frequency. Most calculations indicate 500-year frequency is 1.25 to 1.33 times greater than the 100-year frequency [3,8]; the 1.7 multiplier should be a maximum. Consider the amount of scour that would occur at overtopping stages and also pressure flows. Be aware that storms of lesser frequency may cause larger scour stress on the bridge.

61.3.2 Bridge Scour Calculation

All the equations for estimating contraction and local scour are based on laboratory experiments with limited field verification [11]. However, the equations recommended in this section are considered to be the most applicable for estimating scour depths. Designers also need to give different considerations to clear-water scour and live-bed scour at highway crossings and encroachments.

Prior to applying the bridge scour estimating methods, it is necessary to (1) obtain the fixed-bed channel hydraulics, (2) determine the long-term impact of degradation or aggradation on the bed profile, (3) adjust the fixed-bed hydraulics to reflect either degradation or aggradation impact, and (4) compute the bridge hydraulics accordingly.

61.3.2.1 Specific Design Approach

Following are the recommended steps for determining scour depth at bridges:

Step 1: Analyze long-term bed elevation change.
Step 2: Compute the magnitude of contraction scour.
Step 3: Compute the magnitude of local scour at abutments.
Step 4: Compute the magnitude of local scour at piers.
Step 5: Estimate and evaluate the total potential scour depths.

The bridge engineers should evaluate if the individual estimates of contraction and local scour depths from Step 2 to 4 are reasonable and evaluate the total scour derived from Step 5.

61.3.2.2 Detailed Procedures

1. **Analyze Long-Term Bed Elevation Change** — The face of bridge sections showing bed elevation are available in the maintenance bridge books, old preliminary reports, and sometimes in FEMA studies and U.S. Corps of Engineers studies. Use this information to estimate aggradation or degradation.
2. **Compute the Magnitude of Contraction Scour** — It is best to keep the bridge out of the normal channel width. However, if any of the following conditions are present, calculate contraction scour.
   a. Structure over channel in floodplain where the flows are forced through the structure due to bridge approaches
   b. Structure over channel where river width becomes narrow
   c. Relief structure in overbank area with little or no bed material transport
   d. Relief structure in overbank area with bed material transport

   The general equation for determining contraction scour is

$$y_s = y_2 - y_1 \tag{61.11}$$

   where
   y_s = depth of scour
   y_1 = average water depth in the main channel
   y_2 = average water depth in the contracted section

   Other contraction scour formulas are given in the November 1995 HEC-18 publication — also refer to the workbook or HEC-18 for the various conditions listed above [11]. The detailed scour calculation procedures can be referenced from this circular for either live-bed or clear-water contraction scour.
3. **Compute the Magnitude of Local Scour at Abutments** — Again, it is best to keep the abutments out of the main channel flow. Refer to publication HEC-18 from FHWA [13]. The scour formulas in the publication tend to give excessive scour depths.
4. **Compute the Magnitude of Local Scour at Piers** — The pier alignment is the most critical factor in determining scour depth. Piers should align with stream flow. When flow direction changes with stages, cylindrical piers or some variation may be the best alternative. Be cautious, since large-diameter cylindrical piers can cause considerable scour. Pier width and pier nose are also critical elements in causing excessive scour depth.

Assuming a sand bed channel, an acceptable method to determine the maximum possible scour depth for both live-bed and clear-water channel proposed by the Colorado State University [11] is as follows:

$$\frac{y_s}{y_1} = 2.0\,K_1 K_2 K_3 \left(\frac{a}{y_1}\right)^{0.65} F_r^{\,0.43} l \tag{61.12}$$

where
y_s = scour depth
y_1 = flow depth just upstream of the pier
K_1 = correction for pier shape from Figure 61.5 and Table 61.3
K_2 = correction for angle of attack of flow from Table 61.4
K_3 = correction for bed condition from Table 61.5
a = pier width
l = pier length
F_r = Froude number = $\dfrac{V}{(gy)}$ (just upstream from bridge)

Drift retention should be considered when calculating pier width/type.

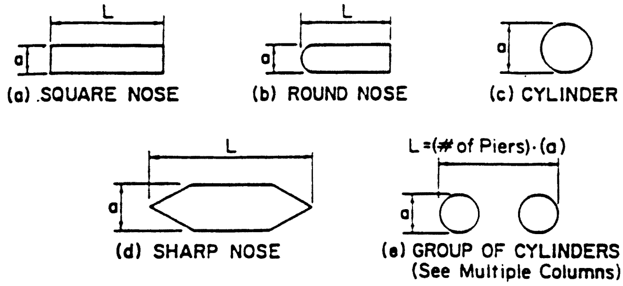

FIGURE 61.5 Common pier shapes.

TABLE 61.3 Correction Factor, K_1, for Pier Nose Shape

Shape of Pier Nose	K_1
Square nose	1.1
Round nose	1.0
Circular cylinder	1.0
Sharp nose	0.9
Group of cylinders	1.0

TABLE 61.4 Correction Factor, K_2, for Flow Angle of Attack

Angle	$L/a = 4$	$L/a = 8$	$L/a = 12$
0	1.0	1.0	1.0
15	1.5	2.0	2.5
30	2.0	2.75	3.5
45	2.3	3.3	4.3
90	2.5	3.9	5

TABLE 61.5 Increase in Equilibrium Pier Scour Depths K_3 for Bed Conditions

Bed Conditions	Dune Height H, ft	K_3
Clear-water scour	N/A	1.1
Plane bed and antidune flow	N/A	1.1
Small dunes	$10 > H > 2$	1.1
Medium dunes	$30 > H > 10$	1.1–1.2
Large dunes	$H > 30$	1.3

61.3.2.3 Estimate and Evaluate Total Potential Scour Depths

Total potential scour depths is usually the sum of long-term bed elevation change (only degradation is usually considered in scour computation), contraction scour, and local scour. Historical scour depths and depths of scourable material are determined by geology. When estimated depths from the above methods are in conflict with geology, the conflict should be resolved by the hydraulic engineer and the geotechnical engineer; based on economics and experience, it is best to provide for maximum anticipated problems.

61.3.3 Bridge Scour Investigation and Prevention

61.3.3.1 Steps to Evaluate Bridge Scour

It is recommended that an interdisciplinary team of hydraulic, geotechnical, and bridge engineers should conduct the evaluation of bridge scour. The following approach is recommended for evaluating the vulnerability of existing bridges to scour [11]:

Step 1. Screen all bridges over waterways into five categories: (1) low risk, (2) scour-susceptible, (3) scour-critical, (4) unknown foundations, or (5) tidal. Bridges that are particularly vulnerable to scour failure should be identified immediately and the associated scour problem addressed. These particularly vulnerable bridges are

1. Bridges currently experiencing scour or that have a history of scour problems during past floods as identified from maintenance records, experience, and bridge inspection records
2. Bridges over erodible streambeds with design features that make them vulnerable to scour
3. Bridges on aggressive streams and waterways
4. Bridges located on stream reaches with adverse flow characteristics

Step 2. Prioritize the scour-susceptible bridges and bridges with unknown foundations by conducting a preliminary office and field examination of the list of structures compiled in Step 1 using the following factors as a guide:

1. The potential for bridge collapse or for damage to the bridge in the event of a major flood
2. The functional classification of the highway on which the bridge is located.
3. The effect of a bridge collapse on the safety of the traveling public and on the operation of the overall transportation system for the area or region

Step 3. Conduct office and field scour evaluations of the bridges on the prioritized list in Step 2 using an interdisciplinary team of hydraulic, geotechnical, and bridge engineers:

1. In the United States, FHWA recommends using 500-year flood or a flow 1.7 times the 100-year flood where the 500-year flood is unknown to estimate scour [3,6]. Then analyze the foundations for vertical and lateral stability for this condition of scour. The maximum scour depths that the existing foundation can withstand are compared with the total scour depth estimated. An engineering assessment must be then made whether the bridge should be classified as a scour-critical bridge.
2. Enter the results of the evaluation study in the inventory in accordance with the instructions in the FHWA "Bridge Recording and Coding Guide" [7].

Step 4. For bridges identified as scour critical from the office and field review in Steps 2 and 3, determine a plan of action for correcting the scour problem (see Section 61.3.3.3).

61.3.3.2 Introduction to Bridge Scour Inspection

The bridge scour inspection is one of the most important parts of preventing bridge scour from endangering bridges. Two main objectives to be accomplished in inspecting bridges for scour are

1. To record the present condition of the bridge and the stream accurately; and
2. To identify conditions that are indicative of potential problems with scour and stream stability for further review and evaluation by other experts.

In this section, the bridge inspection practice recommended by U.S. FHWA [6,10] is presented for engineers to follow as guidance.

61.3.3.2.1 Office Review

It is highly recommended that an office review of bridge plans and previous inspection reports be conducted prior to making the bridge inspection. Information obtained from the office review provides a better foundation for inspecting the bridge and the stream. The following questions should be answered in the office review:

- Has an engineering scour evaluation been conducted? If so, is the bridge scour critical?
- If the bridge is scour-critical, has a plan of action been made for monitoring the bridge and/or installing scour prevention measures?
- What do comparisons of streambed cross sections taken during successive inspections reveal about the stream bed? Is it stable? Degrading? Aggrading? Moving laterally? Are there scour holes around piers and abutments?
- What equipment is needed to obtain stream-bed cross sections?
- Are there sketches and aerial photographs to indicate the planform locations of the stream and whether the main channel is changing direction at the bridge?
- What type of bridge foundation was constructed? Do the foundations appear to be vulnerable to scour?
- Do special conditions exist requiring particular methods and equipment for underwater inspections?
- Are there special items that should be looked at including damaged riprap, stream channel at adverse angle of flow, problems with debris, etc.?

61.3.3.2.2 Bridge Scour Inspection Guidance

The condition of the bridge waterway opening, substructure, channel protection, and scour prevention measures should be evaluated along with the condition of the stream during the bridge inspection. The following approaches are presented for inspecting and evaluating the present condition of the bridge foundation for scour and the overall scour potential at the bridge.

Substructure is the key item for rating the bridge foundations for vulnerability to scour damage. Both existing and potential problems with scour should be reported so that an interdisciplinary team can make a scour evaluation when a bridge inspection finds that a scour problem has already occurred. If the bridge is determined to be scour critical, the rating of the substructures should be evaluated to ensure that existing scour problems have been considered. The following items should be considered in inspecting the present condition of bridge foundations:

- Evidence of movement of piers and abutments such as rotational movement and settlement;
- Damage to scour countermeasures protecting the foundations such as riprap, guide banks, sheet piling, sills, etc.;
- Changes in streambed elevation at foundations, such as undermining of footings, exposure of piles; and
- Changes in streambed cross section at the bridge, including location and depth of scour holes.

In order to evaluate the conditions of the foundations, the inspectors should take cross sections of the stream and measure scour holes at piers and abutments. If equipment or conditions do not permit measurement of the stream bottom, it should be noted for further investigation.

To take and plot measurement of stream bottom elevations in relation to the bridge foundations is considered the single most important aspect of inspecting the bridge for actual or potential damage from scour. When the stream bottom cannot be accurately measured by conventional means, there are other special measures that need to be taken to determine the condition of the substructures or foundations such as using divers and using electronic scour detection equipment. For the purposes of evaluating resistance to scour of the substructures, the questions remain essentially the same for foundations in deep water as for foundations in shallow water [7] as follows:

- How does the stream cross section look at the bridge?
- Have there been any changes as compared with previous cross section measurements? If so, does this indicate that (1) the stream is aggrading or degrading or (2) is local or contraction scour occurring around piers and abutments?
- What are the shapes and depths of scour holes?
- Is the foundation footing, pile cap, or the piling exposed to the stream flow, and, if so, what is the extent and probable consequences of this condition?
- Has riprap around a pier been moved or removed?

Any condition that a bridge inspector considers to be an emergency or of a potentially hazardous nature should be reported immediately. This information as well as other conditions, which do not pose an immediate hazard but still warrant further investigation, should be conveyed to the interdisciplinary team for further review.

61.3.3.3 Introduction to Bridge Scour Prevention

Scour prevention measures are generally incorporated after the initial construction of a bridge to make it less vulnerable to damage or failure from scour. A plan of preventive action usually has three major components [11]:

1. Timely installation of temporary scour prevention measures;
2. Development and implementation of a monitoring program;
3. A schedule for timely design and construction of permanent scour prevention measures.

For new bridges [11], the following is a summary of the best solutions for minimizing scour damage:

1. Locating the bridge to avoid adverse flood flow patterns;
2. Streamlining bridge elements to minimize obstructions to the flow;
3. Designing foundations safe from scour;
4. Founding bridge pier foundations sufficiently deep to not require riprap or other prevention measures; and
5. Founding abutment foundations above the estimated local scour depth when the abutment is protected by well-designed riprap or other suitable measures.

For existing bridges, the available scour prevention alternatives are summarized as follows:

1. Monitoring scour depths and closing the bridge if excessive bridge scour exists;
2. Providing riprap at piers and/or abutments and monitoring the scour conditions;
3. Constructing guide banks or spur dikes;
4. Constructing channel improvements;
5. Strengthening the bridge foundations;
6. Constructing sills or drop structures; and
7. Constructing relief bridges or lengthening existing bridges.

These scour prevention measures should be evaluated using sound hydraulic engineering practice. For detailed bridge scour prevention measures and types of prevention measures, refer to Chapter 7 of "Evaluating Scour at Bridges" from U.S. FHWA. [10,11,18,19].

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